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nhc
northwest hydraulic consultants ltd

NHC Ref. No. 3006489

2021 September 30
Revised February 7, 2022

EPTA DEVELPOMENT CORP.
1910-1177 West Hastings St
Vancouver, BC, V6E 2K3

Attention: Alex Bill Tsakumis, Principal
Copy to: Rod Gonzales, P.Eng., Hub
Via email: rod@hub-inc.com

Re: Katzi First Nation
Bonson Road Stormwater Outfall Design

1.0 Introduction

EPTA Development Corp. is developing the Eagle Meadows Business Park in the Katzie First Nation in Pitt Meadows, BC. Hub Engineering is providing civil engineering services for the proposed development. The development is located on the north bank (right side) of the Fraser River. and downstream of the Golden Ears Bridge. The project is to include a stormwater outfall that runs north to south along Bonson Road, to discharges into the Fraser River. The outlet is located roughly 900 m downstream of the Golden Ears Bridge, 400 m downstream of the upstream end of Barnston Island, and less than 20 m downstream of an exiting pier and dock gangway. A viewing platform is located less than 10 m downstreamof the proposed outfall site (**Figure 1**).

Northwest Hydraulic Consultants Ltd. (NHC) was retained by EPTA Development Corp. to provide hydrotechnical engineering services for the Bonson Road stormwater outfall design. This letter report presents the design of the outfall.

Figure 1. **Location of the project site and proposed outfall (Google, 2019).**

1.1 Scope of the Work

The following tasks were completed as part of NHC's scope of work:

- Assessment of current site condition concerning bank stability, access, and general configuration based on-site visit.
- Geomorphic assessment of Fraser River reach close to the outfall location to consider changes in thawleg location, width and depth of the channel.
- Assessment of Fraser River channel hydraulics during design flow events to evaluate erosion and scour potential.
- Hydrotechnical design of the outfall including backflow prevention, headwall structure, and bouyancy.
- Hydrotechnical design of the bank armouring to withstand the expected erosion, scour, and geomorphic channel changes.

1.2 Design Guides and Codes

The following design codes and reference documents were used for the design basis:

- BC MOTI Supplement to TAC Geometric Design Guide (2007).
- TAC Guide to Bridge Hydraulics (2004).
- Province of British Columbia Riprap Design and Construction Guide (NHC 2000)

- Province of British Columbia Dike Design & Construction Guide: Best Management Practices (MWLAP, 2003).
- Dike Maintenance Act Approval for Pipe Crossings of Dikes (MFLNRO 2014).

2.0 Site Investigation

A site visit was conducted by Dale Muir and Hanna Hamid of NHC on March 12, 2021, between 1:00 and 2.30 pm (PST). The investigation concentrated on the Fraser River foreshore at Bonson Road. Observations of the channel, bank stability, existing bank protection and signs of past erosion along the bank line were made. The estimated water level at the site during the field visit was approximately El. -1.1 m (CGVD28) based on Avadepth **(Canadian Coast Guard, n.d.)**. For this report, the site description has been limited to the right bank from the dock upstream of the site (south end of Bonson Road) to the existing stormwater outfall located 280 m downstream.

The site is located at the inside of a relatively sharp bend as the channel gradually constricts as it transitions to two channels, one on each side of Barnston Island (**Figure 2**).

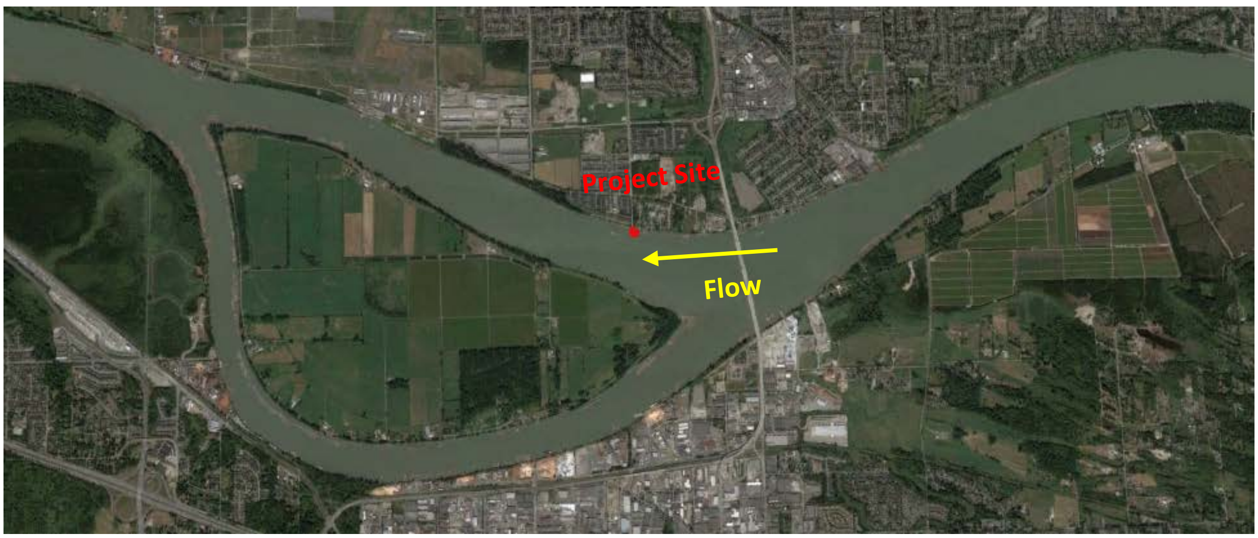

Figure 2. Location of the site (red dot) on the inside bend as the Fraser River splits around Barnston Island, flow is right (east) to left (west), (Google, 2020).

A shallow sloping bench of sediment (roughly 1% slope) with a width of 6 to 30 m extends from the bank towards the centre of the channel. This bench is present from Golden Ears Bridge to 2.5 to 3 km downstream of the site **(Figure 3)**. At the proposed outfall site, the bench is fairly narrow (close to 6 m) with a 5 m wide apron of angular shot rock (150 mm minus) 4 to 5 m wide. The bank projects up from the apron with a slope of roughly 1.5H:1V and appears to be 2.5 to 3 m high. Ageing rock riprap (D50 = 300 to 400 mm) provides limited protection of the bank; mainly the lower portion of the bank, with dense vegetation and signs of limited ongoing erosion higher up the slope (**Photo 1** and **Photo 2**). The underlying material appears to be consolidated silty clay river deposits.

Figure 3. Satellite image showing the sediment bench along the base of the bank (Google, 2020).

Photo 1. Existing bank protection just at the south of Bonson road. Signs of erosion can be seen in the unprotected bank in the background of the dock.

Photo 2. Nearly vertical bank and failing riprap, 30 m downstream from the dock.

On the top of the bank is a relatively flat floodplain. The floodplain is developed with Bonson Road and residential homes. The homes upstream (east) of Bonson Road appear to have existed since at least the mid-1990s (based on the review of available satellite images), and quite likely, substantially earlier. These homes are relatively close to the bank (20 to 30 m) with River Road providing separation between the homes and the Fraser River. The homes downstream of the site were constructed from 2003 to 2010. These homes are set back roughly 35 to 45 m from the top of the bank. Grass parkland, a pedestrian path, and grass surfaced dike (roughly 1 m high) surface the floodplain between the downstream homes and the top of the bank.

The existing bank armouring and outfall structure downstream of the site appears to be relatively newly constructed; likely installed when the top of bank development was constructed (mid-2000's). For the most part, the riprap downstream of the proposed site appears to be slightly larger (D50 = 350 to 450 mm), a bit lower slope (1.5 to 2H:1V) and in better condition than the riprap at the site (**Photo 3**). At the outfall, the riprap shows signs of movement and localized failure. The bank has not substantially eroded, but underlying geotextile and underlying silty-clay banks have been exposed (**Photo 4**).

Photo 3. Larger-sized riprap near the existing stormwater outfall. Large rocks in the water suggest signs of failure or loss of stones during placement.

Photo 4. Riprap failure exposing the filter fabric and underlying soil at the existing stormwater outfall location.

3.0 Geomorphic Assessment

3.1 Site Overview

The Fraser River drains 232,000 km^2 of south-central British Columbia, Canada and discharges into the Strait of Georgia at Sand Heads on the southwest coast of the province. The study site is located approximately 52 km upstream of Sand Heads. Peak flow and sediment transport occur in mid to late spring due to snowmelt in the interior basin. The mean daily discharge (Q) between 1986 to 2016, derived from Water Survey of Canada (WSC) station gauge 08MF005 (100 km upstream of the study site at Hope, BC), varies between 1,010 m^3 sec^{-1} during winter (December to February), to 5,740 m^3 sec^{-1} during freshet (May to July) with a mean annual flood peak of 8,420 m^3 sec^{-1}.

Downstream from Mission, BC, the Fraser River is sand bedded, flowing in an irregular, sinuous pattern with large vegetated islands, and transitions to a modern delta near New Westminster. The proposed location for the Bonson Road Stormwater outfall is approximately 30 km downstream from Mission and 17 km upstream of New Westminster on the north bank in Bishops Reach **(Figure 4).** The main channel upstream of Barnston Island (Derby Reach) has an approximate width of 500 m. Immediately downstream of Golden Ears Bridge (constructed from 2006 to 2009), the river widens to approximately 1,000 m. As it approaches Barnston Island, flow splits between the main channel (Bishops Reach) and the smaller southern branch (Parsons Channel). Parsons Channel is approximately 300 m wide at the entrance and the Bishops Reach is about 500 m wide. Discharge measurements by Public Service and Procurement Canada (PSPC; formerly Public Works and Government Services Canada) collected in May-June 2005 and November 2006 to January 2006 indicated the Parsons Channel carries 30% of the flow and Bishops Reach carries 70% of the flow (NHC, 2006). The proposed outfall location is in a relatively shallow part of the channel, as flow passing Golden Ears Bridge is directed to the north side of Barnston Island, where the lower bed elevation provides evidence of scouring **(Figure 4)**.

The location of the proposed outfall on the inside bend of the reach combined with the extensive river training and bank armouring, suggests a limited risk of large-scale change, thalweg shifts or meander migrations which could result in increased depth at the study site. Historic trends in bed levels to assess patterns of aggradation and degradation were reviewed in further detail below.

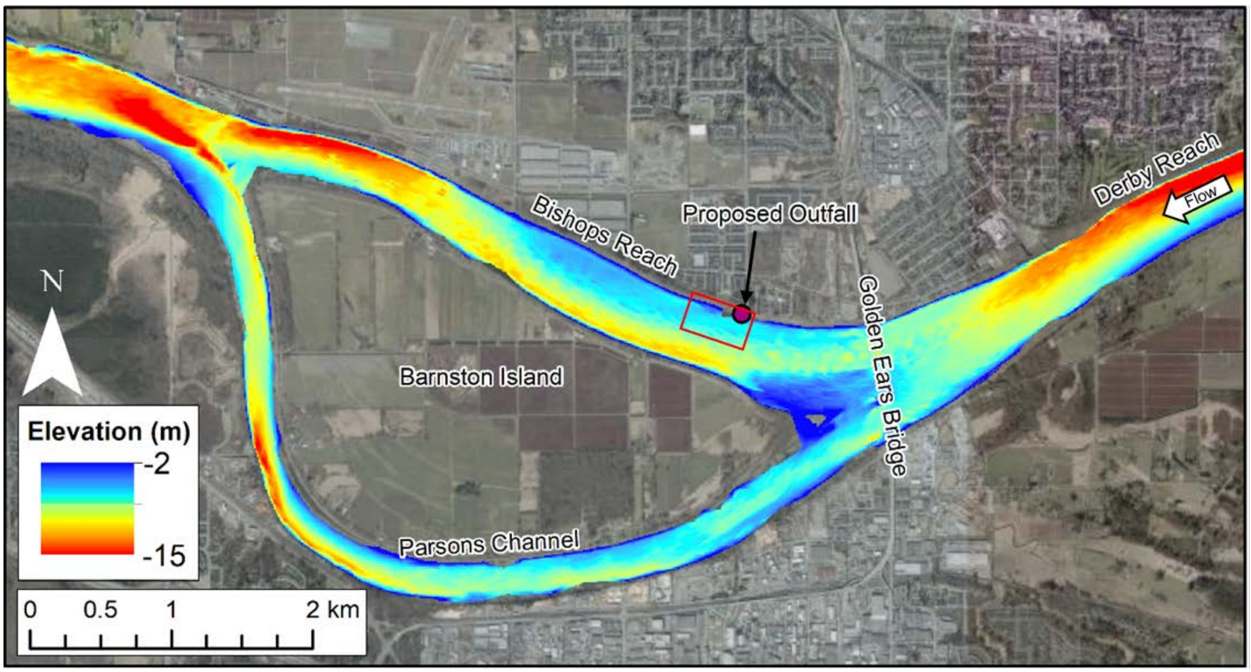

Figure 4. Bathymetry around Barnston Island derived from February/March 2019 single beam survey from PSPC. The red box indicates the location of the analysis shown in Figure 6.

3.2 Available Data

Bathymetric data with coverage near the proposed outfall region are available from PSPC and the Canadian Hydrographic Survey (CHS) **(Table 1)**. Two PSPC surveys from 1951 and 1984 and a 1991 survey from CHS have been digitizing from physical bathymetric maps, while the other surveys from 1998 to 2019 are provided digitally from PSPC. The single-beam surveys with coverage near the proposed outfall location are generally from lower winter flows. Single beam surveys are generally 50 to 100 m apparent but have been gridded at a 10 m resolution here. All elevations are reported to Canadian Geodetic Vertical Datum 1928 (CGVD28).

Table 1. Single beam bathymetric surveys available in Bishops Reach.

Source	Year	Date	Survey Type	Discharge (m^3/s)
PSPC	1951	Unknown	Single beam	
PSPC	1984	Unknown	Single beam	
CHS	1991	Unknown	Single beam	
PSPC	1998	Mar 29/31, April 8/15	Single beam	1,500 to 1,480
PSPC	2003	Oct 8/28/29, Nov 3/4	Single beam	1,180 to 2,730
PSPC	2007	Feb 14/15/22/27	Single beam	739 to 708
PSPC	2010	Oct 6/7, Nov 1	Single beam	2,330 to 1,630
PSPC	2016	May 10	Single beam	5,610
PSPC	2019	Feb 26/28, Mar 1	Single beam	698 to 634

3.3 Long-term Spatial Bed Trends

The spatial difference maps in **Figure 5** allow a comparison between bed levels observed in 2019 to those from 1951, 1996, 2010 and 2016. Overall, the 2019 channel is up to 3 to 4 m lower in this reach than it was in 1951 **(Figure 5a)**, consistent with long-term degradation observed in reaches downstream of this site. The degradation has been linked to dredging activity and the installation of river training structures (Nelson et al., 2017). Comparisons to more recent years (1996, 2010, 2016) show localized patterns of deposition and erosion rather than widespread degradation. Upstream of the bridge, the bed was about 1 to 2 m lower in 2019 compared to 1996, while some deposition is evident on the north side of Barnston Island (**Figure 5b)**. The area within an approximate 500 m radius of the proposed outfall location does not show any obvious erosion or deposition. The 2019 to 2010 difference map **(Figure 5c)** reveals some widespread moderate deposition on the order of about 1 m around the study site, with substantial erosion (> 3 m) on the left bank (opposite side of the river). Since the construction of the Golden Ears Bridge was completed in 2009, these changes likely reflect local effects on the flow caused by the bridge. There is some evidence of bed lowering in Bishops Channel of less than 1 m from 2019 to 2016 localized on the south side of the channel, but overall there is no evidence of substantial changes to the bed, over the 3 years **(Figure 5d).**

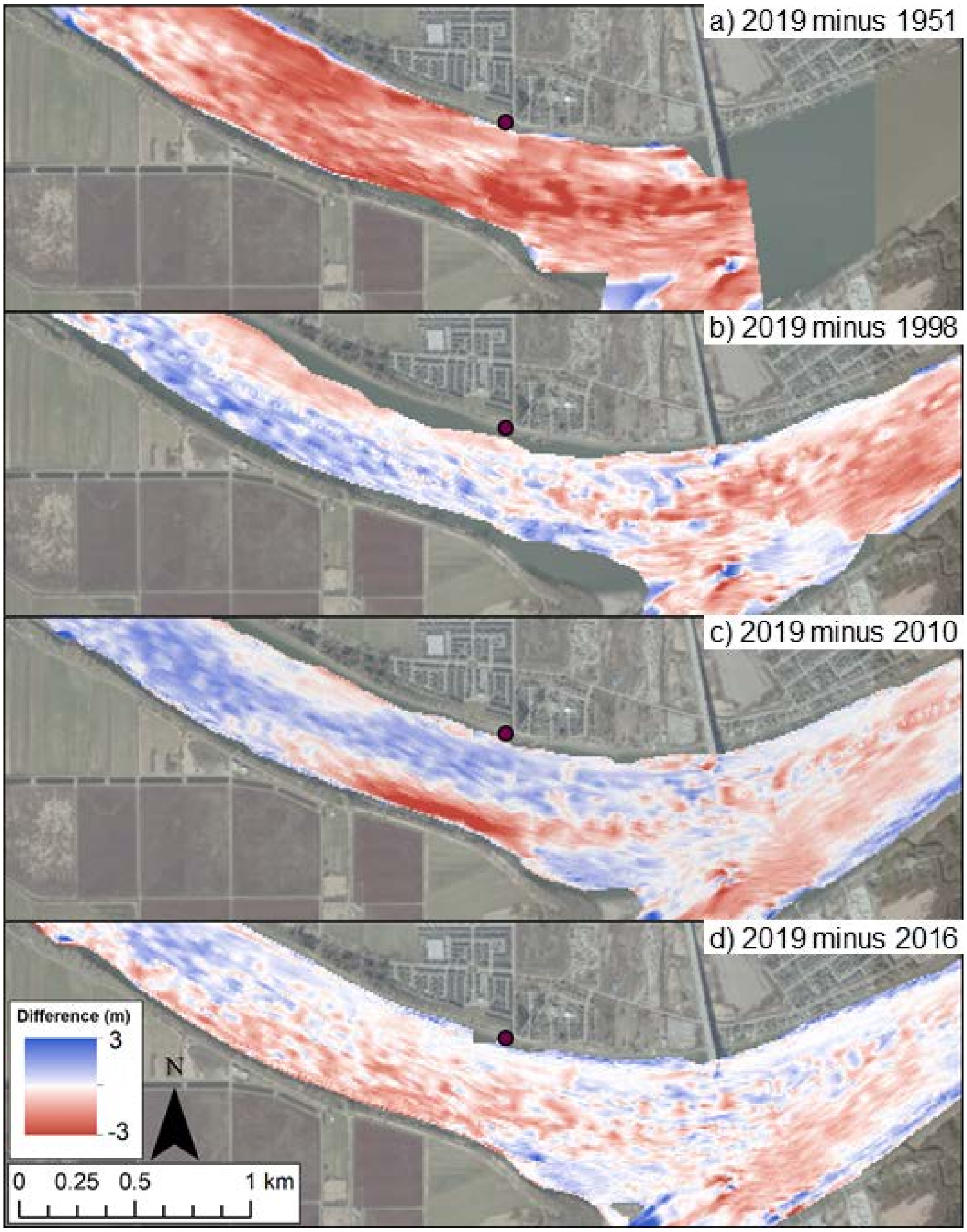

Figure 5. Bed level elevation changes from (a) 2019 to 1951, (b) 2019 to 1998, (c) 2019 to 2010, and (d) 2019 to 2010. Blue indicates deposition while red shows erosion.

3.4 Localized Historic Bed Levels

While the difference maps in **Figure 5** allow for an examination of spatial changes between two periods in time, they do not provide much insight into potential systematic degradation (erosion) or aggradation (deposition) in the channel. To assess localize bed level changes through time near the prosed outfall site, three two-dimensional profiles, drawn along stream direction, were extracted through the nine surveys (**Figure 6** and **Figure 7**). Profile 1 is approximately 190 m, Profile 2 is 110 m and Profile 3 is 40 m from the outfall location. Overall, variability in bed levels decreases closer to the shore. The bed levels along Profile 1, near the center of the channel, show that bed elevation has varied up to 4 meters through time, with the highest bed levels in 1951 and the lowest in 2007 and 2010 **(Figure 7a)**. The lower levels in 2007 and 2010 likely are due to substantial changes to the channel associated with the installation of the Golden Ears Bridge and the dredging that was required. More recent surveys show that the bed has returned to levels consistent with pre-bridge installation. The pattern observed in Profile 2 is similar to that in Profile 1 **(Figure 7b).**

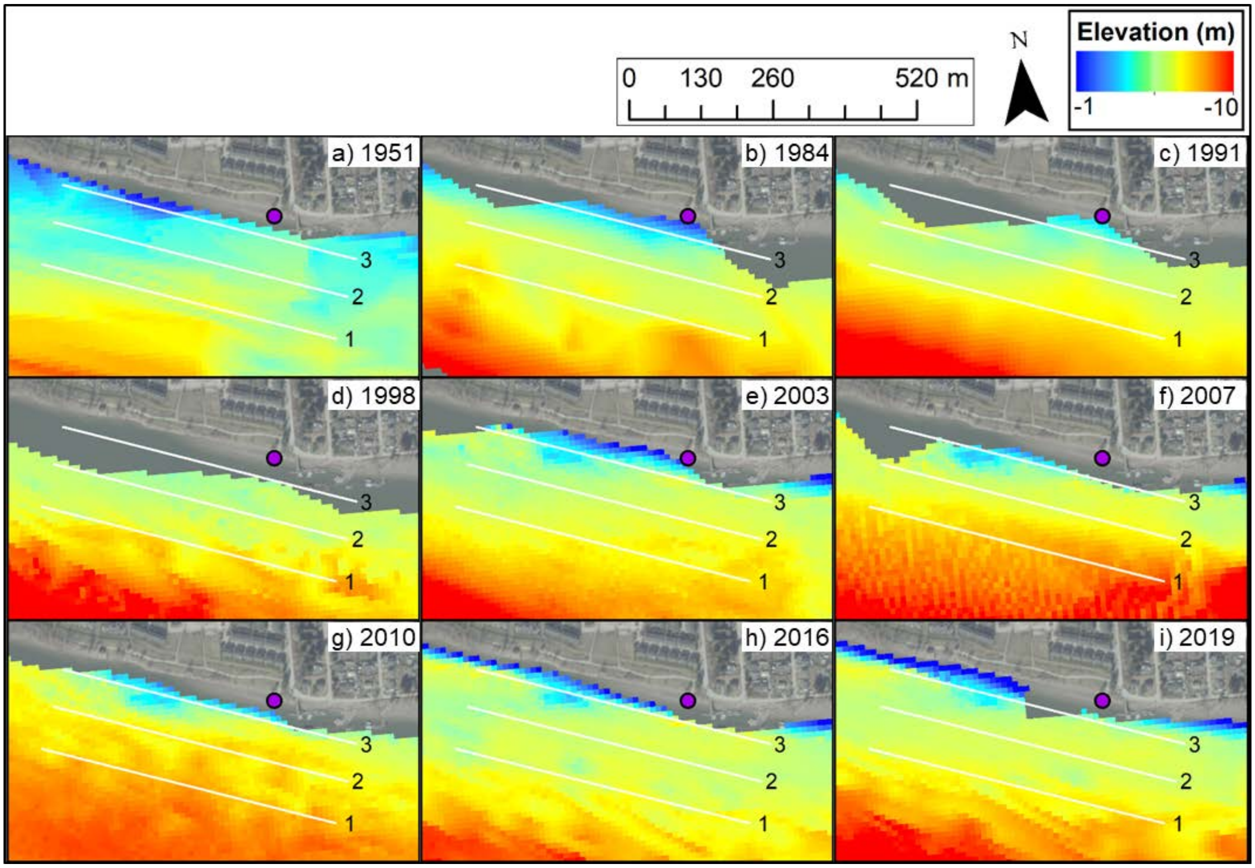

Figure 6. Bathymetry derived from PSPC surveys listed in Table 1. The purple circle indicates the proposed outfall location.

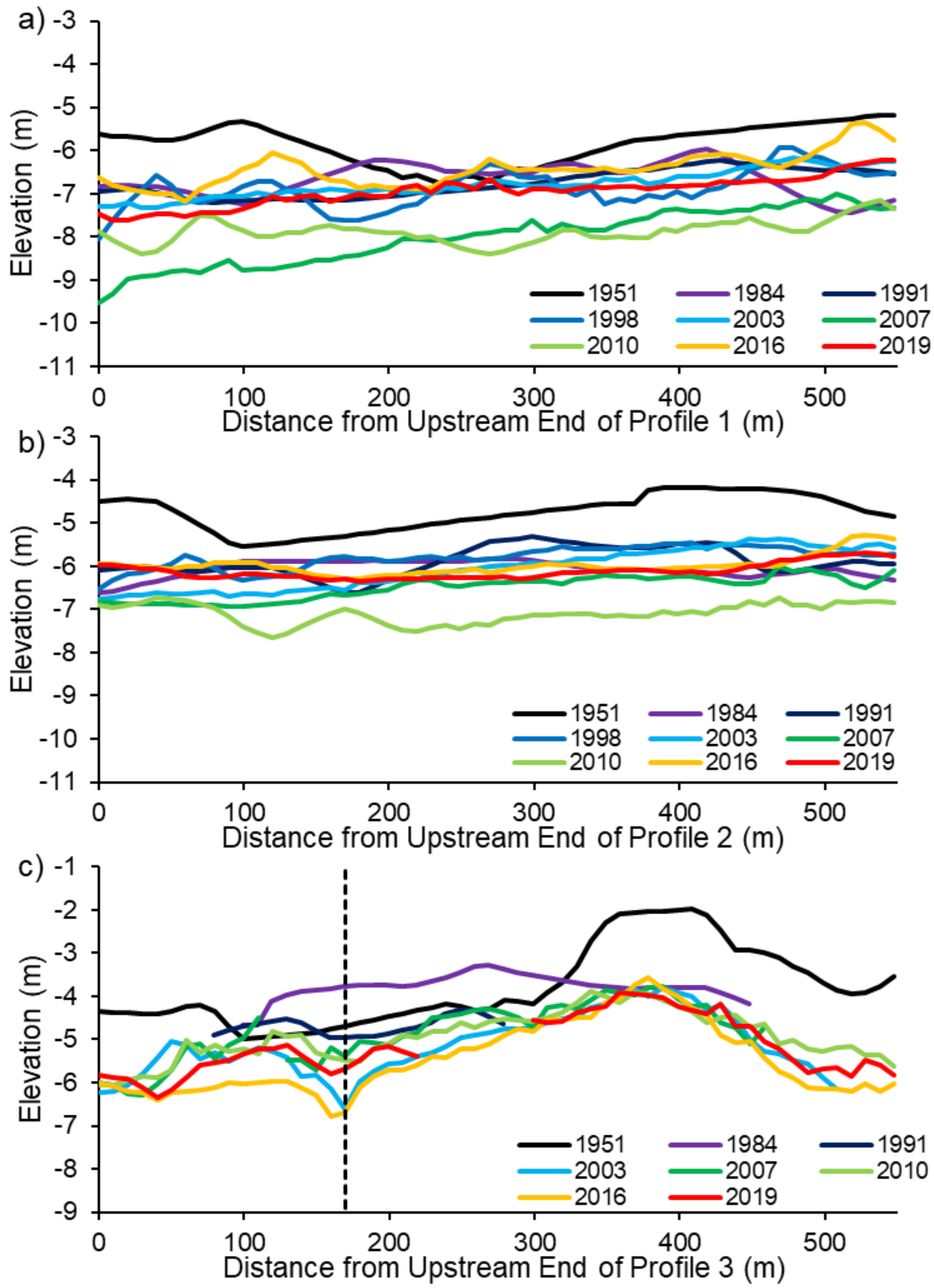

Figure 7. **Bed level profiles extracted from (a) Profile 1, (b) Profile 2, and (c) Profile 3 in Figure 6. The vertical dashed line indicates the approximate location of the proposed outfall location nearest to the profile.**

Profile 3, extracted at a location roughly 40 m of the bank from the proposed outfall location, shows the least amount of variability, although bed levels do fluctuate through time. The 1951 and 1984 profiles show the highest bed levels, consistent with widespread degradation observed in **Figure 5a**. There is an area in the 1951 profile of exceptionally high bed elevation downstream of the proposed site. However, this may reflect a measurement error or a temporary change as all other surveys at this location are lower and remarkably consistent, ranging within 0.5 m. Surveys since 1991 show that the bed levels nearest to the proposed outfall location range up to 2 m. There is no evidence of systematic scour at this location as the bed levels appear to randomly fluctuate from year to year within this range. The random fluctuations are likely linked to different flow conditions from year to year, but more detailed measurements would be needed to resolve the linkage.

The largest flows on record occurred in 1894 and 1948. Channel conditions around these floods are not captured in this analysis. However, a moderately high flow (approximately 1-in-15 year flood) occurred in 1997, only 6 months before the 1998 survey. The comparison does not indicate any noticeable degradation or lowering in the 1998 survey, to accommodate the 1997 flood. Moderate (about 1-in-15 year) floods also occurred in 2012 and 2018, and there is no evidence in the 2016 or 2019 surveys that these higher flows substantially affected the bed levels around the study site.

4.0 Hydraulic Assessment of Fraser River

NHC (2008) developed a one-dimensional (1D) hydraulic model of the Lower Fraser River from Mission to the mouth with the Salish Sea. The model was adapted for this study to calculate water surface profiles and cross-sectional average hydraulics at the project site. 1:100 year design flow is used for the initial outfall and erosion protection design. The initial design was checked against the hydraulics for the highest Fraser River flood of record, the 1894 flood (flow of 17,000 m^3/s observed at Hope) with a moderate climate change scenario with a 1 m sea-level rise (MFLNRORD and NHC, 2014). This flood is roughly equivalent to the 500-year flood. The sectional-average flow velocity and water surface elevation within the project site are provided in **Table 2** for the design floods.

Table 2. Water surface elevation and velocity for cross-sections within the project site simulated for 1:500 year design flow.

Location	Cross-section identifier	1: 100 Year Design Flow (14,300 m^3/s at Hope)		1894 Historical Flood with 1 m Sea Level Rise (17,000 m^3/s at Hope)	
		Water Surface Elevation (m)	Velocity (m/s)	Water Surface Elevation (m)	Velocity (m/s)
Downstream	FRASER' 51943	5.16	2.25	5.98	2.28
Project site	FRASER' 52349	5.18	2.32	6.01	2.39
Project site	FRASER' 52707	5.23	2.3	6.06	2.33
Upstream	FRASER' 53123	5.33	1.86	6.19	1.87

5.0 Hydrotechnical Design of Outfall

5.1 Headwall Design

NHC's scope of work included the design of a headwall for a concrete stormwater outfall pipe at the south end of the Bonson Road that discharges into the Fraser River. The stormwater outfall pipe was designed by HUB Engineering Inc. to convey the 10-year storm (1.11 m^3/s) and the 100-year storm (1.75 m^3/s) with a slope of 0.45%. NHC was provided with the pipe diameter (1.2 m) and alignment by HUB Engineering. It is estimated that at the outlet, during the design discharge the pipe will be around

60% full with an average flow velocity less than 2.5 m/s. Despite the pipe design not being part of the current design scope, it is recommended that the pipe at the outlet be concrete. This is to provide sufficient weight to resist buouancy forces when Fraser River water levels are above the pipe. Since the pipe crosses through a dike, geotechnical design must include a means to retain and prevent the migration of soils along the path of the pipe surface (MFLNRO, 2014).

Based on the specification received from a local supplier, NHC recommended a custom precast headwall as shown on the attached drawings. The headwall is to include sufficient clearance to install the floodgate and a minimum rise between the bottom of the gate and floor of the outfall of 0.2 m to limit the risk of blockage from debris or sediment (as shown on the attached drawing). It is expected that handrails are required to meet local government safety requirements.

5.2 Floodgate

A floodgate is to be installed to the headwall to prevent backflow through the culvert during high Fraser River water levels. The standard approach is to use a top hinged, cast iron flap gate. This approach is generally the most cost effective, however it is susceptible to debris and sediment obstructing the opening and closing of the gate. The likelihood of obstruction increases with lowering the position of the outfall. A low outfall elevation is desired to better accommodate upstream stormwater collection and conveyance. The proposed elevation is at 0.8 m, this is roughly 0.2 m below the existing grade of the surrounding bed at the proposed outfall location. Despite the expection of localized scour at the outfall (resulting from Fraser River flow and the discharge from the outfall), the site maybe susceptible to intermittent sedimentation. In order to reduce the susceptibility to obstruction, an elastomer duckbill style floodgate is proposed.

The duckbill valves have little headloss (see Figure 8). In comparison to flap gates, duck bill gates cost more (1.5 to 2x) but are corrosion resistant, less susceptible to damage and wear, expected to require less maintenance, easier to install, and less susceptible to obstruction.

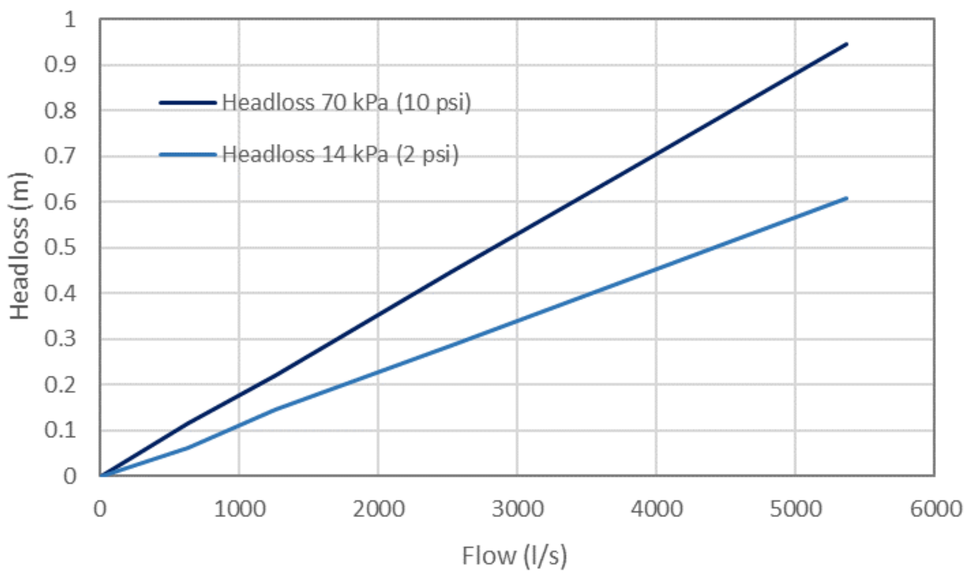

Figure 8. Approximate head loss from a duck bill gate (1.2 m diameter) (based on correspondence with Tideflex, 2020).

5.3 Armouring Design

Riprap armouring is recommend to increase the robustness of the bank and headwall in the resistance to erosion and scour. The size and requirements for graded riprap has been determined using the U.S. Army Corps of Engineers method (TAC, 2004; USACE, 1991). The bank slope has been set at 1.5H:1V to develop a stable bank while limiting the protrusion of the structure from the adjacent banks. Based on the design flow condition and the bank slope, a BC Ministry of Transportation (MoT) 250 kg Class riprap is recommended for the project site. This class of stone has a median grain size diameter (D_{50}) of roughly 570 mm. This is larger then the size of riprap placed along the foreshore near the existing stormwater outfall (roughly D_{50} of 400 mm, **Photos 4 & 5**), but is warranted particularly when considering the loss of stones at the existing outfall.

Riprap extents and specifications have been provided in the attached drawing. The new riprap layer should have a thickness of 1 m and is to be separated from the underlying material using a granual filter. The granular filter has been suggested instead of geotextile, as granular filters are more resilient to localized movement and are not susceptible to punctures, seams, or tears as fabric filters are.

The riprap is to protect the banks as well as extend out from the bank as an apron. The apron is to resist erosion and scour from outfall flow as well as Fraser River flows around the outfall. The bank and apron riprap as designed is to be sufficient to withstand erosion for flow velocities expected during the Fraser River flows up to the 500-year flood as well as from the outflow from the outfall. During extreme Fraser River flood flows, river levels will be overbank. In addition to this condition, design checks of riprap stability were conducted for high flows contained within the banks, more extreme flood flows expected with climate change (*moderate climate change projection* from FLNRO 2014), and wave events potentially occurring at this site from wind and vessel traffic (design wave height of 0.5 m).

Despite the design, the riprap armouring and outfall may experience damage or sedimentation over time. As presented in a previous section (Section 3, Geomorphic Assessment), the bed of the Fraser River cound adjust vertically by 2 m or more over time, or the thalweg could migrate towards the right bank. Such channel changes or localized damages that could occur from impacts from debris, deterioration of the riprap, concrete, or gate, could require maintenance to maintain the robustness of the proposed design.

The proposed riprap armouring and concrete outfall structure have been designed to provide protection against scour and erosion for the outfall, and in doing so, exceeds the existing resistance to such actions. However, this localized bank work is not part of, nor attempst to provide erosion protection for the dike set back on Bonson Road (MWLAP, 2003).

5.4 Timing and Cost

It is recommended that installation timing targets low Fraser River flow, low daytime tides, and the instream work window of least harm to fish. Typically Fraser River flow is greatest from May through August. Stormwater flows are typically greatest October to January. Low tides during daylight hours are most prevalent April through September, however the tides may not be as limiting as flow at this site. Due to the elevation of the pipe outlet, installation of the headwall may require isolation and dewatering of the area. The appropriate methodology for the isolation and dewatering, such as cofferdam or bulk bags, is the responsibility of the contractor.

A coarse estimate of cost and delivery time is presented in Table 3 for the headwall, floodgate, riprap, and miscellaneous materials (granular filters, granular base, handrails, and associated hardward). These costs are based on estimates from the supplier as well as from previous project, but may not be representative of current, local cost.

Table 3. Outfall components estimated cost and time for delivery.

Components	Estimated Cost	Estimated Delivery time
Headwall	$25,000	6 weeks
Duck bill gate	$60,000	12 weeks
Rock riprap	$20,000	1 week
Misc. materials	$10,000	3 weeks

6.0 Closure

We appreciate the opportunity to work on this project and hope this document meets your current needs. However, please do to not hesitate to contact the undersigned by email (hhamid@nhcweb.com | dmuir@nhcweb.com) or phone (6043980.6011) to discuss.

Sincerely,

Northwest Hydraulic Consultants Ltd.

Prepared by:

Feb 14/2022

Hanna Hamid PhD, EIT
Project Engineer

Reviewed by:

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA D.P. MUIR 27657

2022 FEB 10

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Project Manager

Prepared with support from: Ryan Bradly Ph.D., Geomorphology

ENCLOSURE

DISCLAIMER

This document has been prepared by **Northwest Hydraulic Consultants Ltd.** for the benefit of **EPTA DEVELPOMENT CORP.** for specific application to the **Katzi First Nation-Bonson Road Stormwater Outfall Design**. The information and data contained herein represent **Northwest Hydraulic Consultants Ltd.** best professional judgment in light of the knowledge and information available to **Northwest Hydraulic Consultants Ltd.** at the time of preparation, and was prepared in accordance with generally accepted engineering practices.

Except as required by law, this report and the information and data contained herein are to be treated as confidential and may be used and relied upon only by **EPTA DEVELPOMENT CORP.**, its officers and employees. **Northwest Hydraulic Consultants Ltd.** denies any liability whatsoever to other parties who may obtain access to this report for any injury, loss or damage suffered by such parties arising from their use of, or reliance upon, this report or any of its contents.

7.0 References

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ATTACHMENT 1:
REFERNCE STORMWATER DRAWING

POST-DEVELOPMENT - BONSON ROAD
STORM SEWER DESIGN - RATIONAL METHOD

Location: Eagle Meadows Business Park
Ref. No.: 20001
IDF Curve: Pitt Meadows (Works Yard)
Return Period: 100 Year

Q_{peak} = Design Flow (m^3/s)
A = Area (ha)
R = Runoff Coefficient
I = Rainfall Intensity (mm/hr)
N = 0.00278

Tc = Ti + Tt
Tc = Time of Concentration (min)
Ti = Inlet Time (min)
Tt = Travel Time (min)
$I = aT^b$ where I in mm/hr, T in hr
a = 29.143
b = -0.545

n = Roughness Coefficient
V_{cap} = Velocity at Capacity (m/s)
Q_{cap} = Flow at Capacity (m^3/s)

Date: 10-Jan-22
Calc. By: Hub Engineering Inc. - MC/KK
Sheet: 2 of 2

Location		Tributary Area					Runoff					Sewer Design						HGL Condition	HGL Slope
From MH	To MH	Area No.	A (ha)	R	RA	∑ (AR)	Ti (min)	Tt (min)	Tc (min)	I (mm/hr)	Q_{peak} (m^3/s)	Q_{cap} (m^3/s)	Diameter (mm)	n	Slope (%)	V_{cap} (m/s)	Length (m)		(%)
CITY OF PITT MEADOWS - BONSON ROAD																			
EX. D-1	EX. D-2	A	0.13	0.55	0.07	0.07	10.00	1.69	11.69	71.0	0.014	0.033	250	0.013	0.310	0.67	68.5	IN GROUND	0.056
EX. D-2	EX. D-3	B	0.74	0.55	0.41	0.48	11.69	2.16	13.86	64.7	0.086	0.032	250	0.013	0.290	0.65	84.7	SURFACE	2.097
EX. D-3	EX. D-4	C	0.57	0.55	0.31	0.79	13.86	1.00	14.86	62.3	0.137	0.132	375	0.013	0.570	1.20	72.1	SURFACE	0.612
EX. D-4	EX. D-5	D	0.11	0.55	0.06	0.85	14.86	0.43	15.29	61.3	0.145	0.213	600	0.013	0.120	0.75	19.6	SURFACE	0.056
CITY OF PITT MEADOWS - BONSON ROAD																			
MH D-8	EX. D-5	E	0.15	0.85	0.13	0.13	5.00	0.41	5.41	108.0	0.038	0.268	600	0.013	0.190	0.95	23.2	SURFACE	0.004
EX. D-5	EX. D-6				0.00	0.98	15.29	0.77	16.07	59.7	0.163	0.353	600	0.013	0.330	1.25	58.0	SURFACE	0.070
			∑ 1.70 ha																

Jan-11-2022 - 4:56 PM G:\Projects\20001.cta\B1 Design\Storm Analysis\[2022-01-07 STMCP Calculations.xlsx]100-YR Flow (Bonson Post)

PRE-DEVELOPMENT - BONSON ROAD
STORM SEWER DESIGN - RATIONAL METHOD

Location: Eagle Meadows Business Park
Ref. No.: 20001
IDF Curve: Pitt Meadows (Works Yard)
Return Period: 100 Year

Q_{peak} = Design Flow (m^3/s)
A = Area (ha)
R = Runoff Coefficient
I = Rainfall Intensity (mm/hr)
N = 0.00278

Tc = Ti + Tt
Tc = Time of Concentration (min)
Ti = Inlet Time (min)
Tt = Travel Time (min)
$I = aT^b$ where I in mm/hr, T in hr
a = 29.143
b = -0.545

n = Roughness Coefficient
V_{cap} = Velocity at Capacity (m/s)
Q_{cap} = Flow at Capacity (m^3/s)

Date: 10-Jan-22
Calc. By: Hub Engineering Inc. - MC/KK
Sheet: 1 of 2

Location		Tributary Area					Runoff					Sewer Design						HGL Condition	HGL Slope
From MH	To MH	Area No.	A (ha)	R	RA	∑ (AR)	Ti (min)	Tt (min)	Tc (min)	I (mm/hr)	Q_{peak} (m^3/s)	Q_{cap} (m^3/s)	Diameter (mm)	n	Slope (%)	V_{cap} (m/s)	Length (m)		(%)
CITY OF PITT MEADOWS - BONSON ROAD																			
EX. D-1	EX. D-2	A	0.13	0.55	0.07	0.07	10.00	1.69	11.69	71.0	0.014	0.033	250	0.013	0.310	0.67	68.5	IN GROUND	0.056
EX. D-2	EX. D-3	B	0.74	0.55	0.41	0.48	11.69	2.16	13.86	64.7	0.086	0.032	250	0.013	0.290	0.65	84.7	SURFACE	2.097
EX. D-3	EX. D-4	C	0.57	0.55	0.31	0.79	13.86	1.00	14.86	62.3	0.137	0.132	375	0.013	0.570	1.20	72.1	SURFACE	0.612
EX. D-4	EX. D-5	D	0.11	0.55	0.06	0.85	14.86	0.43	15.29	61.3	0.145	0.213	600	0.013	0.120	0.75	19.6	SURFACE	0.056
CITY OF PITT MEADOWS - BONSON ROAD																			
MH D-8	EX. D-5	2,3,4,5,6,E	30.76	0.40	12.30	12.30	45.00	0.41	45.41	33.9	1.160	0.268	600	0.013	0.190	0.95	23.2	SURFACE	3.570
EX. D-5	EX. D-6				0.00	13.16	45.41	0.77	46.18	33.6	1.229	0.353	600	0.013	0.330	1.25	58.0	SURFACE	4.007
			∑ 32.31 ha																

Jan-11-2022 - 4:57 PM G:\Projects\20001.cta\B1 Design\Storm Analysis\[2022-01-07 STMCP Calculations.xlsx]100-YR Flow (Bonson Pre)

POST-DEVELOPMENT
STORM SEWER DESIGN - INFORWORKS ICM 13.1.5

LOCATION: Katzie Reserve No.1
REF. No.: 20001
Rain Gauge: Katzie Pump Station
Return Period 10 Year and 100 Year 24hr Rainstorms

INFORWORKS ICM 13.1.5

Ø=Pipe Diameter (mm)
n=Roughness Coefficient
S=Slope of Pipe (%)
Vcap=Velocity at Capacity (m/s)
L=Length of Pipe (m)
Qcap=Flow at Capacity (m^3/s)

Date: January 27, 2022
Calc. By: MN
Sheet: 1 of 1

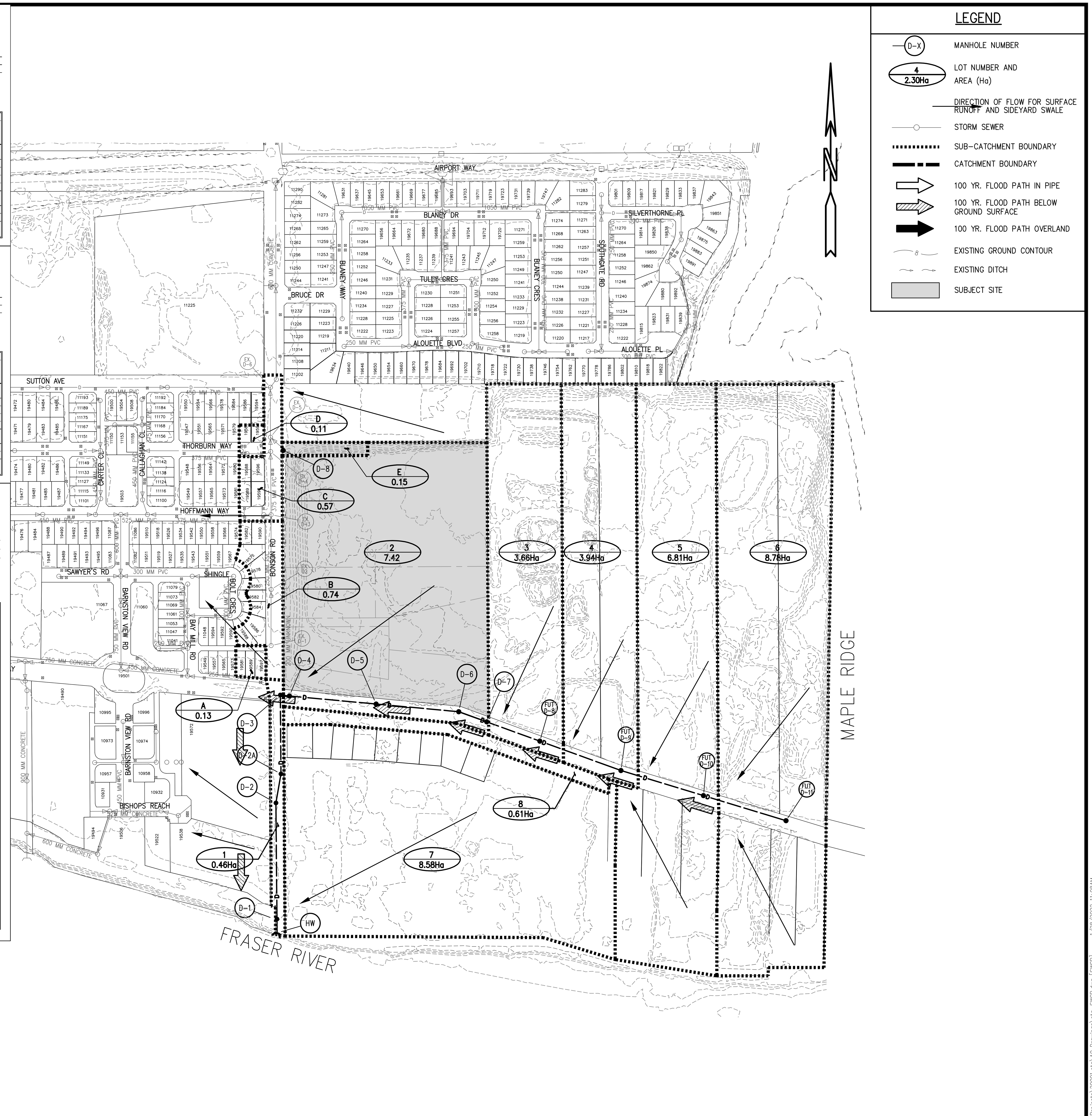
Location		Segment	Tributary Area			Max Flow in Pipe		Sewer Design						100 yr. HGL Condition at Upstream Node
From	To	Link Name	Area No.	A (ha)	Impervious (%)	Q - 10 Year (m^3/s)	Q - 100 Year (m^3/s)	Qcap (m^3/s)	Ø (mm)	n	S (%)	Vcap (m/s)	L (m)	
Fraser Way														
FUT D-11	FUT D-10	MH.D-11.1	6	8.78	90	0.130	0.284	0.905	900	0.013	0.250	1.42	100.0	IN GROUND
FUT D-10	FUT D-9	MH.D-10.1	5	6.81	90	0.222	0.511	0.905	900	0.013	0.250	1.42	100.0	IN GROUND
FUT D-9	FUT D-8	MH.D-9.1	4	3.94	90	0.281	0.596	1.221	1050	0.013	0.200	1.41	100.0	IN GROUND
FUT D-8	D-7	MH.D-8.1	3	3.66	90	0.343	0.715	1.489	1200	0.013	0.146	1.32	65.5	IN GROUND
D-7	D-6	MH.D-7.1	-	-	-	0.344	0.715	1.488	1200	0.013	0.146	1.32	34.3	IN GROUND
D-6	D-5	MH.D-6.1	-	-	-	0.345	0.715	1.486	1200	0.013	0.145	1.31	96.3	IN GROUND
D-5	D-4	MH.D-5.1	-	-	-	0.435	0.920	1.452	1200	0.013	0.139	1.28	100.9	IN GROUND
D-4	D-3	MH.D-4.1	2	7.42	90	0.435	0.920	2.054	1200	0.013	0.278	1.82	10.8	IN GROUND
Bonson Road														
D-3	D-2A	MH.D-3.1	1	0.46	90	0.483	0.952	2.149	1200	0.013	0.304	1.90	92.0	IN GROUND
			8	0.61	20									
D-2A	D-2		-	-	-	0.483	0.952	2.149	1200	0.013	0.304	1.90	34.0	IN GROUND
D-2	D-1	MH.D-2.1	-	-	-	0.483	0.952	2.123	1200	0.013	0.297	1.88	134.8	IN GROUND
D-1	H/W	MH.D-1.1	7	8.58	40	1.113	1.746	2.627	1200	0.013	0.455	2.32	17.6	IN GROUND

Note: Fraser River HGL elevations provided by Northwest Hydraulics Consultants (10yr = 3.7m) (100yr = 4.7m)

Note: All commercial lots are assumed to restrict 100yr and 10yr flows to pre-development conditions.

Imperv. Roughness= 0.013
Perv. Roughness= 0.250
Imperv. Storage Depth= 0.071 mm
Perv. Storage Depth= 0.280 mm
Initial Infiltration= 76.00 mm/hr
Limiting Infiltration= 2.50 mm/hr
Decay Factor= 2.00 hr^{-1}
Maximum Infiltration= 50 mm

Jan-27-2022 - 1:30 PM G:\Projects\20001.cta\B1 Design\Storm Analysis\ICM Model\[2022-01-27 SWCP for 20001.xlsx]Infow orks ICM INTERIM



LEGAL DESCRIPTION: ----

SURVEY BENCHMARK
MON: 88H0617 LOC: -
SCALE FACTOR:
ELEV.: 6.525m (GEODETIC)

REV.	DATE	DESCRIPTION	BY
4	NOV 17/21	ISSUED FOR MUNICIPAL REVIEW	KK
3	OCT 07/21	ISSUED FOR REVIEW	KK
2	JUL 25/21	ISSUED FOR MUNICIPAL REVIEW	MC
1	JUN 17/21	ISSUED FOR MUNICIPAL CONCEPTUAL REVIEW	MC

NOT FOR CONSTRUCTION

FOR COORDINATION ONLY

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BUSINESS PARK

CLIENT
EM BUSINESS PARK LTD.
1910 – 1177 WEST HASTINGS STREET
VANCOUVER, B.C., V6E 2K3, TEL: (604) 270-1890

TITLE
STORM WATER CATCHMENT PLAN

SEAL
Jan 28, 2022

SCALE: HOR. 1:2500
VERT.
DESIGNED MC/MN/KK
DRAWN AKG
REVIEWED KL/RFG

DATE (YYYY.MM.DD) FEB 2020
CONSULTANT PROJ. NO. 20001
DWG. NO. 16
REV. 4

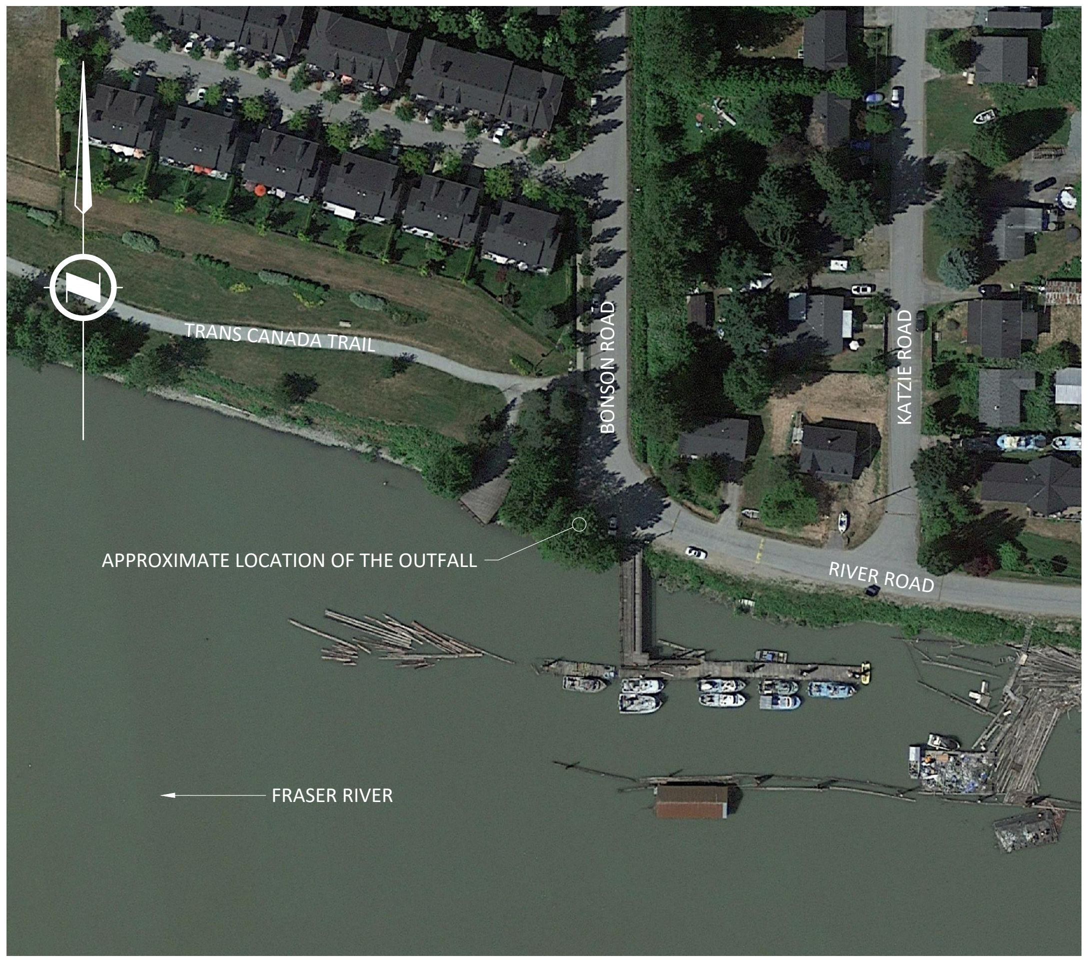
MUNICIPAL PROJECT NUMBER -
DRAWING TYPE
DRAINAGE

DESTROY ALL PRINTS BEARING PREVIOUS NUMBER

ATTACHMENT 2:
OUTFALL DRAWING

EPTA DEVELOPMENT CORPORATION

BONSON ROAD OUTFALL



SITE PLAN
SCALE = NOT TO SCALE

DRAWING INDEX	
TITLE	REVISION
SITE PLAN, DRAWING INDEX, AND NOTES	0
PLAN, PROFILE, AND SECTIONS	0

1. GENERAL NOTES

   1.1. ALL WORK IS TO COMPLY WITH CURRENT AUTHORIZATIONS AND PERMITS.
   1.2. ALL ELEVATIONS, DIMENSIONS, AND QUANTITIES SHALL BE VERIFIED BEFORE CONSTRUCTION COMMENCEMENT.
   1.3. DIMENSIONS ARE IN METRES UNLESS OTHERWISE STATED.
   1.4. ALL ELEVATIONS ARE RELATIVE TO CANADIAN GEODETIC VERTICAL DATUM OF 1928 (CGVD28).
   1.5. TABULATED ESTIMATES OF MASS AND VOLUMES ARE CLEAN LINE ESTIMATES AND ALLOWANCES FOR SETTLEMENT AND/LOSS ARE NOT INCLUDED.
   1.6. SLOPES SHALL BE GRADED TO PROVIDE A SMOOTH, UNIFORM SURFACE. ALL STUMPS, LARGE ROCK, BRUSH, OR OTHER DEBRIS SHALL BE REMOVED. ALL DEPRESSIONS SHALL BE FILLED, AND LOOSE OR UNSTABLE SOILS SHALL BE REPLACED.
   1.7. ESTIMATED MATERIAL QUANTITIES ARE BASED ON CLEAN LINE ESTIMATES. CONTRACTOR RESPONSIBLE TO CONFIRM QUANTITIES REQUIRED.

MATERIAL QUANTITIES			
ITEM	DESCRIPTION	UNIT	APPROXIMATE QUANTITY
1	CONCRETE OUTFALL	LS	1
2	FLOOD GATE	LS	1
3	HANDRAIL	LS	1
4	250-kg RIPRAP	m^3	130
5	GRANULAR FILTER	m^3	45
6	CRUSHED GRAVEL	m^3	1
7	DRAIN ROCK	m^3	3

2. MATERIALS

   2.1. RIPRAP
      2.1.1. 250-kg RIPRAP IS TO BE USED FOR THE BANK ARMOURING.
      2.1.2. ROCK SHALL BE PREDOMINATELY ANGULAR AND BLOCKY.
      2.1.3. ROCK IS TO BE ROUGHLY EQUI-DIMENSIONAL; LENGTH NOT MORE THAN 2.4 TIMES THE WIDTH OR THICKNESS AS MEASURED AT THE MIDDLE OF THE STONE.
      2.1.4. PLACEMENT OF RIPRAP SHALL BE CARRIED OUT BY EXCAVATOR. END DUMPING USING CHUTES OR SIMILAR METHODS WILL NOT BE PERMITTED.
      2.1.5. RIPRAP GRADATION TO BE CONFIRMED PRIOR TO DELIVERY AND ON SITE BY THE SITE ENGINEER.

   2.2. GRANULAR FILTER
      2.2.1. GRANULAR FILTER IS TO BE USED TO PROVIDE A FILTER LAYER BETWEEN THE RIPRAP AND THE UNDERLYING ROCK BANK/CHANNEL MATERIALS.
      2.2.2. THIS MATERIAL IS TO BE WELL GRADED GRAVEL, SAND, AND COBBLE.
      2.2.3. PLACEMENT OF THE FILTER ROCK SHALL BE INSPECTED BY THE SITE ENGINEER PRIOR TO BE PLACEMENT OF OVERLAYING MATERIAL, AND IMMEDIATE FOLLOWED BY PLACEMENT OF THE OVERLAYING MATERIALS.

   2.3. CRUSHED GRAVEL
      2.3.1. CRUSHED GRAVEL TO BE USED FOR HEADWALL FOUNDATION.
      2.3.2. CRUSHED GRAVEL TO BE 19 mm CLEAR CRUSH AS PER MMCD STANDARD SPECIFICATION.
      2.3.3. UNDERLYING SOIL TO BE VERIFIED BY GEOTECH ENGINEER BEFORE PLACEMENT OF GRAVEL.
      2.3.4. GRAVEL PLACEMENT TO BE CONFIRMED BY GEOTECH ENGINEER PRIOR TO INSTALLATION OF HEADWALL.

   2.4. DRAIN ROCK
      2.4.1. DRAIN ROCK TO BE USED AT BACK SIDE (LANDSIDE) OF HEADWALL
      2.4.2. DRAIN ROCK TO BE 25 mm DRAIN ROCK AS PER MMCD STANDARD SPECIFICATION.

   2.5. ROCK GRADATION

ROCK GRADATION				
PERCENT PASSING (STONE SIZE IN MILLIMETRES FOR PERCENT PASSING)				
MATERIAL	15%	50%	85%	100%
250-kg RIPRAP	≥260	≥570	≥820	≤1000
GRANULAR FILTER	7.5-10	30-65	55-100	≤150

   2.6. OUTFALL
      2.6.1. OUTFALL TO BE CUSTOM PRECAST REINFORCED CONCRETE, ENGINEERED AND SUPPLIED BY LANGLEY CONCRETE (I.E. 2.7 X 2.5 m HEADWALL MAX SERIES) OR APPROVED EQUIVALENT.
      2.6.2. OUTFALL TO BE APPROVED BY PROJECT ENGINEER PRIOR TO ORDERING.
      2.6.3. CONTRACTOR TO VERIFY SELECTED HEADWALL, FLOODGATE, AND PIPE ARE COMPATIBLE (I.E. SUFFICIENT OPENING SIZE AND SPACING).
      2.6.4. HANDLING, STORAGE, AND INSTALLATION TO FOLLOW SUPPLIER/MANUFACTURER'S RECOMMENDATIONS.

   2.7. FLOODGATE
      2.7.1. OUTFALL FLOODGATE TO BE NEOPRENE DUCK-BILL CHECK VALVE WITH STAINLESS STEEL FASTENERS, SUCH AS TIDEFLEX TF-1 OR APPROVED EQUIVALENT.
      2.7.2. FLOOD GATE TO BE APPROVED BY PROJECT ENGINEER PRIOR TO ORDERING
      2.7.3. CONTRACTOR TO VERIFY SELECTED HEADWALL, FLOODGATE, AND PIPE ARE COMPATIBLE (I.E. SUFFICIENT OPENING SIZE , SPACING, AND GATE FITS ON O.D. OF PIPE).
      2.7.4. HANDLING, STORAGE, AND INSTALLATION TO FOLLOW SUPPLIER/MANUFACTURER'S RECOMMENDATIONS.

NOT FOR CONSTRUCTION

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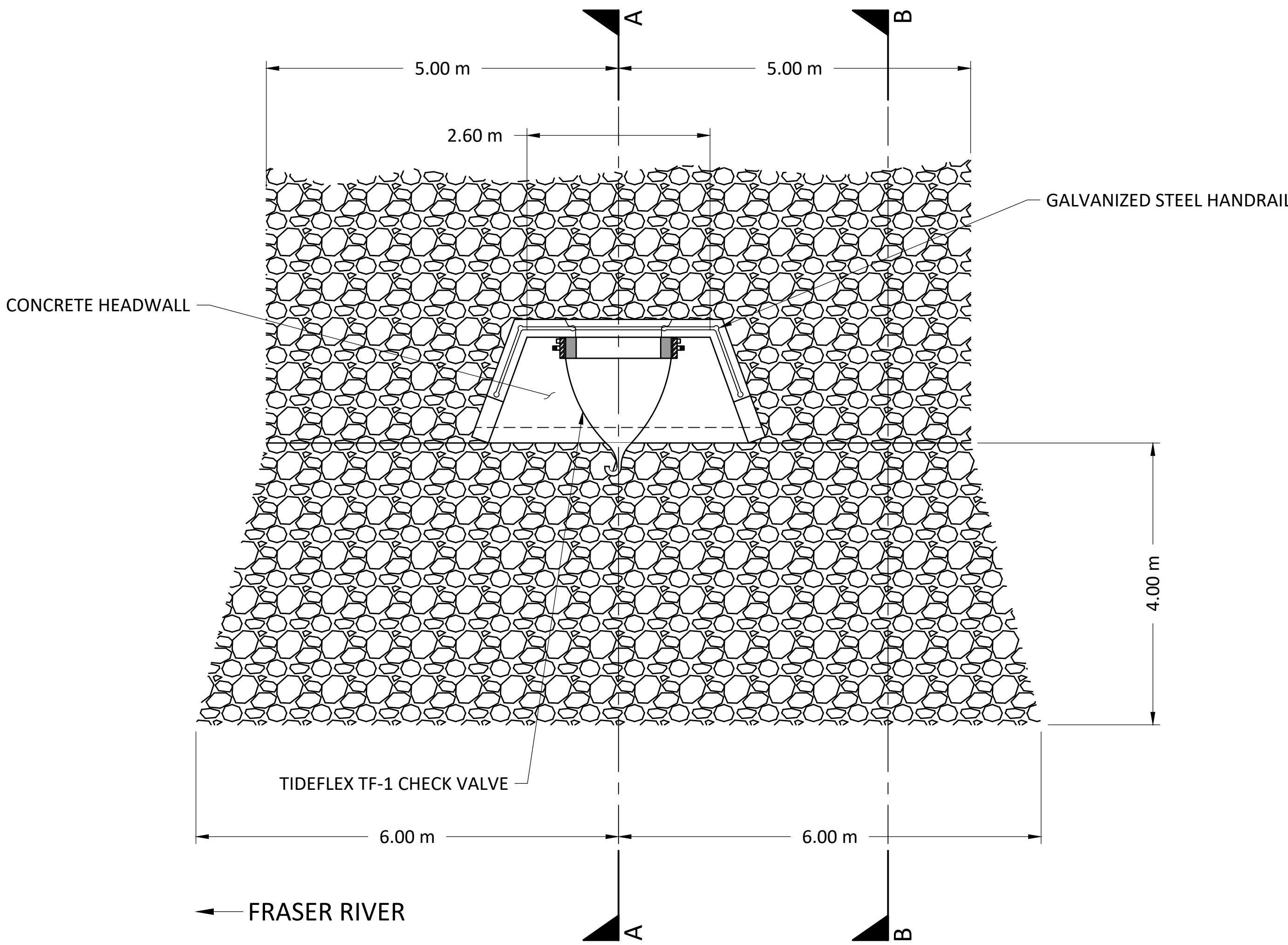
REVISIONS		
0	14 Sep 2021	ISSUED FOR REVIEW

DRAWING INFORMATION	
DATE	10 Feb 2022
DESIGNED BY	HXH
DRAWN BY	BXH
CHECKED BY	DPM
SHEET SIZE	B (11" x 17")

BONSON ROAD OUTFALL

Site Plan, Drawing Index, and Notes

PROJECT NUMBER	3006489
DRAWING NUMBER	3006489-1
SHEET NUMBER	1 OF 2
REVISION	0



PLAN
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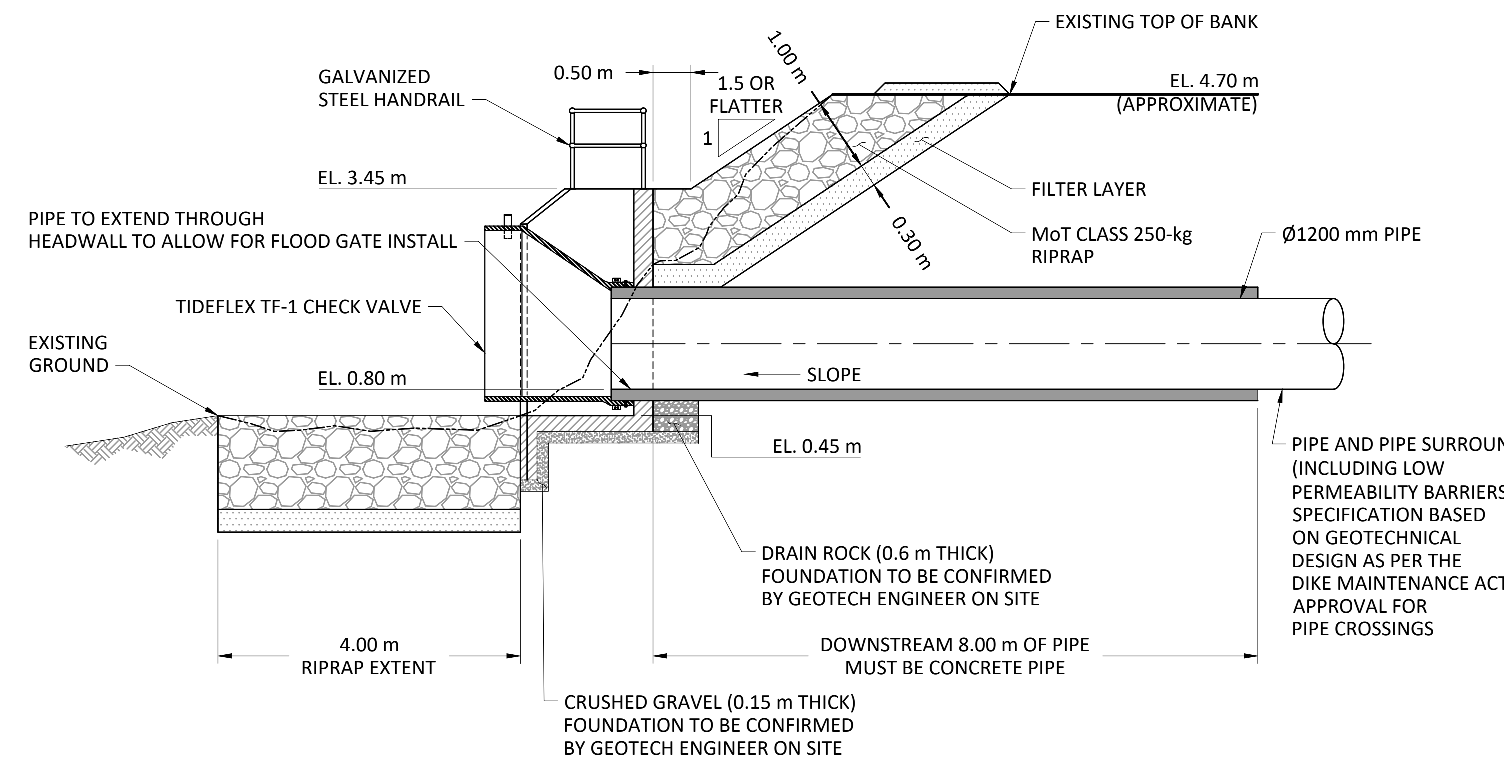

SECTION A
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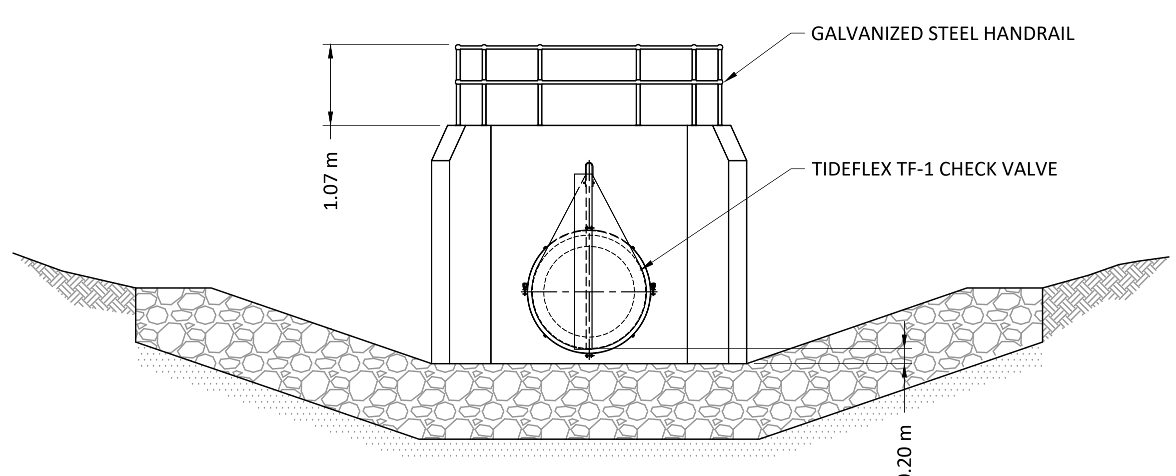

ELEVATION
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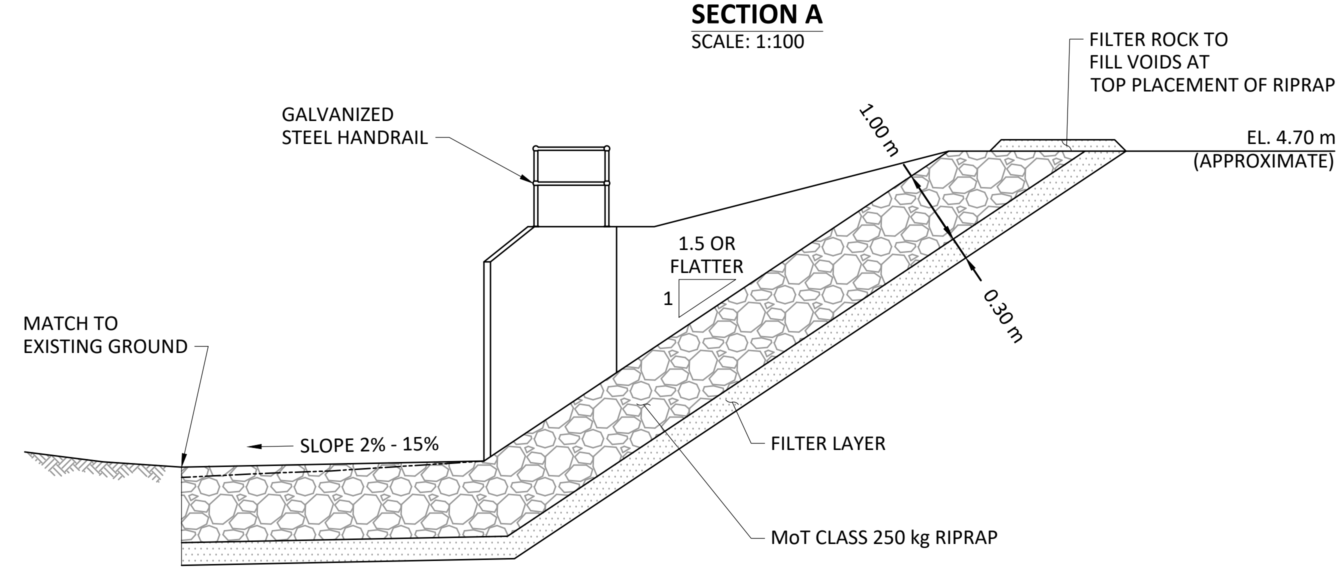

SECTION B - ALONG BANK ARMOUR
SCALE: 1:100

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REVISIONS		
0	10 Feb 2022	ISSUED FOR REVIEW

DRAWING INFORMATION	
DATE	10 Feb 2022
DESIGNED BY	HXH
DRAWN BY	BXH
CHECKED BY	DPM
SHEET SIZE	B (11" x 17")

BONSON ROAD OUTFALL

Plan, Profile, and Sections

PROJECT NUMBER	3006489
DRAWING NUMBER	3006489-2
SHEET NUMBER	2 OF 2
REVISION	0