Stantec

Memo

To:	Mark DiDonato, Highway Design Engineer Ministry of Transportation and Infrastructure Kamloops, BC	From:	Shannon Quarterman P.Eng., PMP Grant Irvine, P.Eng. Stantec Kamloops, BC
Project/File:	Sackum Overhead No. 1491 Replacement	Date:	April 1, 2025

1 INTRODUCTION

Stantec Inc. (Stantec) has been retained by the Ministry of Transportation and Infrastructure (MOTI) to complete the structural and highway engineering services for the design of a new replacement structure for the existing Sackum Overhead No. 1491 which is located approximately 22.4 km north of Lytton. The existing overhead is a road-rail grade separated structure that carries Highway 1 traffic over the Canadian Pacific Kansas City (CPKC) rail lines.

This memo provides recommendations for the highway drainage for Sackum Overhead No. 1491 Replacement. This memo does not include the hydrological analysis of Sackum Creek.

2 DRAINAGE DESIGN

2.1 Design Return Period

The design return period for the storm water inlets was selected based on Table 1010.A in the BC Supplement to TAC, which states that storm water inlets shall have a design return period of 5 years.

The design return period for the ditches adjacent to Highway 1 was selected based on Table 1010.A in the BC Supplement to TAC, which states that highway ditches shall have a design return period of 25 years.

The design return period for the cross culvert on Highway 1 was selected based on Table 1010.A in the BC Supplement to TAC, which states that culverts less than a 3m span for ditch drainage network shall have a design return period of 100 years for an arterial road.

2.2 Runoff Coefficient

The dimensionless runoff coefficient, C, for the watersheds was estimated at 0.300, based on RTAC Table 2.4.1, for medium textured loams, well drained.

2.3 Time of Concentration

The time of concentration for each sub catchment, Tc, was calculated using the Hathaway Formula, which is applicable to small interior basins with light forest. The formula, as described in Section 1020.04 of the MOTI Supplement, is as follows:

$$T_c = \frac{(rL)^{0.467}}{1.65S^{0.385}}$$

Where: Tc = total time of concentration, in hours

L = longest flow path from the most remote part of the basin, in km;

S = average slope of the total stream length, in m/m; and,

r = roughness coefficient, 0.4, pasture, range land (BC Supplement to TAC p 1020-5

2.4 Rainfall Intensity

Rainfall IDF data was obtained from the IDF_CC Tool 7.0, on November 17, 2023. IDF_CC Tool 7.0 is a publicly available web-based intensity-duration-frequency tool which adapts local rainfall statistics to climate change. Ungauged IDFs located at Lat 50.31120° and Long -121.39583° were used for this assessment. The ungauged IDFs were compared to the gauged IDFs for Lytton ID:1114741, and there were no substantial differences between the gauged and ungauged locations.

Historical rainfall IDF values for both the gauged, and the ungauged location are shown in Appendix A.

IDF_CC Tool 7.0 was used to estimate the effect that climate change has on rainfall intensities at the ungauged location to year 2100. Using the PCIC-Bias Corrected and assuming a Representative Concentration Pathway (RCP) 8.5 climate change scenario, the climate change corrected IDF values are shown in Appendix A.

2.5 Design Flow Calculations

As described in the BC Supplement to TAC, for rural watersheds up to 10km^2, the Rational Method may be suitable to calculate design flow and is described in the following relationship:

$$Q_p = \frac{CiA}{360}$$

Where:

QP = peak flow rate in the specified watershed, m3/s

C = Runoff coefficient dependent on soil type and land use of area

i = Rainfall intensity for the design period, mm/hr

A = Area of watershed in question, ha

The average flow velocity was calculated as follows:

$$v = \frac{R^{0.67}S^{0.5}}{n}$$

Where:

v = the average flow velocity, m/s

R = the hydraulic radius = A/P, m

A = the cross-sectional area of flow, m2

P = wetted perimeter, m

S = the friction or channel slope, m/m

n = the Manning's roughness coefficient

3 HIGHWAY DITCHES

3.1 Ditch Capacity Requirements

The approximate catchment areas for the ditch on the south approach of Sackum Overhead are captured in Figure 1. Area 1 is the area nearest the top of the page, and Area 4 is the area nearest the bottom of the page.

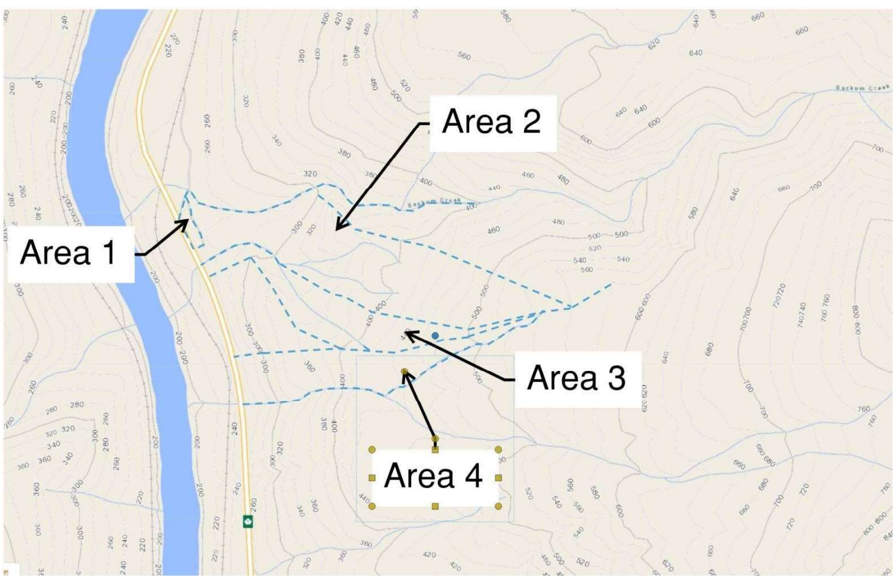

Figure 1 - Catchment Area South of Proposed Sackum OH

The catchment areas were delineated in such a way to determine the approximate conveyance requirements of each area due to various constraints such as the rock buttress at the bin wall and areas where prevention of upslope cuts results in a shallow ditch. Upslope cuts are required to be prevented to avoid impacting archaeologically sensitive areas near the existing cemetery. At the high point on the alignment, Sta. 107+44.807, the ditch requirements will be less than the ditch requirements at the LOC at Sta. 105+00.

The flows were determined using the methods described above; calculations can be found in Appendix B. A summary of the expected flows, and cumulative flows, can be found in Table 1.

Table 2 – Expected flows by catchment area, along with cumulative flows

Approx Station Range	Expected Flow (m^3/s)	Expected Cumulative Flow (m^3/s)
Area 1	0.01	0.01
Area 2	0.24	0.25
Area 3	0.08	0.35
Area 4	0.003	0.35

3.2 Ditch Sizing Calculations and Design

The depth requirement of the ditches was calculated iteratively using the Continuity Equation:

$$Q = vA$$

Where:

Q = the discharge, m^3/s

v = the average flow velocity, m/s

A = the cross-sectional area of flow, m2

Table 3.2.3 Section II-B2 in the RTAC Drainage Manual, Volume I, Manning's roughness coefficient for a lined open channel with grass, some weeds, is given with a range of 0.030 to 0.035. For this drainage exercise, the value of 0.035 was used.

Based on guidance in MOTI's BC Supplement to TAC Section 1030.02 (Channel Depth), the roadside drainage ditch depth should be designed such the ditch invert is a minimum 0.3m below the bottom of the SGSB layer. The depth of drainage channels should be such that the design flow is contained at the roadway, with an allowance for freeboard. The recommended minimum freeboard is 0.3m below the top of roadway for small drainage channels; larger drainage channels should have a greater freeboard allowance. The maximum allowable depth of flow in minor ditches is 0.6m. The bottom width of a roadside ditch should be at least 1m and should accommodate the design flow that is expected at the highway.

There are constraints with respect to the ditch on the east side of the northbound approach, which limit the available space for roadside ditches. Upslope cuts are required to be avoided as much as possible, to avoid archaeologically sensitive areas. Adjacent to the existing bin wall, a rock buttress will be used, which will limit the possibility of a roadside ditch. Table 2, below, summarizes the catchment area constraints, proposed ditch type based on the constraints, and available ditch depth below subgrade. The below proposed ditch types and approximate depths below subgrade, are what is available, based on project constraints.

Table 2 – Catchment Area, Constraints, and proposed ditch type

Approx Station Range	Catchment Area	Constraint	Proposed Ditch Type	Approx Depth below Subgrade
107+00 – 107+40	Area 1	Avoid upslope cut	V-ditch	0- 0.2
106+60 – 107+00	Area 1	Avoid upslope cut	V-ditch	0.2 - 0.5
106+10 – 106+60	Area 2 / Area 1	Existing bin wall; rock buttress to bin wall	No ditch – 300 mm conveyance pipe	N/A
106+00 – 106+10	Area 3	N/A - fill	N/A - fill	N/A - fill
105+60 – 106+00	Area 4 / Area 3	Avoid upslope cut	V-ditch	Varies; 0.1 - 0.5
105+00 – 105+60	Area 4	None	Flat bottom, existing conditions,	Varies; 0.3-0.4

In the area between approximately Sta 106+10 through Sta 106+60, adjacent to the existing bin wall, a ditch is not possible. In this station range, a conveyance pipe is proposed, which is sized to convey the flow that is expected from Area 1, as shown in Figure 1. The sizing calculations for the conveyance pipe can be found in Appendix B. The 300 mm conveyance pipe shall be a CSP pipe and shall be routed to a variable depth catch basin, and then discharged into the northbound ditch at approximately Sta. 105+93.000.

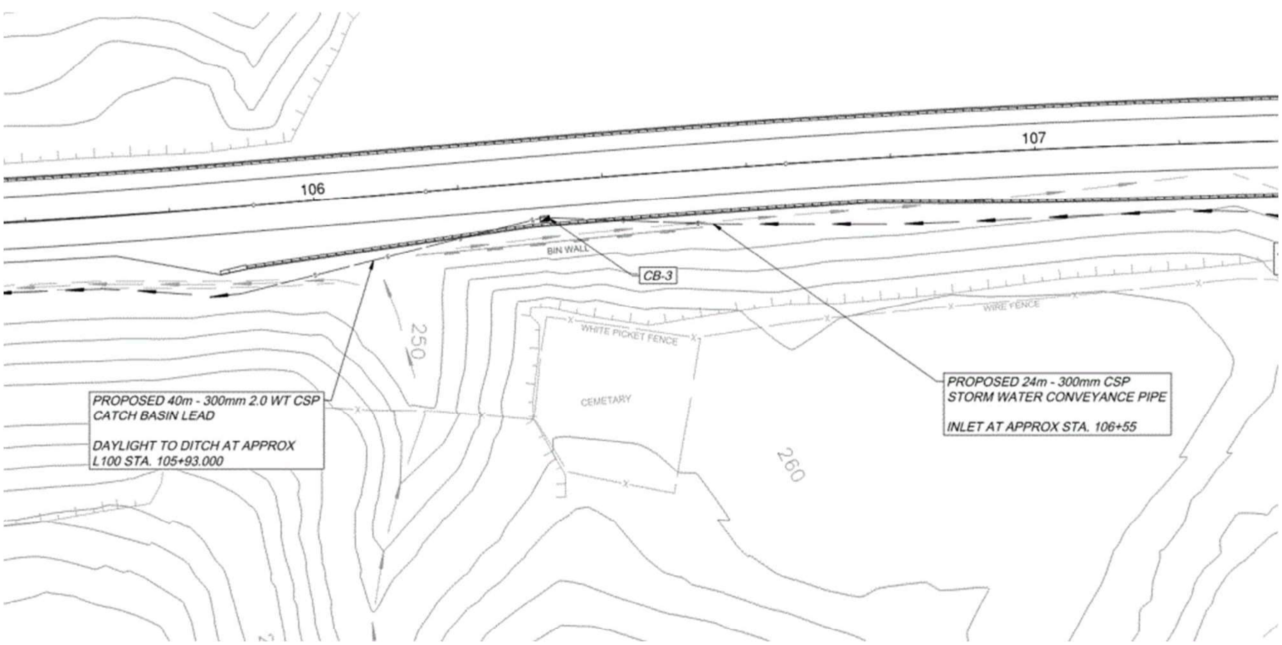

Figure 2 -300 mm Conveyance Pipe to convey storm drainage adjacent to existing bin wall

In some instances, during a 25-year return period event, the ditches could experience storm flow at a height that exceeds the elevation of the subgrade. In the ditch areas that to not meet the guidance set out in the BC

Supplement to TAC and could potentially experience storm flow at a height that exceeds the elevation of the top of the subgrade, it will be prudent to ensure that the highway structure, sub-grade and grade are protected from erosion of flowing water. The expected velocities in the ditch are low enough that 10 kg riprap will be sufficient to protect highway structures from erosion. Expected calculated velocities are shown in Appendix C. A summary of storm flow height in the ditches and riprap requirements can be found in Table 3, and supporting calculations in Appendix C.

Table 3 – Potential water height above top of subgrade, flow velocity, erosion protection requirements

Approx Station Range	**Catchment Area**	**Potential water height above top of subgrade (m)**	**Flow Velocity (m/s)**	**Riprap Class (kg)**
107+00 – 107+40	Area 1	0	0.5	Not required
106+60 – 107+00	Area 1	Approx 0.07	1.1	Not required for flow velocity, but 10 kg riprap on highway foreslope to be used to protect road structure
105+60 – 106+00	Area 4 / Area 3	Approx 0.3	1.2	Not required for flow velocity, but 10 kg riprap on highway foreslope to be used to protect road structure
105+00 – 105+60	Area 4	Approx 0.1	1.2	Not required for flow velocity, but 10 kg riprap on highway foreslope to be used to protect road structure

4 HIGHWAY CROSS CULVERT

A cross culvert is required to convey storm water runoff between the existing highway embankment, north of the existing overhead structure, and the new alignment, to the east side of the new alignment. This catchment area is very small, see Figure 2.

The Q_{100} 100-year peak event flow is 0.053 m^3/s, using the Rational Method, when taking climate change into account, and 0.043 m^3/s for a historical flow. The gauged vs. ungauged locations were compared, and the ungauged location had a slightly higher flowrate.

The lowest capacity, Q, on Figure 1040.B in the BC Supplement to TAC is 0.1 m^3/s, which corresponds to a minimum culvert size of 400mm. Based on recommendations in the BC Supplement to TAC, for a culvert under a highway is 600 mm, the recommended culvert size for the cross-culvert at this location is 600mm.

A further analysis of this culvert was completed, using a more conservative value for the surface cover, to determine if this culvert has enough capacity, should the area between the proposed highway, and the western side of the existing highway, between approx. Sta. 108+90 to Sta. 111+40, be developed by the Nicomen Indian Band. A surface cover value of 0.02, from page 1020-05 in the BC Supplement to TAC was used, which is a smooth, impervious cover. This assumes that virtually none of the water that falls within this area is absorbed and will all eventually be directed to the 600mm cross culvert. Using a value of 0.02 for roughness coefficient, a Q100 100-year peak event flow is 0.096m3/s, using the Rational Method, and taking climate change into account. This is still within the capacity for the recommended 600mm culvert.

It was reviewed to have a cross culvert along the existing highway embankment, at approximately Sta. 110+80 (stationing from proposed alignment), however there are some challenges with this approach. The embankment on the west side of the existing highway is steep, and erosion protection in the form of riprap would be extremely difficult to construct.

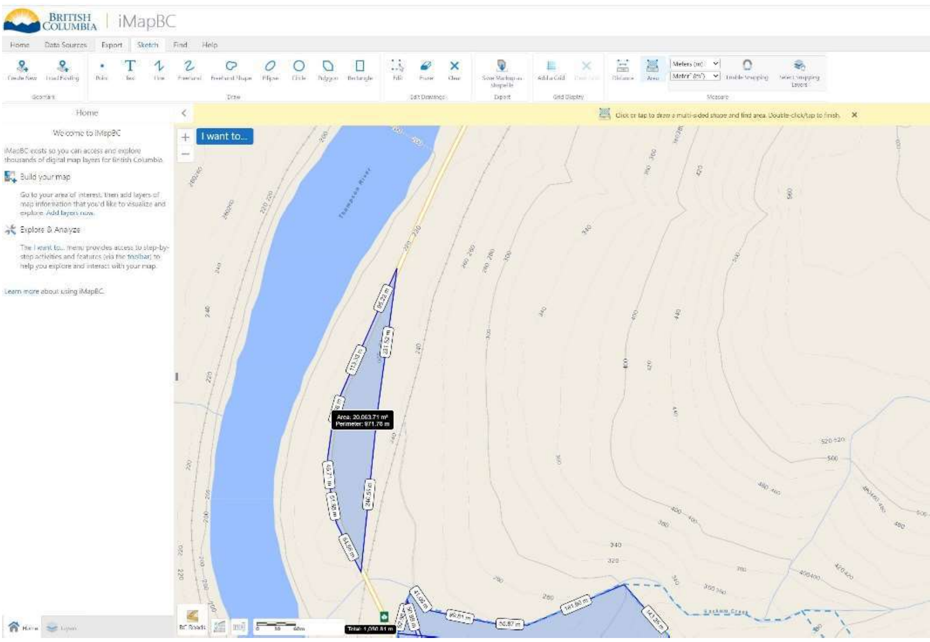

Figure 2 - Catchment Area North of Existing Sackum OH

The culvert discharge velocity was verified in Hy-8, using the culvert lengths and inverts, using a Q_{100} flow. Using the discharge velocity of 3.9m/s, a riprap size of 100kg was calculated, using Figure 1030.A in the BC Supplement to TAC. Due to a lack of availability of 100kg riprap onsite, the riprap will be upsized from 100kg to 250kg riprap.

Culvert flow calculations, Hy-8 analysis and riprap sizing details can be found in Appendix D.

5 PAVEMENT DRAINAGE

Pavement drainage was designed according to the BC MOTI Supplement to TAC Geometric Design Guide. Pavement runoff was determined using data from the IDF_CC Tool, for a return period of 5 years, and a time of concentration equal to 5 minutes, which corresponds to a rainfall intensity of 12.74 mm/h. Catch basin spacings were calculated using the Spreadsheet/Calculator Method, a Manning Roughness Coefficient of 0.02 for rough texture asphalt pavement (RTAC Drainage Manual Table 3.2.3, VI B2) and a ponding width of 65% of paved shoulder width. Catch basin placement was determined after completing pavement drainage calculations, and reviewing grades, superelevation, effective width of contributing area, high points, locations of structures.

Where catch basin spacing calculations exceeded the maximum spacing of 150m, the recommended maximum spacing guideline of 150m was utilized.

Where the gutter velocity is less than 1.5 m/s, a single grate inlet will be utilized. Where the gutter velocity is greater than 1.5 m/s, a twin grate inlet will be utilized.

Locations of catch basins are shown in Table 1. All catch basins are to be SP582-02.01 – Precast Reinforced Concrete Catch Basins. CB-4, and CB-5 shall have Twin Inlet Catch Basin Frame SP 582-05.04.

Table 2: Catch basin calculated values

Catch Basin	**Station**	**Design Ponding Width, (m)**	**Width, effective inlet catchment, (m)**	**Maximum Flow Velocity, Gutter Flow, (m/s)**	**Design Flow Rate Gutter Flow, (m^3/s)**	**Intercepted Flow, (m^3/s)**	**Catch Basin Efficiency (%)**
CB-1	105+00 LT	1.3	0.69	0.82	0.012	0.010	86.6%
CB-2	105+00 RT	This CB will have a manhole lid; it is being used to direct water from CB-1 to the NB ditch					
CB-3	106+37 LT	1.3	0.69	0.82	0.012	0.010	86.6%
CB-4	107+74 RT	1.3	0.69	0.48	0.007	0.007	86.6%
CB-5	109+13RT	1.3	0.88	0.28	0.005	0.005	94.9%
CB-6	110+70 RT	1.3	0.69	2.7	0.091	0.077	83.9%
CB-7	112+50 RT	1.3	0.69	2.7	0.075	0.063	83.9%
CB-8	113+00 RT	1.3	0.69	1.4	0.018	0.016	86.6%
CB-9	113+00 LT	1.3	0.69	1.4	0.018	0.016	86.6%

CLOSURE

This memo describes Stantec's basis and rationale for the drainage infrastructure that is to be used for the Sackum Overhead Replacement Project.

Regards,

STANTEC CONSULTING LTD.

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APPENDIX A: IDF_CC Tables and Curve Data compiled on November 17, 2023

IDF for: LYTTON ID:1114741

Station Info | IDF historical data | IDF under climate change

Station name: LYTTON

ID: 1114741

Latitude: 50.23

Longitude: -121.58

Starting year: 1970

Ending year: 1990

Number of years (with data): 21

IDF for: LYTTON ID:1114741

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Climate Model Selection | SSP1.26 | SSP2.45 | SSP5.85 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

Total precipitation amounts presented in mm and precipitation intensity rates presented in mm/h for different return periods (T) presented in years

○ Total PPT (mm) ◉ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	19.56	26.58	32.34	39.01	41.19	49.24	58.85
10 min	14.17	19.06	23.74	30.27	32.62	41.95	53.00
15 min	11.69	15.09	18.51	23.50	25.32	33.10	42.06
30 min	9.22	12.42	15.18	18.70	19.85	24.34	29.65
1 h	7.30	9.78	11.78	14.09	14.82	17.54	20.79
2 h	5.94	8.11	9.65	11.04	11.49	12.96	14.39
6 h	3.87	5.02	5.82	6.58	6.81	7.60	8.34
12 h	2.58	3.42	3.86	4.21	4.32	4.60	4.84
24 h	1.56	2.08	2.35	2.56	2.63	2.80	2.94

IDF for: LYTTON ID:1114741

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Climate Model Selection | SSP1.26 | SSP2.45 | SSP5.85 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

The table below provides the coefficients for the interpolated equations fitted to the average IDF for future scenario RCP 8.5

T (years)	Coefficient A	Coefficient B	Coefficient t_0
2	7.5	-0.454	0.061
5	9.9	-0.450	0.052
10	12.1	-0.479	0.072
20	14.9	-0.523	0.097
25	15.8	-0.535	0.102
50	19.4	-0.586	0.124
100	23.4	-0.635	0.143

Use the coefficients provided in the table above with the following equation:

$$i\left(\frac{mm}{h}\right) - A \cdot (t + t_0)^B$$

Where:

i is the precipitation intensity rate in $\frac{mm}{h}$
A, B and t_0, are the coefficients for each return period (T) in years
t, the time (duration) of the precipitation event in hours (h)

IDF for: LYTTON ID:1114741

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

○ Total PPT (mm) ◉ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	17.35	23.04	27.55	32.52	34.25	40.06	46.65
10 min	12.50	16.64	20.60	25.68	27.62	34.80	44.21
15 min	10.27	13.20	16.18	20.19	21.76	27.80	36.08
30 min	8.17	10.80	13.04	15.65	16.59	19.86	23.78
1 h	6.48	8.48	10.03	11.73	12.31	14.26	16.45
2 h	5.27	6.98	8.09	9.14	9.47	10.48	11.47
6 h	3.44	4.32	4.90	5.45	5.62	6.14	6.65
12 h	2.28	2.91	3.23	3.48	3.55	3.75	3.90
24 h	1.37	1.77	1.96	2.12	2.16	2.28	2.37

IDF for: LYTTON ID:1114741

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

The table below provides coefficients for the interpolation equations fitted to the IDF curve using the GEV distribution.

T (years)	Coefficient A	Coefficient B	Coefficient t_0
2	6.6	-0.454	0.063
5	8.6	-0.460	0.061
10	10.4	0.490	0.078
20	12.5	-0.528	0.097
25	13.3	-0.542	0.104
50	15.9	-0.588	0.122
100	18.9	-0.637	0.139

Use the coefficients provided in the table above with the following equation:

$$i\left(\frac{mm}{h}\right) = A \cdot (t + t_0)^B$$

Where:

i is the precipitation intensity rate in $\frac{mm}{h}$
A, B and t_0, are the coefficients for each return period (T) in years
t, the time (duration) of the precipitation event in hours (h)

Ungauged IDF for: Lat: 50.31120 °, Lon: -121.39583 °

Station Info | IDF historical data | IDF under climate change

Station name: Ungauged IDF

Ungauged IDF for the following coordinates:

Latitude: 50.31120 °

Longitude: -121.39583 °

Ungauged IDF for: Lat: 50.31120 °, Lon: -121.39583 °

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Climate Model Selection | SSP1.26 | SSP2.45 | SSP5.85 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

Total precipitation amounts presented in mm and precipitation intensity rates presented in mm/h for different return periods (T) presented in years

○ Total PPT (mm) ◉ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	20.23	28.17	34.79	42.22	44.63	53.31	62.27
10 min	14.58	20.32	25.88	33.12	35.53	45.42	57.45
15 min	11.99	16.12	20.30	26.01	27.91	36.07	46.49
30 min	9.43	12.99	16.10	19.75	20.96	25.45	30.35
1 h	7.50	10.20	12.37	14.73	15.49	18.16	20.82
2 h	6.00	8.22	9.78	11.30	11.75	13.22	14.46
6 h	3.89	5.08	5.92	6.73	6.97	7.76	8.43
12 h	2.61	3.45	3.93	4.35	4.43	4.75	4.95
24 h	1.59	2.12	2.44	2.70	2.76	2.98	3.12

Ungauged IDF for: Lat: 50.31120 °, Lon: -121.39583 °

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Climate Model Selection | SSP1.26 | SSP2.45 | SSP5.85 | Comparison Graphs

Tables | Plots | Interpolation Equations | Box Plot - Uncertainty

The table below provides the coefficients for the interpolated equations fitted to the average IDF for future scenario RCP 8.5

T (years)	Coefficient A	Coefficient B	Coefficient t_0
2	7.6	-0.453	0.053
5	10.2	-0.458	0.047
10	12.6	-0.485	0.058
20	15.5	-0.521	0.073
25	16.4	-0.534	0.078
50	19.8	-0.577	0.093
100	23.4	-0.622	0.107

Use the coefficients provided in the table above with the following equation:

$$i\left(\frac{mm}{h}\right) = A \cdot (t + t_0)^B$$

Where:

i is the precipitation intensity rate in $\frac{mm}{h}$
A, B and t_0, are the coefficients for each return period (T) in years
t, the time (duration) of the precipitation event in hours (h)

Ungauged IDF for: Lat: 50.31120 º, Lon: -121.39583 º

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

Total precipitation amounts are presented in mm and precipitation intensity rates are presented in mm/h for different return periods (T) presented in years

○ Total PPT (mm) ◉ Intensity rates (mm/h)

T (years)	2	5	10	20	25	50	100
5 min	17.93	24.17	29.14	34.77	36.58	43.08	50.52
10 min	12.92	17.43	21.67	27.28	29.12	36.71	46.60
15 min	10.63	13.83	17.00	21.42	22.87	29.15	37.71
30 min	8.36	11.15	13.49	16.27	17.17	20.57	24.62
1 h	6.64	8.75	10.36	12.13	12.69	14.68	16.89
2 h	5.32	7.05	8.19	9.31	9.63	10.68	11.73
6 h	3.45	4.36	4.96	5.55	5.71	6.27	6.84
12 h	2.31	2.96	3.29	3.58	3.63	3.84	4.02
24 h	1.41	1.82	2.04	2.23	2.27	2.41	2.53

Ungauged IDF for: Lat: 50.31120 º, Lon: -121.39583 º

Station Info | IDF historical data | IDF under climate change

GEV | Gumbel

Tables | Plots | Interpolation Equations

The table below provides coefficients for the interpolation equations fitted to the IDF curve using the GEV distribution.

T (years)	Coefficient A	Coefficient B	Coefficient t_0
2	6.7	-0.453	0.053
5	8.8	-0.458	0.047
10	10.6	-0.485	0.058
20	12.7	-0.521	0.073
25	13.4	-0.534	0.078
50	16.0	-0.577	0.093
100	19.0	-0.622	0.107

Use the coefficients provided in the table above with the following equation:

$$i\left(\frac{mm}{h}\right) = A \cdot (t + t_0)^n$$

Where:

i is the precipitation intensity rate in $\frac{mm}{h}$
A, B and t_0, are the coefficients for each return period (T) in years
t, the time (duration) of the precipitation event in hours (h)

APPENDIX B: Ditch Capacity Requirements

Appendix B: Flow calculations and ditch sizing for Sackum Overhead

Sackum Hydrology									Hathaway	Hathaway	Historic	
Watershed/Cul. #	Approximate Station Range	A (Ha)	A (m^2)	A (km^2)	r (for Hathaway)	L (m)	High Elev. (m)	Low Elev. (m)	Melton Ratio (Elev.high-Elev.Low)/ SqrtArea	s (m/m)	T_c (min)	Tc (hr)
Area 1	106+30-107+40	0.3	2906.000	0.003	0.4	10	270	260	0.2	1.0000	3	0.05
Area 2	106+10+106+30	14.0	140468.000	0.140	0.4	989	560	260	0.8	0.3033	31	0.52
Area 3	105+90 - 106+10	5.8	57855.000	0.058	0.4	936	540	260	1.2	0.2991	30	0.51
Area 4	105+00 - 105+90	3.9	39251.000	0.039	0.4	349	400	260	0.7	0.4011	18	186

			Climate Change									
Watershed/Cul. #	A_{25}	B_{25}	t_o	$i_{historic}$ (mm/h)	A	B	t_o	i_{cc} (mm/h)	C	c	Q_{cc}25	Q_{cc}25 CUMULATIVE
Area 1	13.3	-0.542	0.104	37.1898	15.8	-0.535	0.102	43.9117	0.300	0.01	0.01	0.01
Area 2	13.3	-0.542	0.104	17.1797	15.8	-0.535	0.102	20.3766	0.300	0.20	0.24	0.25
Area 3	13.3	-0.542	0.104	17.3545	15.8	-0.535	0.102	20.5819	0.300	0.08	0.10	0.35
Area 4	13.3	-0.542	0.104	0.7834	15.8	-0.535	0.102	0.9654	0.300	0.00	0.003	0.35

Appendix B: Conveyance Pipe Adjacent to Existing Bin Wall

Area Number	Flow, Q, (m3/s)	Cumulative Flow, Qcumu, (m3/s)	Slope (%)	Pipe Diameter, D, (mm)	Internal Pipe Diameter, ID, (mm)	Pipe, Wetted Perimeter, Pw, (m)	Pipe, Cross Sectional Area, Axs (m2)	Hydraulic Radius, Rh, (m)	Pipe Capacity, Qcap, (m3/s)	Pipe Full Velocity, v (m/s)	Pipe Length, L (m)	Travel Time (min)	% Design Capacity	Pipe Material	n, Manning's Number
	Q = C x i x A / 360					= pi x ID	= pi x (ID/2)^2	= Axs / Pw	= (1/n) x Axs x Rh^2/3 x S^1/2	= Qcap/Axs		= L/v	= Qcap / Qcumu		
Area 1	0.01063	0.01063	2.25%	300	298	1.8724	0.0697	0.03725	0.0486	0.7	30	0.72	21.9%	CSP	0.024

APPENDIX C: Ditch Sizing and Velocities

Ditch Area 1:

Manning's Roughness = 0.035, III B, 2, Grass, some weeds 0.030-0.035				
Area 1				
Inputs V- Ditch				
h	Height	=	0.11	m
ss1	fore slope	=	2	:1
ss2	back slope	=	2	:1
S	channel slope	=	0.019	m/m
n	manning's roughness	=	0.035	
Qcc	max flow, 25 year return period	=	0.01	m^3/s
Calculations				
W1	Width1		0.22	
W2	Width2		0.22	
H1	Hypotenuse1	=	0.25	
H2	Hypotenuse2	=	0.25	
A	(1/2)*(W1+W2)*h	=	0.02	m^2
P	(sqrt(W1^2+h^2))+(sqrt(W2^2+h^2))	=	0.49	m
R	A/P	=	0.05	m
v	R^0.67*S^0.5/n	=	0.52	m/s
v	parallel against bank	=	0.35	m/s
Vm	Direct Impingement = 2 * v		1.05	m/s
Q	v*A	=	0.01	m^3/s
Check				
Flows accomodated			TRUE	

Ditch Area 2:

Manning's Roughness = 0.035, III B, 2, Grass, some weeds 0.030-0.035				
Area 2				
Inputs V- Ditch				
h	Height	=	0.34	m
ss1	fore slope	=	2	:1
ss2	back slope	=	2	:1
S	channel slope	=	0.019	m/m
n	manning's roughness	=	0.035	
Qcc	max flow, 25 year return period	=	0.25	m^3/s
Calculations				
W1	Width1		0.68	
W2	Width2		0.68	
H1	Hypotenuse1	=	0.76	
H2	Hypotenuse2	=	0.76	
A	(1/2)*(W1+W2)*h	=	0.23	m^2
P	(sqrt(W1^2+h^2))+(sqrt(W2^2+h^2))	=	1.52	m
R	A/P	=	0.15	m
v	R^0.67*S^0.5/n	=	1.11	m/s
v	parallel against bank	=	0.74	m/s
Vm	Direct Impingement = 2 * v		2.23	m/s
Q	v*A	=	0.26	m^3/s
Check				
Flows accomodated			TRUE	

Ditch Area 3:

Manning's Roughness = 0.035, III B, 2, Grass, some weeds 0.030-0.035				
Area 3				
Inputs V- Ditch				
h	Height	=	0.38	m
ss1	fore slope	=	2	:1
ss2	back slope	=	2	:1
S	channel slope	=	0.019	m/m
n	manning's roughness	=	0.035	
Qcc	max flow, 25 year return period	=	0.35	m^3/s
Calculations				
W1	Width1		**0.76**	
W2	Width2		**0.76**	
H1	Hypotenuse1	=	**0.85**	
H2	Hypotenuse2	=	**0.85**	
A	(1/2)*(W1+W2)*h	=	**0.29**	m^2
P	(sqrt(W1^2+h^2))+(sqrt(W2^2+h^2))	=	**1.71**	m
R	A/P	=	**0.17**	m
v	R^0.67*S^0.5/n	=	**1.21**	m/s
v	parallel against bank	=	**0.80**	m/s
Vm	Direct Impingement = 2 * v		**2.41**	m/s
Q	v*A	=	**0.35**	m^3/s
Check				
Flows accomodated			TRUE	

Ditch Area 4:

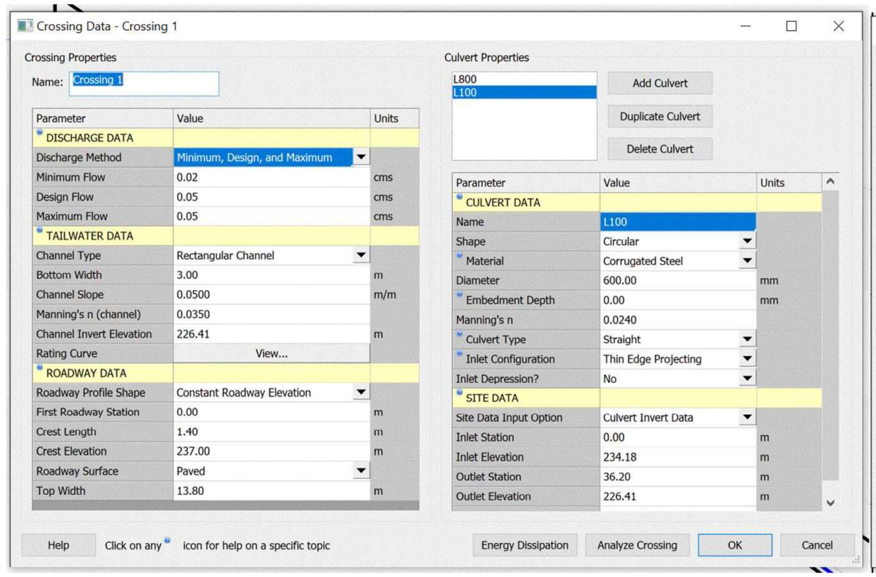
Manning's Roughness = 0.035, III B, 2, Grass, some weeds 0.030-0.035				
Area 4				
Inputs V- Ditch				
h	Height	=	0.38	m
ss1	fore slope	=	2	:1
ss2	back slope	=	2	:1
S	channel slope	=	0.019	m/m
n	manning's roughness	=	0.035	
Qcc	max flow, 25 year return period	=	0.35	m^3/s
Calculations				
W1	Width1		**0.76**	
W2	Width2		**0.76**	
H1	Hypotenuse1	=	**0.85**	
H2	Hypotenuse2	=	**0.85**	
A	(1/2)*(W1+W2)*h	=	**0.29**	m^2
P	(sqrt(W1^2+h^2))+(sqrt(W2^2+h^2))	=	**1.71**	m
R	A/P	=	**0.17**	m
v	R^0.67*S^0.5/n	=	**1.21**	m/s
v	parallel against bank	=	**0.80**	m/s
Vm	Direct Impingement = 2 * v		**2.41**	m/s
Q	v*A	=	**0.35**	m^3/s
Check				
Flows accomodated			TRUE	

APPENDIX D: Culvert Sizing Calculations and Images

Appendix D Figure 1: Culvert Sizing Calculations

Sackum - Cross Culvert -									Hathaway	Melton Ratio	Historic	Historic			Climate Change								Pipe Sizes - Rational
Watershed/Cul. #	A (Ha)	A (km^2)	r	L (m)	High Elev. (m)	Low Elev. (m)	s (m/m)	T_c (hr)	T_c (min)		A_{100}	B_{100}	t_o	$i_{historic}$	A	B	t_o	i_{cc}	C	$Q_{historic}$	Q_{CC}	% CC	Suggested
Cross-Culvert 1-ungauged	2.0	0.020	0.4	397	240	220	0.0504	0.5164	30.9842	0.141	19	-0.622	0.107	25.4922	23.4	-0.622	0.107	31.3956	0.300	0.0426	0.0525	23.2	400

Appendix D Figure 2: Culvert Inputs for Hy-8

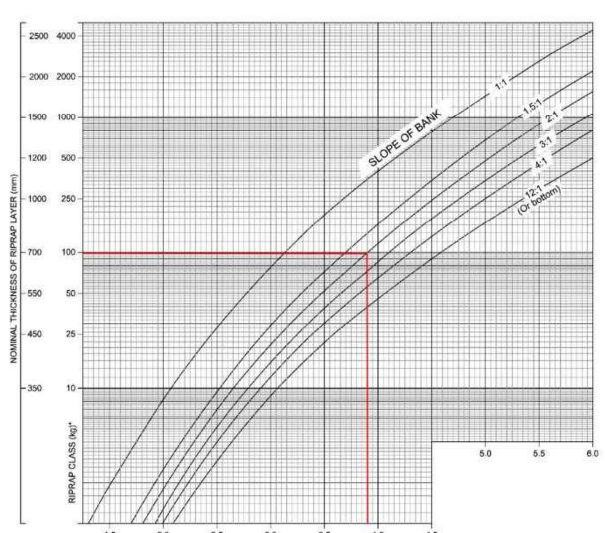


Appendix D Figure 3: Hy-8 Analysis for Culvert

Culvert Summary Table - L100

Total Discharge	Culvert Discharge	Headwater Elevation	Inlet Control Depth(m)	Outlet Control Depth(m)	Flow	Normal Depth	Critical Depth	Outlet Depth	Tailwater Depth	Outlet Velocity (m/s)	Tailwater Velocity (m/s)
0.02	0.02	234.29	0.11	0.0*	1-S2n	0.04	0.09	0.04	0.02	1.65	0.41
0.02	0.02	234.29	0.11	0.0*	1-S2n	0.05	0.09	0.05	0.02	1.82	0.43
0.03	0.03	234.30	0.12	0.0*	1-S2n	0.05	0.10	0.05	0.02	2.02	0.45
0.03	0.03	234.31	0.13	0.0*	1-S2n	0.06	0.11	0.06	0.02	2.26	0.47
0.03	0.03	234.32	0.14	0.0*	1-S2n	0.06	0.11	0.06	0.02	2.50	0.49
0.03	0.03	234.32	0.14	0.0*	1-S2n	0.06	0.12	0.06	0.02	2.75	0.51
0.04	0.04	234.33	0.15	0.0*	1-S2n	0.06	0.12	0.06	0.02	3.01	0.53
0.04	0.04	234.34	0.16	0.0*	1-S2n	0.06	0.13	0.06	0.03	3.30	0.54
0.04	0.04	234.34	0.16	0.0*	1-S2n	0.07	0.13	0.07	0.03	3.57	0.56
0.05	0.05	234.35	0.17	0.0*	1-S2n	0.07	0.14	0.07	0.03	3.90	0.57
0.05	0.05	234.35	0.17	0.0*	1-S2n	0.07	0.14	0.07	0.03	4.21	0.59

Appendix D Figure 4: Riprap Sizing for Culvert Outlet



APPENDIX E: Pavement Drainage Calculations

Stantec

		Design Input	105+00 to 106+37 (LOC to SE development)	106+37 (SE development) to 107+44.8 (HP)	107+57 (end of barrier) to 107+76 (bridge abutment)	107+74 (last CB in advance of structure) to 109+13	109+13 to 110+71	111+00 (begin barrier flare) to 112+55 end of SE	112+55 end of SE to 113+40 LOC
SW	=	Paved Shoulder width	2 m	2 m	2 m	2 m	2 m	2 m	2 m
sy	=	Longitudinal grade	0.01909 m/m	0.01909 m/m	0.0065 m/m	0.0015 m/m	0.068 m/m	0.068 m/m	0.068 m/m
sx	=	Crossfall	0.023 m/m	0.023 m/m	0.023 m/m	0.03 m/m	0.053 m/m	0.053 m/m	0.02 m/m
n	=	Manning's roughness coefficient	0.016 asphalt; RTAC Table 3.2.3 V1 B2	0.016 asphalt; RTAC Table 3.2.3 V1 B2	0.016 asphalt; RTAC Table 3.2.3 V1 B2	0.016 asphalt; RTAC Table 3.2.3 V1 B2	0.016 asphalt; RTAC Table 3.2.3 V1 B2	0.016 asphalt; RTAC Table 3.2.3 V1 B2	0.016 asphalt; RTAC Table 3.2.3 V1 B2
i	=	Rainfall intensity corresponding to tc equal to 5 minutes, 5 year return period	12.74 mm/hr	12.74 mm/hr	12.74 mm/hr	12.74 mm/hr	12.74 mm/hr	12.74 mm/hr	12.74 mm/hr
width	=	Effective width of contributing area	16.25 m	11.8 m	11.8 m	11.2 m	11.8 m	11.8 m	6.2 m
Cw	=	Width weighted runoff coefficient	0.95 asphalt	0.95 asphalt	0.95 asphalt	0.95 asphalt	0.95 asphalt	0.95 asphalt	0.95 asphalt
w	=	Grate catchment width	0.625 m	0.625 m	0.625 m	0.625 m	0.625 m	0.625 m	0.625 m
Note:		w = 0.625m for Depressed B.C. Bicycle Safe Grate							
		w = 0.600m for paved spillway							
PW	=	IF(SW<1.85,1.2,SW*0.65)	1.3 m	1.3 m	1.3 m	1.3 m	1.3 m	1.3 m	1.3 m
yo	=	PW*sx	0.0299 m	0.0299 m	0.0299 m	0.039 m	0.0689 m	0.0689 m	0.026 m
Rs	=	sx/sy	1.2048	1.2048	3.5385	20.0000	0.7794	0.7794	0.2941
weff	=	if(Rs<5.1,1.1*w,if(Rs<10.1,1.2*w,if(Rs<15.2,1.3*w,if(Rs<20.1,1.4*w,1.5*w))))	0.6875 m	0.6875 m	0.6875 m	0.875 m	0.6875 m	0.6875 m	0.6875 m
v	=	yo^0.67*sy^0.5/n	0.8222 m/s	0.8222 m/s	0.4798 m/s	0.2754 m/s	2.7148 m/s	2.7148 m/s	1.4131 m/s
Qo	=	0.375*sy^0.5*yo^2.67/(n*sx)	0.011985 m³/s	0.011985 m³/s	0.006993 m³/s	0.005236 m³/s	0.091187 m³/s	0.091187 m³/s	0.017911 m³/s
yover	=	(PW-weff)*sx	0.0140875 m	0.0140875 m	0.0140875 m	0.01275 m	0.0324625 m	0.0324625 m	0.01225 m
Qover	=	.375*sy^0.5*yover^2.67/(n*sx)	0.0016068 m³/s	0.0016068 m³/s	0.0009376 m³/s	0.0002646 m³/s	0.0122259 m³/s	0.0122259 m³/s	0.0024014 m³/s
Qint	=	if(v<2,Qo-Qover,if(v<2.5,Qo-1.1*Qover,if(v<3,Qo-1.2*Qover,Qo-1.3*Qover)))	0.010378 m³/s	0.010378 m³/s	0.006056 m³/s	0.004971 m³/s	0.076516 m³/s	0.076516 m³/s	0.015509 m³/s
Eff	=	Qint/Qo*100	86.5926 %	86.5926 %	86.5926 %	94.9468 %	83.9111 %	83.9111 %	86.5926 %
CBone	=	if(v<2,Qo/(Cw*i*width/(360*10000)),1.2*Qo/(Cw*i*width/(360*10000)))	219.33 m	302.04 m	176.24 m	139.02 m	2757.72 m	2757.72 m	859.09 m
CBtwo	=	if(v<2,Qint/(Cw*i*width/(360*10000)),1.2*Qint/(Cw*i*width/(360*10000)))	189.92 m	261.54 m	152.61 m	131.99 m	2314.03 m	2314.03 m	743.91 m
		RECOMMENDATIONS:	219 m first spacing, 190 m after	302 m first spacing, 262 m after	176 m first spacing, 153 m after	139 m first spacing, 132 m after	2758 m first spacing, 2314 m after	2758 m first spacing, 2314 m after	859 m first spacing, 744 m after