City of Vaughan

Huntington Road EA

Drainage and Hydrology
Final Report

March 2017

Revised September 2017

Sanchez Engineering Inc.

for

PARSONS

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1.0 INTRODUCTION

1.1 General

Parsons was retained by the City of Vaughan to provide the consulting services required for the Environmental Assessment Study for Part A – Huntington Road from Langstaff Road to McGillivray Road and Part B – Huntington Road from Major Mackenzie Drive to Nashville Road, a total of 6.2 km±. Parsons retained Sanchez Engineering Inc. to undertake the Drainage and Hydrology component of the assignment. This report summarizes the results of the drainage and hydrology investigations and studies completed in accordance with the Terms of Reference prepared by the City of Vaughan.

1.2 Description of Project

Figure 1 illustrates the location of the study area within the City of Vaughan. In Part A, the EA Study considered widening of Huntington Road from the existing 2-lanes to future 4-lanes. In Part B, The EA examined urbanization of the road section and adding turning lanes where warranted.

Figure 1 – Study Area (from Vaughan RFP)

Huntington Road is classified as a Major Collector Road in the City of Vaughan Official Plan. The section of Huntington Road from McGillivray Road to Major Mackenzie Drive does not form part of this Study, as this section will form part of the future Highway 427 when it is extended (c.f. 427 Transportation Corridor EA, by MRC, January 2010).

Stormwater Management is required for the road segments described above. Culvert hydrology studies are required for a total of six crossings, as summarized in Section 4.0 of this Report. In addition to the culverts, there are five smaller crossings that can be classified as drainage crossings.

1.3 Drainage and Watercourses

The Study Area is located within the watersheds of the Rainbow and Robinson Creeks, which are tributaries of the West Humber River. External drainage to Huntington Road is mainly from the west. The watersheds of the watercourses within the Study Area are shown on Figure 2.

The Humber River - State of the Watershed report by the Toronto Region Conservation Authority classifies Rainbow Creek as a “small riverine warmwater” watercourse, and Robinson Creek as a “small and intermediate riverine warmwater” watercourse.

The Natural Heritage - Existing Conditions draft report by SLR Consultants states that the Study Area is located within the South Slope physiographic region, consisting of smooth, faintly drumlinized clay till plain with deeply incised stream valleys. The overburden consists of coarse-textured glaciolacustrine deposits of sand and gravel with some silt and clay in the segment north of McGillvray Road; and fine-textured glaciolacustrine deposits of silt and clay with some sand and gravel in the segment south of McGillvray Road.

1.4 Drainage Design Objectives

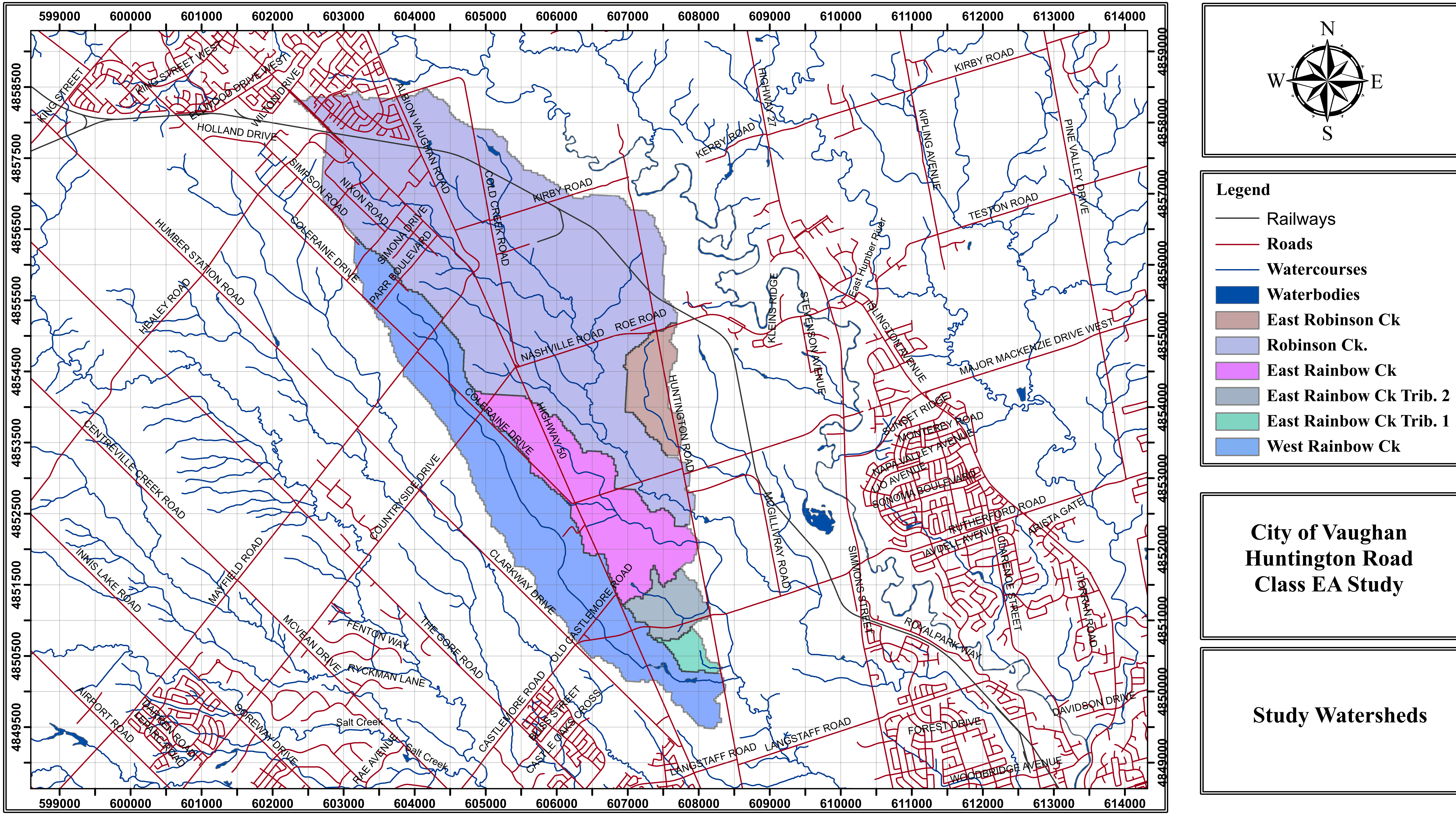
The broad goals of the Drainage and Hydrology Report for the Huntington Road EA can be summarized as follows:

- To minimize risk to road users while meeting current design standards.
- To promote the protection and preservation of the functional integrity of the ecosystems which form the road's environment.

To achieve these goals, the drainage design must meet the following drainage design objectives:

- To develop a cost-effective drainage strategy for management of the Huntington Road storm water drainage.
- To incorporate roadway safety in the design of the drainage system.
- To minimize impacts of storm runoff on upstream and downstream reaches of the watercourses receiving the road's storm runoff.

As part of the Drainage and Hydrology Report, the Study addresses the transverse drainage waterway requirements for structures located on watercourses crossed by Huntington Road within the Study Area. The hydraulic design of these structures must meet the current MTO Highway Drainage Design Standards.



City of Vaughan Huntington Road Class EA Study

Study Watersheds

Sanchez Engineering Inc.

1:50,000

1,250 625 0 1,250 2,500 3,750 5,000 Meters

Contour Interval 1 Metre

All data is provided by the Ontario Ministry of Natural Resource Land Information Ontario (LIO)

2.0 BACKGROUND INFORMATION

The following reports and drawings were available as part of the background data for the project:

2.1 Reports

- 427 Transportation Corridor Environmental Assessment, Final EA Report, MRC, January 2010
- Proposed Collector Street Between Huntington Road and Regional Road 27, PIC No. 3 Report, April 2015
- Vaughan Transportation Master Plan, Appendix E – Existing Environmental Conditions, October 2009
- Vaughan Stormwater Management Master Plan, June 2014
- Vaughan Rainbow Creek Master Plan Update Study, June 2014
- Draft Natural Heritage - Existing Conditions and Impact Assessment Report, SLR Consulting (Canada), November 2014
- Humber River Hydrology Update, Civica, TRCA, June 2015

2.2 Official Plans

- City of Brampton Official Plan
- Town of Caledon Official Plan
- City of Vaughan Official Plan
- Region of York Official Plan

2.3 Drawings

- Survey Request Roll, Parsons
- Preliminary Design Concept Roll, Parsons
- Proposed Alignment, including 2014 Aerial Photography

2.4 Maps and Aerial Photographs

- Toronto and Region Conservation Authority Flood Plain Mapping Sheets 31, 44, and 153
- ArcGIS data, Land Information Ontario
- Ontario Flow Assessment Tools III, Ministry of Natural Resources and Forestry
- 2014 Aerial Photography
- Road Classification Code Map, City of Vaughan, 2011

2.5 Design Guidelines and Criteria Documents

The Government of Ontario, through the Ministries of Transportation, of the Environment, and of Natural Resources and Forestry has prepared a number of directives and manuals that outline the approach and guidelines to be used in the development of drainage and storm water management strategies and designs for highway and roadway projects. Guidelines for drainage and storm water management have also been prepared by other government agencies. The drainage strategy and design presented in this Drainage Report takes into consideration the following documents:

- Canadian Highway Bridge Design Code and Commentary, 2006
- MTO Drainage Management Manual, 1997
- MTO Highway Drainage Design Standards, 2008
- Ontario Provincial Standard Specifications
- MOE Storm Water Management Planning and Design Manual, 2003
- MNR Environmental Guidelines for Access Roads and Water Crossings, 1990
- Guidelines on Erosion and Sediment Control for Urban Construction Sites, 1987
- Erosion & Sediment Control Guideline for Urban Construction, 2006
- TRCA Stormwater Management Criteria, 2012
- Low Impact Development Stormwater Management Planning and Design Guide, 2010
- TRCA Crossings Guideline for Valley and Stream Corridors, 2015
- MNR Fish Habitat Protection Guidelines for Developing Areas, March 1994
- MTO PHY Directive B-014 MTO Drainage Management Policy and Practice, dated July 2007
- MTO PHY Directive B-100 MTO Design Flood Criteria, dated October 1980

2.6 Survey Data

Field survey data for preliminary design was collected by J.D. Barnes Limited in 2014.

2.7 Hydrology and Hydraulics Data

Hydrology and Hydraulics data were available from the Toronto and Region Conservation Authority, in the form of HEC-RAS input data and the corresponding floodline maps noted in Section 2.4.

3.0 DESIGN CRITERIA

3.1 Right-of-Way Drainage

The MTO Highway Drainage Design Standards require that the roadside drainage be designed to convey the 10-year storm flow, and that the major system be capable of conveying the 100-year storm flow with limited flooding of the road traffic lanes.

3.2 Stormwater Management

The main criteria for stormwater quality control are based on the protection of the Rainbow Creek and Robinson Creek tributaries. Therefore, the best management practices proposed must be capable of achieving Enhanced (Level 1) level of protection, as defined in the MOE Stormwater Management Planning & Design Manual, 2003 (MOE SWM Manual). The MOE SWM Manual states that Enhanced protection or greater should be used when sensitive aquatic habitat will be affected by the stormwater discharge. For example, receiving waters that have aquatic communities that require low suspended solids concentrations in their habitat.

The MOE SWM Manual indicates that the levels of protection are based on a general relationship between the long-term average suspended solids discharged to the watercourse and the lethal and chronic effects of suspended solids on aquatic life and habitat. The levels of protection correspond to long-term average suspended solids removal by a SWM facility for the entire range of rainfall events on that site for a long period of time (the MOE SWM Manual requires a period of at least 10 years). The use of a long-term average takes into account the variability in characteristics of rainfall events and the corresponding storm runoff outcomes. Enhanced protection corresponds to the long-term average removal from storm runoff of 80% of total suspended solids.

Additional design criteria are provided by the TRCA 2012 Stormwater Management Criteria and 2010 Low Impact Development Stormwater Management Planning and Design Guide

3.3 Transverse Drainage

3.3.1 Design Flow

The MTO Highway Drainage Design Standards outline the Ministry's policy for the design of highway structures and other drainage facilities. These Standards are generally accepted as the Standards for municipal road drainage in the Province. Based on the Standards, for the case of Huntington Road, a major collector road, the design flood for bridges and culverts with a total span of up to 6.0 m is the 25 year event, and for structures with a total span over 6.0 m is the 50 year return period flow (HDD Standard WC-1 - Design Flows Bridges and Culverts).

3.3.2 Regional Storm

The MTO Highway Drainage Design Standards also provide that consideration shall be given to the Regional flood where the drainage structure could increase the flooding of buildings or developable land during a Regional flood. Relief flow shall be provided wherever feasible at bridge or culvert crossings as required to accommodate such floods. In the case of Huntington Road, a major collector road, the standard permits overtopping of the road by the Regional storm. There is no existing standard that limits the flood depth under the Regional storm during overtopping of a road.

3.3.3 Vertical Clearance

The Canadian Highway Bridge Design Code requirements for vertical clearance for bridge structures are that subject to other constraints – such as highway geometric design and local topography – for the design flood, the minimum vertical clearance between the soffit and the high water level should be at least 1.0 m (Clause 1.9.7 of CHBDC).

The Code allows for the clearance to be less than 1.0 m, if it is approved by the Road Authority. (Clause 1.9.7.1). Cases where the clearance may need to be less than 1.0 m include an existing road (as is the case in this project), where the existing constraints may preclude that the minimum clearance requirement is met.

For closed invert culverts, the Code does not provide any limit on the minimum vertical clearance. However, the MTO Highway Drainage Design Standards (Standard WC provide that the ratio of the headwater depth at the inlet (HW) to the Diameter/Rise of the culvert (D), shall be less than 1.5 for culverts less than 3.0 m high (i.e. rise under 3.0 m). In other words, the headwater level can be above the culvert soffit.

The MTO Highway Drainage Design Standards note in WC-7 Culvert Crossings on a Watercourse that there is no clearance requirements for Closed Footing Culverts or Open Footing Culverts with non-erodible bottom. Notwithstanding this, it is current practice in the Ministry normally to permit conventional closed-invert culverts to flow full at the inlet; however, in some cases submergence of an existing culvert or its extension may obviate the need for an expensive replacement structure. This is particularly applicable to this project, where replacing existing culverts would be a significant expense.

3.3.4 Freeboard

The Canadian Highway Bridge Design Code states that, where geometric and other considerations permit, the approach grade shall be set so that for the design flood, the minimum freeboard from the edge of the through traffic lanes to the high water level is at least 1.0 m. (Clause 1.9.8.1 and 1.9.8.2 of CHBDC).

Cases where the freeboard may need to be set at less than 1.0 m include an existing road (as is the case in this project), where the existing constraints may preclude that the minimum 1.0 m freeboard be achieved.

4.0 HYDROLOGY INVESTIGATIONS

4.1 Watercourses

Field inspections were conducted during September 2014 and August 2015 to examine the condition of the outside ditch drainage system. The field investigations reviewed in general the condition of the existing culverts, including examination of the inlets and outlets to detect hydraulic and erosion problems.

Table 1 - Watersheds

Crossing Station	Waterscourse Name	Drainage Area (ha)
1+330	West Rainbow Creek	612
2+460	Rainbow Creek Tributary	71
3+320	East Rainbow Creek	307
3+768	Robinson Creek	1480
4+490	East Robinson Creek	94

In addition to the watersheds noted in **Table 1**, there are six additional crossings that drain watershed areas smaller than 70 ha (see **Table 4**).

The draft Natural Heritage SLR report indicates that all the crossings in Table 1 have fish habitat or support fishery directly. In addition, the smaller drainage culvert crossings provide indirect fishery support.

4.2 Watershed Characteristics

Table 2 summarizes the watershed characteristics of the five largest crossings within the Study Area. In addition to these crossings, six additional unlisted minor crossings exist, which are drained under the roadway by 600 mm or 750 mm diameter CSP culverts (**Table 4**).

Table 2 - Transverse Drainage

Crossing Station	Drainage Area (ha)	Mean Slope %	Length of Channel (km)	Curve Number CN	Time of Concentration (hr)
1+330	612	1.115	11.7	84	5.72
2+460	71	2.862	1.90	86	0.96
3+320	307	1.228	6.04	86	3.11
3+768	1480	1.361	13.4	77	5.75
4+687	94	1.065	2.85	67	1.70

The five crossings receive runoff from tributaries of Robinson and Rainbow Creek. The watersheds for these six watercourses were delineated using the Ontario Flow Assessment Tools III, developed by the Ministry of Natural Resources and Forestry, based on data by Land Information Ontario.

Transverse drainage within the Project limits is provided by the culverts listed in **Table 2**. All culverts are corrugated steel pipe or pipe arch culverts. In addition, five other small culverts, intended to convey local runoff from one side of the road to the other or which carry mainly overland flow without a well-defined watercourse, are located within the project limits. The minor runoff crossing culverts are 600mm diameter corrugated steel pipe culverts.

The watercourse culverts are summarized in **Table 3.** It is noted that in addition to the culverts listed in **Table 3**, smaller culverts, ranging from 600 mm to 750 mm diameter, provide local drainage.

Table 3 Existing Watercourse Culverts

Watercourse	Station	Culvert Type	Rise or Diameter (mm)	Span (mm)
West Rainbow Creek	1+330	SPCSPA	3200	2100
Rainbow Creek Tributary	2+460	CSPA	1880	1280
East Rainbow Creek	3+320	SPCSPA	2240	1630
Robinson Creek	3+768	SPCSP	3000	
East Robinson Creek	4+687	CSP	1800	1200

The six local drainage culverts are summarized in **Table 4**, including their drainage areas.

Table 4 Existing Local Drainage Culverts

Station	Culvert Type	Diameter (mm)	Drainage Area (ha)
1+140	CSP	600	8.2
1+525	CSP	750	27.0
2+985	CSP	600	9.5
5+793	CSP	600	3.1
5+979	CSP	600	5.4
6+226	CSP	600	4.7

4.3 Design Flows

The drainage areas for the transverse culverts assessed were delineated with the aid of the Ontario Flow Assessment Tools, a GIS-based system made available by the Ministry of Natural Resources and Forestry, based on the Land Information Ontario GIS data system. The drainage areas for all watercourses are shown in **Figure 2**.

4.3.1 Hydrologic Models

The hydrologic models of the watersheds were developed independently of the TRCA models, as the TRCA models were not available to us until later in the study. The models were developed using standard modelling methods and the Visual Hymo 4 software. The model input parameters are provided in Appendix A.

4.3.2 IDF Curves

The design storms used were the AES 12-hour duration storm distribution.

The MTO Intensity Duration Frequency curves, generated using their website IDF Curve Lookup, were used in the calculations to generate the design rain storms. The MTO IDF curves for the site are generated using a grid of existing IDF stations, using the Gumbel distribution and the method of moments (2 parameters). The IDF curves for the site are presented in Appendix A.

The City of Vaughan has developed a series of IDF curves for use in their projects, but were found to be generally lower for the longer durations, which are the ones that are necessary for the modelling of the watercourse flood hydrographs.

For durations of 90 minutes or less, the City's IDF curves are generally higher than the MTO curves, ranging between 15% and 20% higher. Nevertheless, the MTO IDF curves were used in this study for consistency with the remainder of the calculations.

A comparison of the two sets of IDF curves is included in Appendix A.

4.3.3 Calculated Peak Flows

Peak flows at each of the noted crossings were estimated using available information on land use; soil types and topography. The return period flows were calculated using the computer program Visual Otthymo. The results of the hydrologic analysis are presented in **Table 5**.

Table 5 Design Flows for Larger Crossings

Station	2 year (m^3/s)	5 year (m^3/s)	10year (m^3/s)	25 year (m^3/s)	50 year (m^3/s)	100 year (m^3/s)	Regional (m^3/s)
1+330	4.3	8.8	11.7	13.4	14.0	18.6	52.6
2+460	3.5	4.6	5.4	6.3	7.1	7.8	10.3
3+320	7.9	11.1	14.8	17.0	17.7	26.0	50.3
3+768	8.1	23.2	30.9	35.6	37.1	46.0	150.4
4+687	0.4	3.0	4.0	4.6	4.8	5.3	14.2

The flows presented in Table 5 are different from the corresponding (where available) flows calculated in the TRCA hydrologic models of the watercourses. A comparison of the flows is presented in Appendix A. The results indicate that the flows in Table 5 are generally higher than the TRCA flows.

For the local drainage CSP culverts, the flows were calculated using the Rational Method, and checked with the Visual Otthymo method. **Table 6** summarizes the flows for the local drainage crossings.

Table 6 Design Flows for Local Drainage Crossings

Station	2 year (m^3/s)	5 year (m^3/s)	10year (m^3/s)	25 year (m^3/s)	50 year (m^3/s)	100 year (m^3/s)
1+140	0.8	1.1	1.2	1.5	1.6	1.8
1+524	1.4	1.9	2.2	2.5	2.8	3.1
2+985	0.9	1.2	1.4	1.6	1.8	2.0
5+793	0.3	0.3	0.4	0.5	0.5	0.6
5+979	0.5	0.6	0.7	0.8	0.9	1.0
6+226	0.4	0.5	0.6	0.7	0.8	0.9

4.4 Hydraulic Analysis

4.4.1 Modelling

At the time that this study was started, the existing HEC-RAS data for the Study Area was not available for a variety of reasons. The hydraulic model for the study was developed using the 1m contours available from the Region of York, the road profiles for the existing roadways based on drawings for the road and culverts provided by the City, and the road profiles for the proposed improvements as designed by Parsons.

The culvert type and geometry used for the model of existing conditions were extracted from the as-built drawings for the Huntington Road reconstruction carried out in the 1990s. The drawings were provided by the City.

It is noted that the calculations in the model were started using critical depth at the downstream section of the model. The water surface profiles will converge to the actual water surface profiles after a few cross-sections. Since the flood levels of interest are at the crossing and upstream of it, by starting sufficiently far will assure that the model has converged at the site.

It is recognized that the model used in this study and the TRCA model (which was not available at the beginning of this study) are not identical. However, since the purpose of the model is to define the required waterway opening dimensions to meet the hydraulic capacity requirements, it is considered acceptable to compare the existing and proposed conditions based on the model developed in this study.

The culverts for the watercourses listed as Local Drainage were sized using the computer program HY8, of the U.S. Federal Highway Administration (FHWA).

4.4.2 Required Culvert Sizes

The required culvert sizes to provide hydraulic capacity for the 25 year flood at each of the culverts for the transverse drainage are summarized in Table 7. It is noted that the culvert spans will increase for the larger watersheds to meet fluvial geomorphology requirements (see Tables 8 and 9)>

It is noted that the culvert listed above for Station 4+687 may not be required depending on the timing of construction of the Hwy 427 Extension, as this section of Huntington Road would be removed to accommodate the highway.

For fisheries protection, some of the culverts will need to be countersunk to provide a streambed for fish passage. This will be determined during the design of the culverts.

Table B1 in **Appendix B** summarizes the geometric characteristics of the various large drainage culverts, including inverts, top of road elevations at the respective culvert, and the depth of flooding under the Regional Storm.

Table 7 Required Culvert Sizes						
Watercourse	Station	No. of Barrels	Type	Rise or Diameter (mm)	Span (mm)	Length (m)
Local Drainage	1+140	1	CSP	1200		26
West Rainbow Creek	1+330	2	Concrete Box	1800	2400	26
Rainbow Creek Tributary	1+524	1	CSP	1500		26
Rainbow Creek Tributary	2+460	1	Concrete Box	1500	2700	26
Local Drainage	2+985	2	CSP	900		26
East Rainbow Creek	3+320	1	Concrete Box	1800	3000	26
Robinson Creek	3+768	2	Concrete Box	3000	4000	26
East Robinson Creek	4+687	2	Concrete Box	1200	1800	26
Local Drainage	5+793	1	CSP	900		26
Local Drainage	5+979	1	CSP	900		26
Local Drainage	6+226	1	CSP	900		26

With respect to the culvert at Station 1+524, it was noted during the field inspection that the inlet of the culvert is at present covered and that runoff enters the culvert via two CSP pipes placed parallel to the ditch. This configuration will need to be modified in the detail design to ensure that the culvert has an open inlet, with capacity to pass the entire design flows.

4.4.3 Fluvial Geomorphology Considerations

In addition to the strictly hydraulic criteria, based on the MTO Highway Drainage Design Standards, it is necessary to address the fluvial geomorphology requirements of the TRCA. The fluvial geomorphological characteristics of each watercourse were assessed by Water's Edge and are summarized as follows:

Table 8 Fluvial Geomorphological Characteristics (after Water's Edge)

Watercourse	Station	Water's Edge Number	Slope (%)	Channel Width (m)	Channel Depth (m)	Substrate
Local Drainage	1+140	Cu-10				
West Rainbow Creek	1+330	Cu-9	0.78	2.81	0.23	Sand and gravel with cobbles
Rainbow Creek Tributary	1+524	Cu-8				
Rainbow Creek Tributary	2+460	Cu-7	0.51	1.1	0.05	Grass
Local Drainage	2+985					
East Rainbow Creek	3+320	Cu-6	0.31	1.47	0.22	Sandy silt with some gravel
Robinson Creek	3+768	Cu-5	0.28	5.24	0.48	Sandy Gravel
East Robinson Creek	4+687	Cu-4	0.30	1.66	0.03	Grass
Local Drainage	5+793	Cu-3				
Local Drainage	5+979	Cu-2				
Local Drainage	6+226	Cu-1				

The local drainage watercourses were not assessed in detail by Water's Edge.

The TRCA *Crossing Guideline for Valley and Stream Corridors* recommends that replacement crossings be sufficiently wide to reduce their impact on the watercourse. An accepted method is to provide a structure that is sufficiently wide to span the 100 year erosion limit, calculated based on geomorphology principles. Table 9 presents the 100 year erosion limit widths, calculated by Water's Edge for the main watercourses.

To provide the required width of watercourse, the culvert spans will need to be increased from those listed in **Table 7** to the Minimum Culvert Span given in **Table 10.**

At some of the culverts the model shows that the watercourse profile will need be lowered to accommodate the new culverts. The extent of channel modifications will depend on the detail survey of the creek, which is not available at this time. The design of the required watercourse modifications will be undertaken during the detail design of the project.

Table 9 100 Year Erosion Limits				
Watercourse	**Bankfull Width (m)**	**3xBankfull Width (m)**	**100 yr Erosion Limit (m)**	**Min. Culvert Span (m)**
West Rainbow Creek	2.81	8.43	7.9	7.9
Rainbow Creek Tributary	1.07	3.21	2.5	2.5
East Rainbow Creek	1.47	4.41	3.1	3.1
Robinson Creek	5.24	15.72	9.3	9.3
East Robinson Creek	1.66	4.98	3.7	3.7

Table 10 Increased Culvert Spans				
Watercourse	**100 yr Erosion Limit (m)**	**Min. Culvert Span (m)**	**Hydraulic Span (m)**	**Revised Span (m)**
West Rainbow Creek	7.9	7.9	4.8	7.9
Rainbow Creek Tributary	2.5	2.5	2.7	4.8
East Rainbow Creek	3.1	3.1	6.0	5.4
Robinson Creek	9.3	9.3	8.0	9.3
East Robinson Creek	3.7	3.7	3.6	3.7

The size of the culverts would need to be increased from the sizes provided in **Table 7** to meet the span length requirements of **Table 10**. The final opening sizes are provided in **Table 11**.

Table 11 Final Culvert Sizes

Watercourse	Station	No. of Barrels	Type	Rise or Diameter (mm)	Span (mm)	Length (m)
Local Drainage	1+140	1	CSP	1200		26
West Rainbow Creek	1+330	2	Concrete Box	1800	3950	26
Rainbow Creek Tributary	1+524	1	CSP	1500		26
Rainbow Creek Tributary	2+460	1	Concrete Box	1500	2700	26
Local Drainage	2+985	2	CSP	900		26
East Rainbow Creek	3+320	1	Concrete Box	1800	3100	26
Robinson Creek	3+768	2	Concrete Box	3000	4650	26
East Robinson Creek	4+687	2	Concrete Box	1200	1850	26
Local Drainage	5+793	1	CSP	900		26
Local Drainage	5+979	1	CSP	900		26
Local Drainage	6+226	1	CSP	900		26

4.5 Watercourse Realignment

A location of particular interest is the segment of road between Stations 2+420 and 2+800, where the East Rainbow Creek watercourse is located parallel and adjacent to the existing road. At this location, it will be necessary to realign the creek to the east, in order to construct the road widening. A preliminary fluvial geomorphology report has been prepared by Water's Edge Geomorphology to address the required channel realignment design.

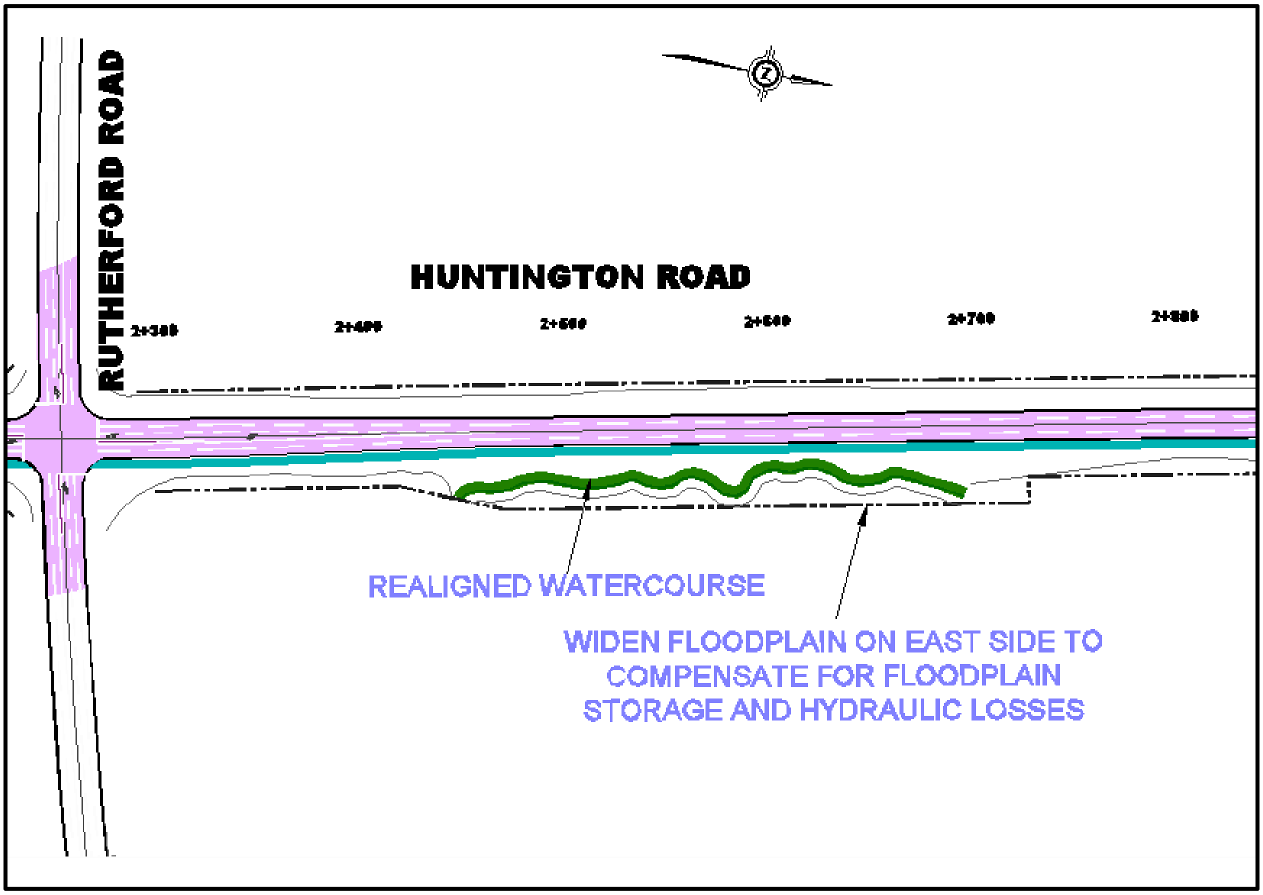


Figure 3 - Channel Realigment Sketch

Hydraulic calculations indicate that the proposed road widening will require widening of the floodplain on the east side of the valley, to accommodate the realigned watercourse, and to minimize the effect on flood levels. The results of the calculations, based on the premise that the volume of the additional road fill will be cut on the east side to compensate, show that there will be no significant change to the flood levels. In other words, the changes in water levels will be less than 0.05 m, which are generally accepted as insignificant changes, as they are less than the error intrinsic in the model. It should be noted that the exact extent of floodplain modification will need to be addressed in the final design.

The MTO Highway Drainage Design Criteria (SD-12 Freeboard Above Adjacent Watercourses or Waterbodies) requires that the road profile be set so that the top of the subgrade is 0.5m higher than the 100 year flood for watercourses that are parallel to the roadway. In the particular case o Huntington Road, meeting the MTO criteria would require a significant watercourse and valley reconstruction, plus raising the road profile substantially (more than 1.5 m above the 100 year flood level). Accordingly, we recommend that the road be raised such that the 0.5 m freeboard be measured from the road profile control.

The proposed road profile takes this requirement into account.

4.5.1 Driveway Culvert near Station 2+600

At the location of the proposed watercourse realignment, the property owner currently has a driveway that crosses the watercourse. Based on the discharges at this location, the driveway has an effect on upstream water levels during the smaller, more frequent floods. The property owner has been in conversation with the City of Vaughan and the Toronto Region Conservation Authority to relocate the driveway or add a second driveway south of the existing one. During the course of the study, Parsons and the City of Vaughan met with the property owner to coordinate the driveway with this study. However, the matter was not resolved to date.

NOTES:
1. ALL UNITS IN METERS UNLESS OTHERWISE SHOWN

SCALE 1:2500

No.	DATE	REVISIONS	BY

Sanchez Engineering Inc.

PARSONS

DESIGN	LS
DRAWN	LSC
CHECKED	LS

HUNTINGTON ROAD EA
LANGSTAFF ROAD TO MAJOR MACKENZIE DRIVE

PROPOSED CULVERTS
PLAN

DWG. NO.

CONT. NO.

SHEET NO. 01

5.0 DRAINAGE AND STORMWATER MANAGEMENT

Currently the majority of the roadway has a rural cross-section, and is drained by roadside ditches. The project will modify the road to an urban section with curb and gutter.

The effect of the proposed changes in the roadway cross-section will be to increase peak flows, but the net effect to the receiving watercourses will not be significant in terms of stormwater quantity. The significant effects will be the potential impact that the proposed cross-section changes will have on water quality.

Several stormwater management methods will be examined during the upcoming period of the study to determine methods and procedures that are necessary to mitigate the potential impacts of the road improvements on water quality.

5.1 Existing Drainage

From Station 0+200 to the south, Huntington Road is a four lane urban section road, and is drained by a storm sewer. From Station 0+200 to Station 0+620 the road drains to the south in roadside ditches. The drainage enters the storm sewer system at ditch inlets located at approximately Station 0+200.

From Station 0+620 northerly to the northern study limit, Huntington Road is a two lane rural section, with either narrow shoulders or no shoulders. The road is drained at present by roadside ditches and culverts. Visual examination of the ditches did not reveal any areas of particular drainage deficiency or erosion/scour issues. The road drainage system outlets at the major watercourses discussed in Section 4.

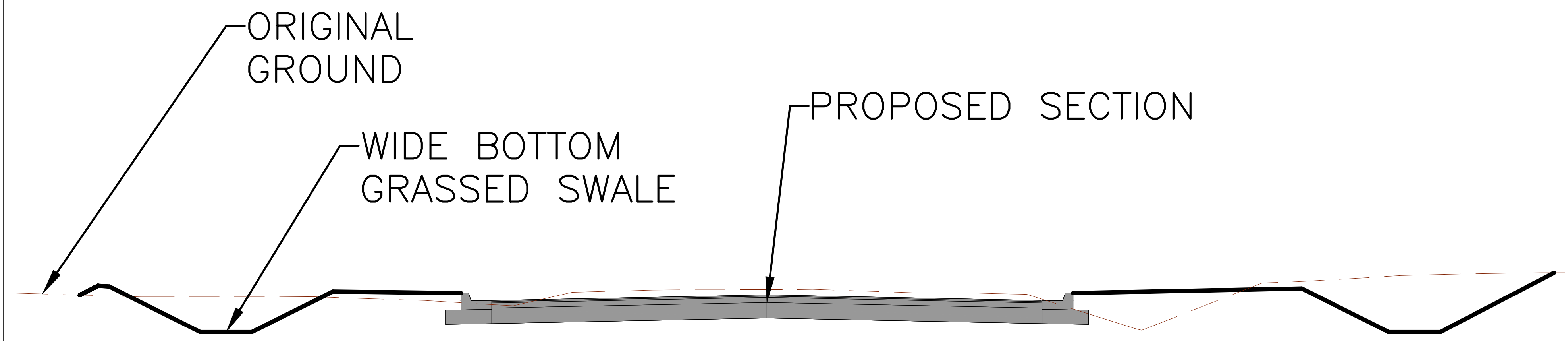
5.2 Proposed Drainage

Table 12 summarizes the change in imperviousness levels for the various sections of the road. **Figure 4** shows the proposed typical section for Huntington Road.

The proportion of the right-of-way that will be impervious is summarized under the column Imperviousness Ratio of Right-of-Way, in **Table 12.**

It is proposed to drain the roadway to outside grassed swales, rather than collecting the runoff in storm sewers for discharge to the watercourses. The proposal is to use curb and gutters to side concrete spillways, which will direct the runoff to the side swales, and then convey the runoff in the swales to the watercourses.

The nature of the underlying soils, mainly clay loam and clay till, precludes the use of many Low Impact Development solutions for this project. Please refer to Section 5.5 for a discussion and assessment of potential LID options.



Sanchez Engineering Inc.

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FIGURE 4
TYPICAL ROAD SECTION

Table 12 Right-of-Way Imperviousness Levels

Area	From Station	To Station	Receiving Tributary	Total Right-of-Way Area (ha)	Total Paved Area in Right-of-Way (ha)		Imperviousness Ratio of Right-of-Way	
					Existing	Proposed	Existing	Proposed
1	0+220	0+609		0.94	0.23	0.56	0.25	0.60
2	0+609	2+100	1, 2	3.97	0.89	2.15	0.23	0.54
3	2+100	3+420	3, 4	3.39	0.79	1.90	0.23	0.56
4	3+420	4+100	5	1.83	0.41	0.98	0.22	0.54
5	4+552	6+118	6	5.33	0.94	2.25	0.18	0.42

5.3 Stormwater Quantity

The right-of-way areas in **Table 12** constitute less than 1% of the overall receiving watercourse drainage areas, with the exception of the Area 5, which represents about 6% of the overall drainage area of Tributary 6.

Given the relatively small proportion of the total drainage areas to Huntington Road that the right-of-way represents, it can be concluded by inspection that the additional imperviousness area resulting from the road improvements will have no significant effect on the peak discharge or volume of runoff in the receiving watercourse.

In other words, the additional impervious area within the right-of-way will have no significant effect on the receiving watercourse peak flows. Consequently, it is recommended that no storm water quantity controls be implemented for this project, aside from the water quantity reductions that are incidental to the stormwater quality measures.

The calculated changes in peak flows for the various storms at the main watercourse crossings are summarized in **Table 13.**

Table 13 Peak Flows - Existing and Proposed Road Conditions

Event	Station	Peak Flow Existing Conditions m^3/s	Peak Flow Proposed Conditions m^3/s	Difference in Peak Flow Existing Conditions m^3/s	Percent Change
H. Hazel	1+524	3.9	3.9	0.00	0.00%
H. Hazel	1+330	79.5	79.5	-0.01	-0.01%
H. Hazel	2+460	10.3	10.3	0.02	0.15%
H. Hazel	3+320	43.1	43.1	0.01	0.02%
H. Hazel	3+768	116.7	116.7	0.00	0.00%
H. Hazel	4+687	7.5	7.4	-0.03	-0.37%

Table 13 Peak Flows - Existing and Proposed Road Conditions					
Event	**Station**	**Peak Flow Existing Conditions m³/s**	**Peak Flow Proposed Conditions m³/s**	**Difference in Peak Flow Existing Conditions m³/s**	**Percent Change**
2-Year	1+524	3.0	3.0	0.00	0.00%
2-Year	1+330	34.1	34.1	-0.01	-0.04%
2-Year	2+460	7.4	7.5	0.08	1.11%
2-Year	3+320	25.5	25.5	0.00	0.02%
2-Year	3+768	40.0	40.0	0.01	0.02%
2-Year	4+687	0.6	0.6	0.01	1.31%
5-Year	1+524	4.3	4.3	0.00	0.00%
5-Year	1+330	48.3	48.3	-0.07	-0.14%
5-Year	2+460	10.2	10.3	0.08	0.82%
5-Year	3+320	36.3	36.3	-0.01	-0.02%
5-Year	3+768	56.5	56.5	-0.01	-0.01%
5-Year	4+687	1.0	1.0	0.01	0.97%
10-Year	1+524	5.1	5.1	0.00	0.00%
10-Year	1+330	61.9	61.8	-0.10	-0.16%
10-Year	2+460	12.0	12.1	0.08	0.69%
10-Year	3+320	43.1	43.1	-0.01	-0.03%
10-Year	3+768	70.1	70.1	-0.06	-0.08%
10-Year	4+687	1.3	1.4	0.01	0.89%
25-Year	1+524	6.2	6.2	0.00	0.00%
25-Year	1+330	75.6	75.5	-0.14	-0.18%
25-Year	2+460	14.4	14.6	0.12	0.82%
25-Year	3+320	51.8	51.7	-0.02	-0.04%
25-Year	3+768	86.4	86.3	-0.02	-0.03%
25-Year	4+687	1.8	1.8	0.01	0.73%

Table 13 Peak Flows - Existing and Proposed Road Conditions

Event	Station	Peak Flow Existing Conditions m^3/s	Peak Flow Proposed Conditions m^3/s	Difference in Peak Flow Existing Conditions m^3/s	Percent Change
50-Year	1+524	6.9	6.9	0.00	0.00%
50-Year	1+330	85.8	85.7	-0.10	-0.12%
50-Year	2+460	16.3	16.4	0.07	0.42%
50-Year	3+320	58.6	58.6	-0.03	-0.05%
50-Year	3+768	98.4	98.4	-0.03	-0.03%
50-Year	4+687	2.1	2.2	0.01	0.65%
100-Year	1+524	7.7	7.7	0.00	0.00%
100-Year	1+330	96.3	96.2	-0.12	-0.12%
100-Year	2+460	18.1	18.2	0.07	0.36%
100-Year	3+320	67.7	67.7	0.03	0.05%
100-Year	3+768	115.0	114.9	-0.04	-0.04%
100-Year	4+687	2.5	2.5	0.02	0.72%

Unit flow relationships were defined by TRCA in the Stormwater Management Criteria, 2012 to provide flow control targets for development within the watershed. The relationships for the Humber River watershed are provided below:

Equation F Sub-Basin 36	Where Q = Target flow in litres per second A = Drainage Area in ha.
Q=29.912-2.316 *ln(A)	
Q=26.566-2.082 *ln(A)	
Q=22.639-1.741 *ln(A)	
Q=17.957-1.373 *ln(A)	
Q=14.652-1.136 *ln(A)	
Q=9.506-0.719 *ln(A)	

The calculated Unit Flows for the five areas described above are provided in **Table 14**. The corresponding Allowable Outflows are listed in **Table 15.**

Table 14 Unit Flows						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Area 1	9.6	14.6	18.0	22.7	26.7	30.1
Area 2	8.5	13.0	16.1	20.2	23.7	26.7
Area 3	8.6	13.2	16.3	20.5	24.0	27.1
Area 4	9.1	13.9	17.1	21.6	25.3	28.5
Area 5	8.3	12.7	15.7	19.7	23.1	26.0

Table 15 Allowable Outflows						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Area 1	9	14	17	21	25	28
Area 2	34	52	64	80	94	106
Area 3	29	45	55	70	81	92
Area 4	17	25	31	40	46	52
Area 5	44	67	83	105	123	139

The calculated outflow from the five areas are presented in **Table 16**, based on the runoff coefficients assuming that no stormwater quantity controls have been implemented. As will be shown, the inclusion of Low Impact Development measures (See Section 5.6) are capable of reducing the volume of runoff sufficiently to effectively lower the runoff coefficient to eliminate the need to provide water quantity control measures.

Table 16 Calculated Outflows without LID						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Area 1	59	78	90	105	117	128
Area 2	172	227	262	308	342	375
Area 3	128	169	196	230	255	280
Area 4	93	123	143	168	186	204
Area 5	158	208	241	283	314	345

5.4 Stormwater Quality

The objective of stormwater management is to protect the receiving watercourses against potential impacts produced by changes in the hydrologic conditions of the tributary area; these impacts include larger flows resulting from changes in surfaces from grass and vegetation to paved areas, and water quality degradation from construction activities and from long term operation of the roads after they are widened and provided with sidewalks and other hard, impervious surfaces.

Typical storm water management alternatives for road construction projects address the water quality impacts associated with the modified land use as well as any quantity control requirements deemed necessary on the basis of the receiving water bodies and local site constraints and considerations. The site-specific storm water management requirements are based on the issues of concern associated with each watercourse and its natural and hydrological features.

5.5 Water Quality Protection Options

Storm water from roadways is known to create non-point source water pollution that is detrimental to the water quality of receiving watercourses. Vehicular-related pollution, such as oil and grease, heavy metals, nutrients and sediments, is directly related to traffic volume. In addition, highways tend to serve as a conduit for pollutants from rural areas draining to them, such as runoff to the right-of-way from agricultural or landscaped areas which transports sediment, pesticides and fertilizers; and for the particulates generated with de-icing sand applications, and as pavement breaks down with use.

Low Impact Development (LID) is a method of stormwater management that attempts to replicate in the post-development environment the pre-development hydrologic regime. This is accomplished by reducing the runoff volume, peak discharge, and associated pollutant loads near the source of runoff, using techniques that intercept and hold runoff. LID aims at using vegetation and infiltration to reduce the runoff volumes and increase the time of travel of runoff. A further objective of LID stormwater management philosophy is to provide a train of treatment measures, such that the highway storm runoff must flow through two or three measures before discharging to the receiving watercourse.

LID was examined for this project to determine the feasibility of implementing one or several LID techniques. One of the advantages of LID is that it treats runoff at, or as closely as possible to, its source.

LID storm water management techniques that are consistent with transportation type construction projects are listed below and a brief description is provided in the subsequent sections.

- Bioretention
- Bio-Slopes
- Catch Basin Controls
- Gutter Filter
- Infiltration Trenches/Strips
- Permeable Pavement
- Surface Sand Filter
- Enhanced Grassed Swales
- Vegetation/Landscaping

Pollution Prevention and Street Sweeping are measures for reduction of the pollutant loading reaching the storm water runoff. They are overall management measures for pollution abatement, which the municipality exercises as part of the City’s normal operation.

In cases where the flows are high, then "end-of-pipe" measures - such as detention ponds, both dry and wet ponds, or oil/grit separators - can be considered if it is not fully possible to implement a combination of the LID storm water management measures. The following sections describe all LID and end-of-pipe measures that were considered for this project.

5.5.1 Bioretention

Bioretention cells consist of vegetated depressions used to filter runoff rapidly using bio-retention soil media. The soil media include mulch and soil. Runoff is stored in the bio-retention cell until it infiltrates into native soils. Bioretention cells help to reduce peak discharges by the temporary storage, and to improve storm runoff water quality by filtration. **Figure 5,** extracted from the LID Design Manual published by the U.S. National Cooperative Highway Research Program (NCHRP), illustrates the concept.

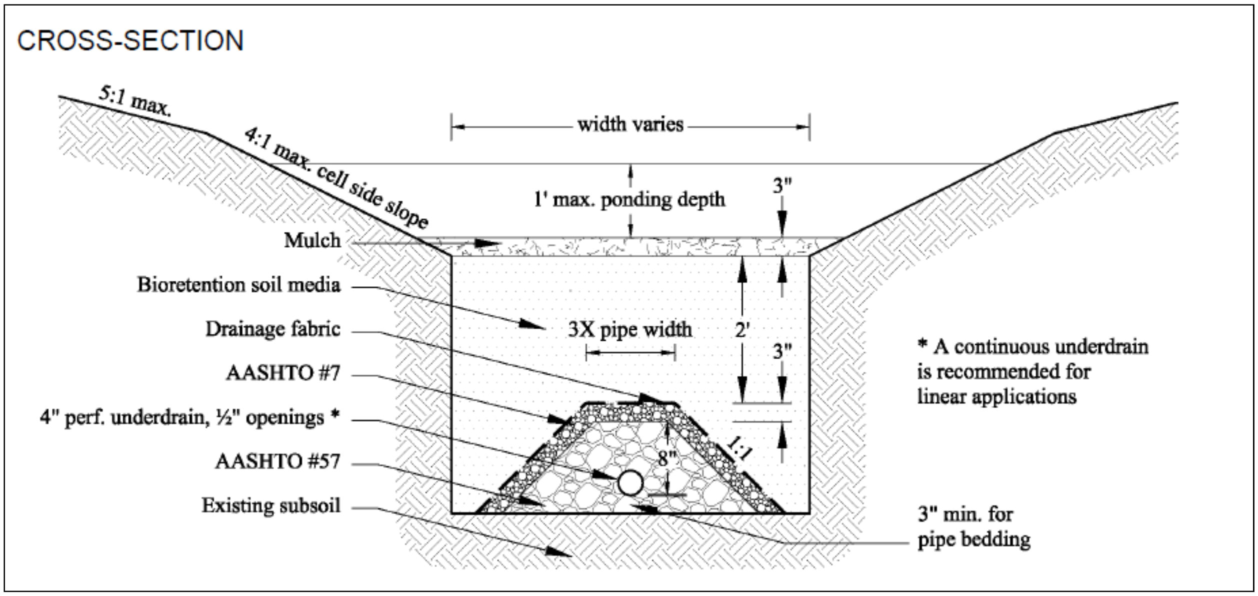


Figure 5 - Bio-retention Cell

5.5.2 Bio-Slopes

Bio-slopes are built within the roadway embankment to provide rapid filtration of the runoff as it leaves the roadway shoulder. The storm runoff is intercepted by a gravel trench, and is filtered through a mix of pea gravel, perlite, dolomite and gypsum. The runoff is not detained and there is no effect on water quantity. However, the filter provides water quality improvement. **Figure 6**, from the same NCHRP publication, illustrates the concept.

In this project, bio-slopes cannot be used because the road will have an urban section.

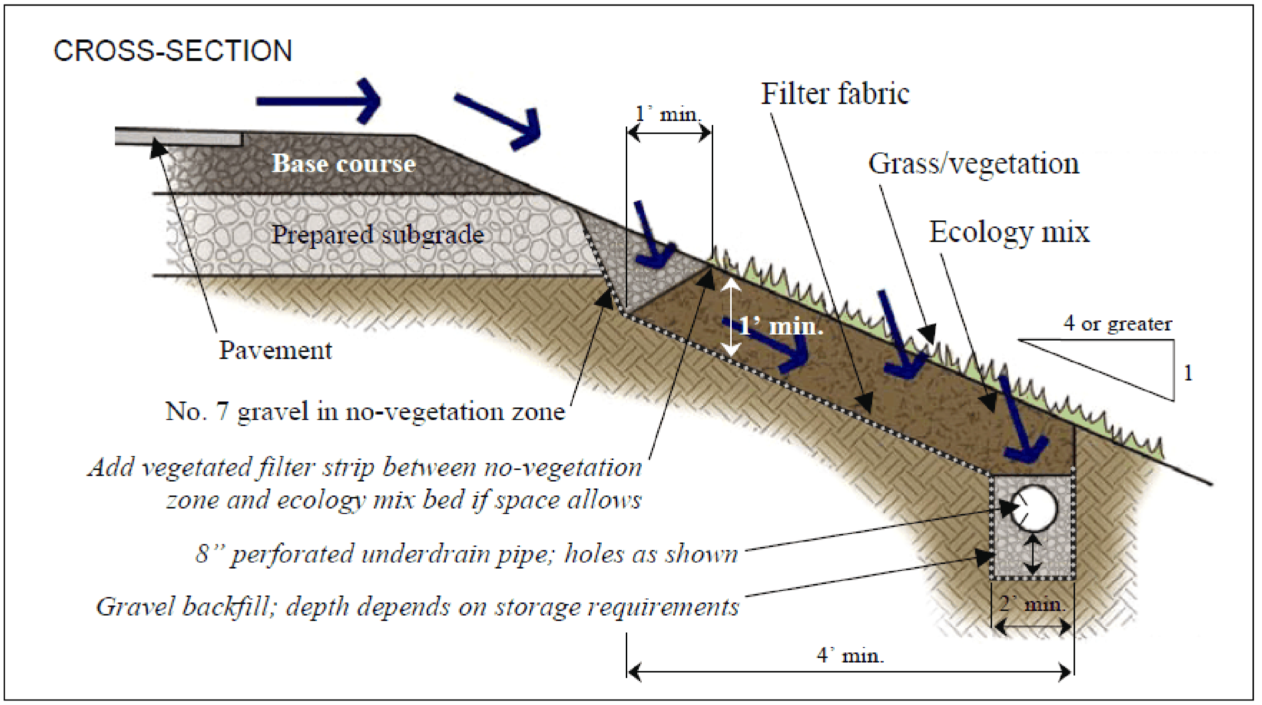
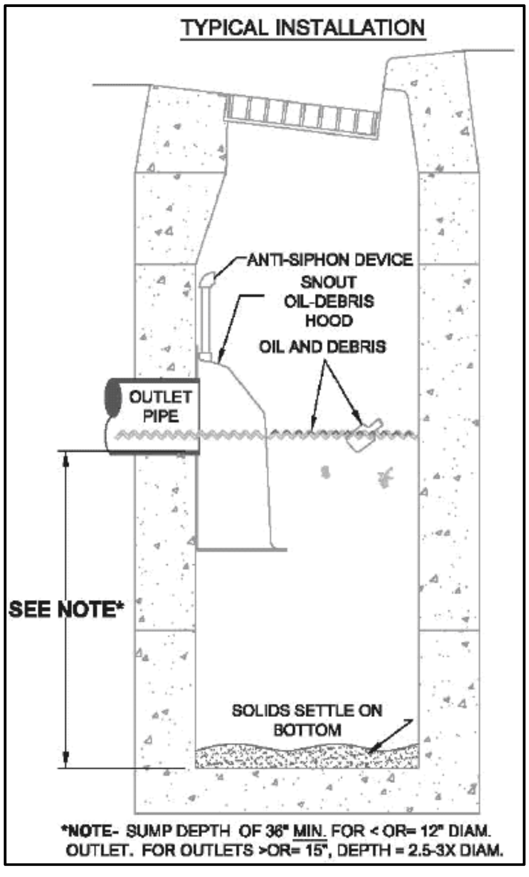


Figure 6 - Bio-Slope



5.5.3 Catch Basin Controls

Catch basin controls are devices to prevent floatables from entering the storm sewer. The catch basin controls can be baffles, covers, geotextile fabrics, and other similar objects place in the catch basins. Their main effectiveness is in reducing the possibility of floatables from arriving at the receiving watercourse. These types of catch basins are maintained in the same manner and frequency as normal catch basins. An illustration from the same Design Manual by the NCHRP follows in **Figure 7**.

Figure 7 - Catch Basin Controls

5.5.4 Gutter Filter

Gutter filters consist of a concrete gutter with a grate cover. The concrete gutter is filled with sand or sand and gravel. Storm runoff is filtered as it flows through the granular media. **Figure 8** from the previous source illustrates the concept.

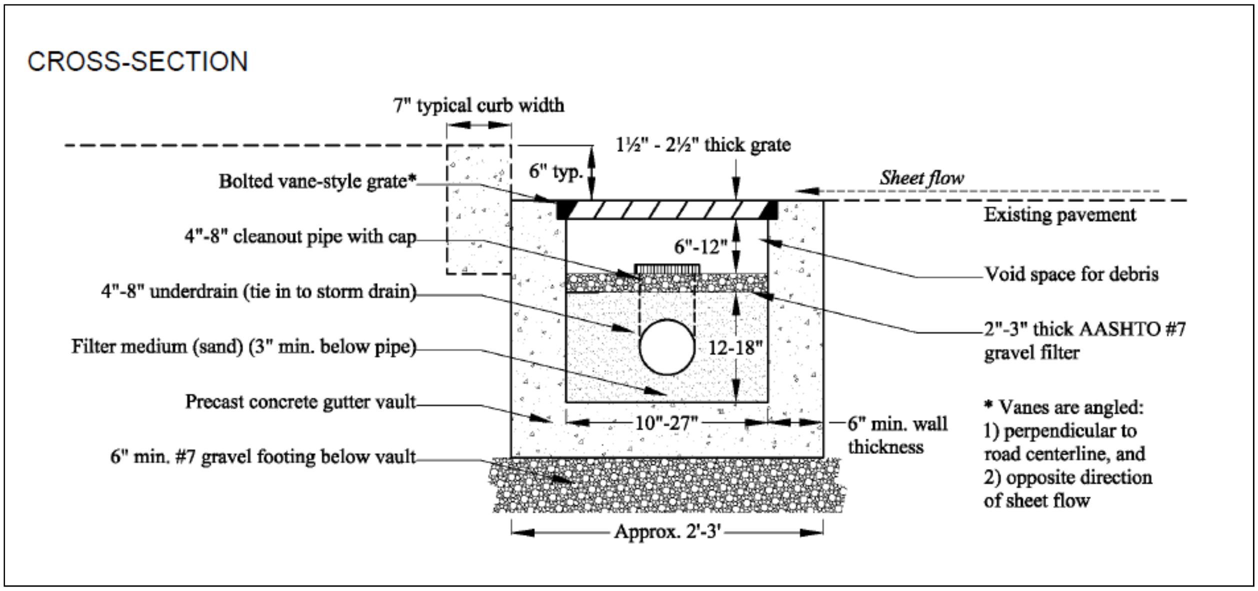


Figure 8 - Gutter Filter

5.5.5 Infiltration Trenches and Strips

Infiltration trenches and strips consist of an excavated trench lined with geotextile and backfilled with stone, as illustrated in **Figure 9**. The purpose of the system is to promote infiltration of storm water into the subsurface soils.

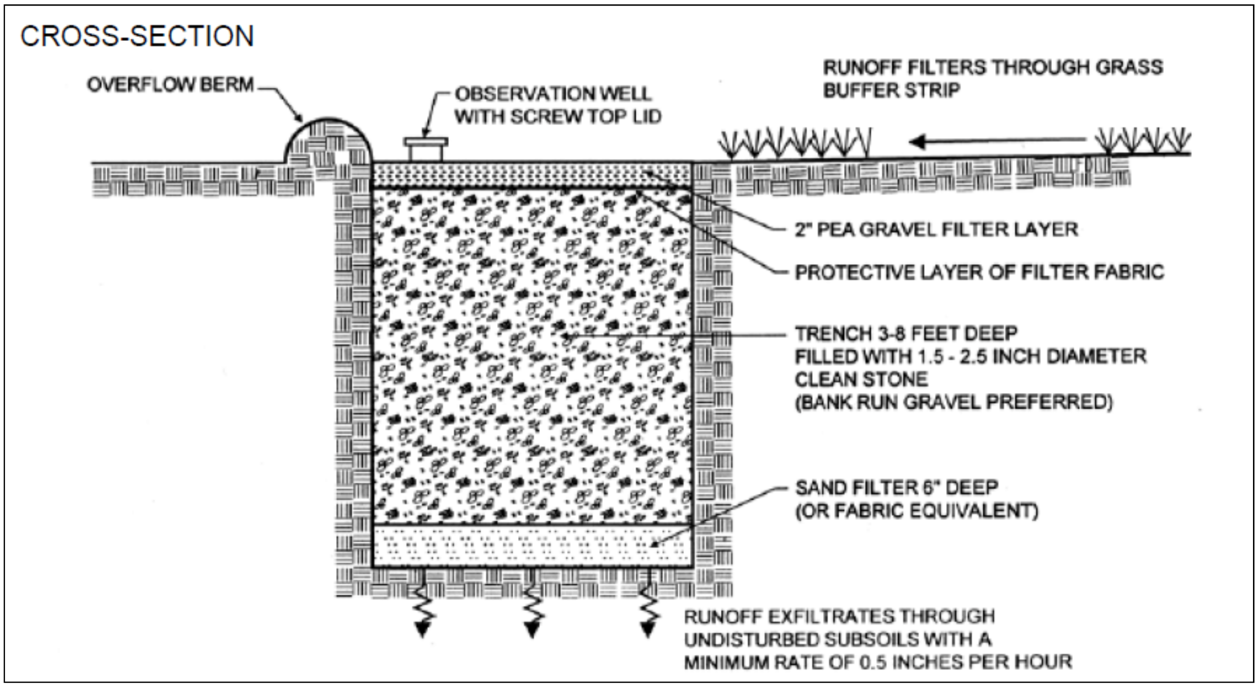


Figure 9 - Infiltration Trench

5.5.6 Surface Sand Filter

A sand filter consists of a sedimentation and a filtration chamber. The filtration chamber is filled with sand, which drains to an underdrain pipe. In general terms, the sand filter follows the same principle as the slow sand filter for water quality purification.

Sand filters can be used in cold climates, with the provision of deeper sand filter beds. In essence, for them to work in the winter they have to be buried, so in fact they become an infiltration trench.

5.5.7 Grassed Swales

The use of grass swales as the primary storm water management practice is appropriate for all levels of protection, where the following conditions are met:

- The grassed swale is at least 80 m long;
- The maximum flow in the grassed swale for the design storm is less than 150 l/s; and
- The maximum flow velocity is less than 0.5 m/s.

Under these conditions, it can be expected that the grassed swale will provide greater than 80% removal of total suspended solids. For example, the Storm Water Technology Fact Sheet, USA EPA, 1999, indicates that the median percent removal is 81%. The Article in Water titled *Pollutant Removal and Hydraulic Reduction Performance of Field Grassed Swales during Runoff Simulation Experiments, Water 2014* indicates that in well designed and properly maintained swales the TSS removal rates can be as high as 99%.

Grassed swales can be used for all levels of protection, where wetlands, wet ponds or sand filters cannot reasonably be utilized because of physical, engineering, property, environmental, or cost considerations; or for levels 3 and 4 protection, where MTO would have to acquire additional property in order to reasonably accommodate infiltration, wetlands, wet ponds or dry ponds.

Water quality treatment with grassed swales is based on the flow velocity in the swale being less than or equal to 0.5 m/s, a maximum flow rate of 150 l/s, and a maximum depth of flow of approximately 0.25 m. In addition, vegetation should be allowed to grow higher than 75 mm to enhance the filtration of suspended solids. **Figure 10**, from the same NCHRP publication, illustrates the concept.

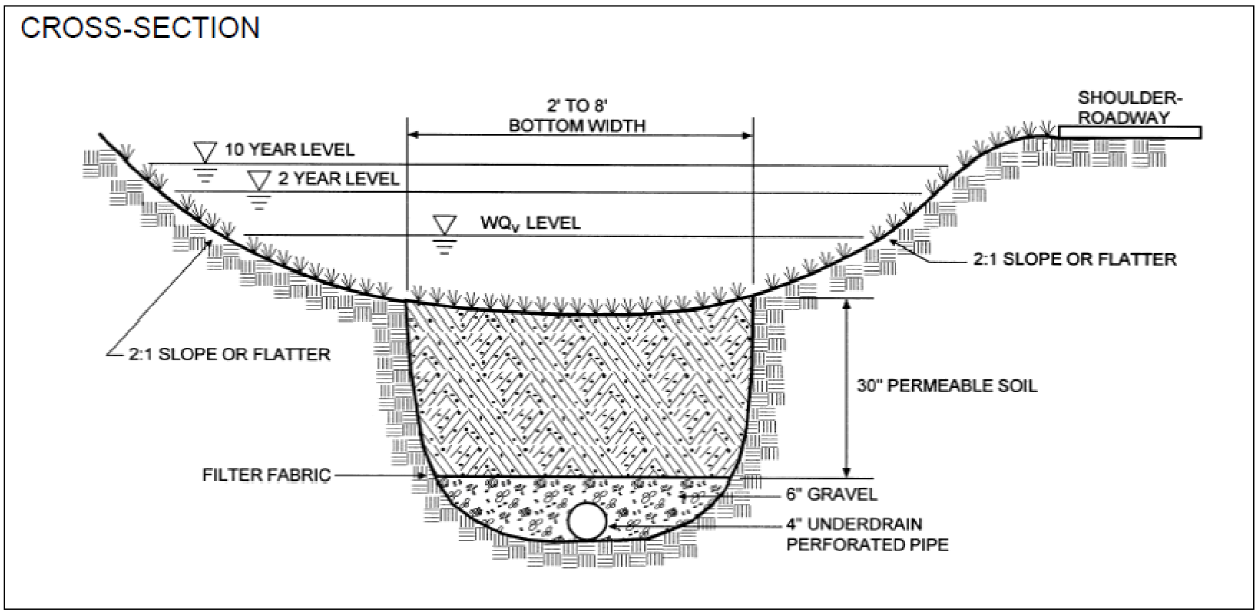


Figure 10 - Grassed Swale

5.5.8 Enhanced Grassed Swales

Wide flat bottoms can be used to improve the water quality performance of grassed swales, thus permitting treatment of a greater overall area. The wide bottom helps to reduce the flow depth and velocity, consequently assisting in the settlement of suspended particles. In cases where the flow velocities cannot be reduced to the desired values, permanent rock flow checks along the swale can be used to promote settling. Flow checks reduce the effective slope of the swale where the slope is too steep to allow the maximum design velocity (0.5 m/s) to be achieved. The ponding behind the flow checks also provides treatment for a larger flow depth or flowrate than would be possible with a standard grassed swale. The values of flow rate, flow velocity, and depth used for grassed swales apply to enhanced grass swales.

5.5.9 Filter Strips

Filter strips are vegetated areas designed to accept runoff in the form of sheet flow. The vegetation filters out sediment and pollutants and promotes infiltration. Filter strips are suitable for small drainage areas (less than 2 ha). The filter strip can have a slope of up to 10% and the flow length can range from 10 to 20 m depending on the slope. The vegetated side slopes of a road function as a filter strip for sheet flow from the highway.

On this project there are no opportunities to use filter strips for stormwater quality.

5.5.10 Extended Detention Dry Ponds

An extended detention dry pond detains runoff during a storm event for approximately 24 hours. Water quality treatment is provided by sedimentation while the runoff is detained in the pond.

A minimum drainage area of 5 ha is generally required in order to provide an outlet orifice of sufficient size to minimize clogging. The length to width ratio should be in the order of 3:1 to 5:1 and the inlet and outlet should be at opposite ends of the facility.

Extended detention dry ponds that operate in a continuous mode are not as effective as extended detention wet ponds in removing storm water pollutants. Generally, dry ponds should only be used when wet ponds or wetlands cannot be implemented due to constraints such as temperature and land availability. Dry ponds are included in the MOEE (2003) Design manual only for aquatic habitat protection levels 3 and 4.

Given that storm water quantity is not a requirement, and that other storm water management measures can be applied to achieve the required storm water quality objectives, extended detention dry ponds are not attractive, since they require additional property; furthermore, their relatively low effectiveness in providing storm water quality improvements, do not make them practical for this project.

5.5.11 Wet Ponds

Wet ponds are designed with a permanent pool of water, which provides long term sedimentation, and storage for extended detention during runoff events. Wet ponds provide water quality treatment by sedimentation during a storm, plus additional sedimentation for smaller sediment particles is attained by detention in the permanent pool.

A minimum drainage area of 5 ha is generally required in order to provide an outlet orifice of sufficient size to minimize clogging and to provide sufficient groundwater inflow to maintain the permanent pool. The length to width ratio should be in the order of 3:1 to 5:1 and the inlet and outlet should be at opposite ends of the facility. In addition, a sediment forebay needs to be provided to settle larger size particles before they enter the permanent pool.

Wet ponds are not a feasible solution in this project because of the small drainage areas that would required storage. Therefore, they will not be considered further.

5.5.12 Oil/Grit Separators

Oil/grit separators (OGS) consist of underground detention chambers designed to trap and retain oil and sediment. The devices use sedimentation for suspended solids and phase separation to trap oil. Currently several manufacturers offer a variety of designs, which provide a relatively wide range of treatment of runoff.

It is generally accepted that OGS are effective for drainage areas smaller than 2 ha, since OGS provide very little flow attenuation. They are designed to provide treatment for relatively frequent runoff events, and by-pass larger flows. Their effectiveness stems from treating runoff events that occur with regularity, but cannot treat design storm level flows.

Notwithstanding the 2 ha traditional drainage limit, newly developed OGS by some manufacturers permit treatment of stormwater quality for much larger drainage areas.

5.6 Water Quality Treatment Selection

In reviewing the range of stormwater quality treatment options in light of the constraints imposed by the site, it can be concluded that the following options could be applied on this project, given the site drainage and soils constraints:

a. Bioretention
b. Catch basin controls
c. Enhanced Grassed swales
d. Oil/grit separators

In the next stage of the design of the road improvements, stormwater quality treatment should include a combination of these options. In particular, it is recommended that the road be drained to grassed swales as per **Figure 4**. This will reduce the need to provide a storm sewer for most of the road length.

5.6.1 Bioretention

Bioretention would require provision of approximately 79 bioretention units, to treat the flow from approximately 15.5 ha of right-of-way drainage. Each bioretention unit will be 4 m wide, 33 m long and 2.5 m deep. The unit cost per bioretention unit is $47,000.

The estimated cost of providing bioretention for the entire project is $3.7 Million.

Using bio-retention to provide stormwater quality control will have the beneficial effect of reducing the volume of runoff generated by the site in the proposed conditions. The 2010 Low Impact Development Stormwater Management Planning and Design Guide indicates that bioretention without underdrain can provide percent runoff reduction of 85%. In effect, the reduction in runoff reduces the runoff coefficient for the tributary area to 15% of the untreated surface. With underdrain, the reduction is about 45%. The resulting outflows are presented in **Table 17**.

Table 17 Calculated Outflows with Bioretention

	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Area 1	20	26	30	35	39	43
Area 2	50	66	76	90	99	109
Area 3	46	60	70	82	91	100
Area 4	30	40	46	54	60	66
Area 5	62	82	95	112	124	136

Further reduction in outflows can be achieved by the use of enhanced grassed swales as discussed in Section 5.6.3.

5.6.2 Catch Basin Controls

Catch basins can be used as pre-treatment prior to discharge to the grassed swales. The catch basins in combination with the inserts will be able to remove a portion of the gross debris carried by storm runoff.

5.6.3 Enhanced Grassed Swales

This study recommends that the road be drained to grassed swales along both sides of the road, rather than using storm sewers and detention ponds. To achieve this, it is proposed that grassed swales in accordance with the typical section in Figure 4 be included throughout the entire project.

On this basis, the typical grass swale will be cost in total $1.5 Million, based on the typical section shown in **Figure 10.**

Using enhanced grassed swales to provide stormwater quality control will have the beneficial effect of reducing the volume of runoff generated by the site in the proposed conditions. The 2010 Low Impact Development Stormwater Management Planning and Design Guide indicates that enhanced grassed swales can provide percent runoff reduction of 20%. In effect, the reduction in runoff reduces the runoff coefficient for the tributary area to 80% of the untreated surface. The resulting outflows are presented in **Table 18**.

Table 18 Calculated Outflows with Bioretention and Grassed Swales

	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Area 1	17	22	26	30	33	37
Area 2	42	55	63	74	83	91
Area 3	39	52	60	70	78	86
Area 4	26	34	39	46	51	56
Area 5	54	72	83	98	108	119

Comparison of the calculated outflows presented in **Table 18** with the Allowable Outflow values in **Table 15** show that they are essentially equivalent. Additional reduction could be achieved by oversizing the bioretention facilities beyond the dimension strictly required for water quality control. See **Appendix A** for the calculations.

In addition to the above, the City of Vaughan plans to treat portions of the runoff produced by Huntington Road south of Rutherford Road at to two proposed stormwater management ponds that will be located on the west side of Huntington Road, between Rutherford Road and Trade Valley Drive. The details of this proposal are contained in the 2005 Stormwater Management Plan by Schaeffers Consulting Engineers.

6.0 EROSION AND SEDIMENT CONTROL

Erosion and sediment control during the construction stage will be governed by the Guidelines on Erosion and Sediment Control for Urban Construction Sites, 1987 and OPSS 805 – Construction Specification for Temporary Erosion and Sediment Control Measures, and will comprise the following minimum measures:

- The site will be enclosed with silt fence before construction starts.
- Only areas strictly required to proceed with construction will be stripped. These areas will be stabilized as soon as practical.
- Any disturbed area will be stabilized as soon as practical, especially swales and ditches.
- Areas stripped of vegetation will be surrounded by silt fence.
- A vegetated buffer will be maintained between disturbed areas and neighbouring properties where practical.
- Drainage ditches and swales will be provided with rock flow check dams or silt soxx, which will be properly installed and anchored in accordance with Ontario Provincial Standards.
- Site access will be covered with clear stone and/or rip rap to reduce tracking of mud by truck tires.

7.0 ESTIMATED COSTS

The estimated costs of the drainage and stormwater management systems are summarized in Table 19.

Table 19 Estimated Cost of Drainage and Hydrology Items

Item	Unit	Quantity	Unit Price	Cost
4.8m Span Creek Crossing (Part A)	m	26	$5,000.00	$130,000
6.0m Span Creek Crossing (Part A)	m	26	$8,000.00	$208,000
9.3m Span Creek Crossing (Part A)	m	26	$20,000.00	$520,000
7.9m Span Creek Crossing (Part A)	m	26	$10,000.00	$260,000
3.7 m Span Creek Crossing (Part B)	m	26	$5,000.00	$130,000
900 Dia. CSP (Part A)	m	52	$1,600.00	$83,200
900 Dia. CSP (Part B)	m	78	$1,600.00	$124,800
1200 Dia. CSP Culvert (Part A)	m	26	$1,870.00	$48,620
Creek Realignment	m	320	$2,500.00	$800,000
Bioretention Units	ea	79	$47,000.00	$3,700,000
Enhanced Grassed Swales	m	12,000	$125.00	$1,500,000

8.0 CONCLUSIONS

Based on the Drainage and Hydrolgy study of Huntington Road it is possible to conclude that:

1. All proposed new development and re-development within the Study Area will require individual stormwater management systems, to address both stormwater quantity and quality, in accordance with previous master drainage reports and stormwater management reports.
2. Stormwater quantity controls are not required for the proposed improvements because the road improvements will not change significantly the peak flows in the watercourses crossing Huntington Road.
3. Stormwater quality controls can be achieved by provision of bio-retention, catch basin controls, grassed swales, and oil/grit separators.
4. The main feature for storm water quality is the provision of two swales on the outside of the road allowance, in lieu of provision of a storm sewer system.
5. Huntington Road is crossed by 11 watercourses, of which five can be considered large watercourse crossings. The remaining six crossings provide for local drainage of the upper portions of larger watercourse drainage (i.e. they are part of the headwaters of the larger tributaries).
6. All existing culverts will need to be replaced to meet the Design Criteria for watercourse crossings.
7. Due to geometric design restrictions, the freeboard stipulated in the CHBDC cannot be provided.
8. The watercourse adjacent to Huntington Road north of Rutherford Road needs to be realigned to accommodated the widened road platform. Consequently, the overall floodplain needs to be widened to the east to compensate for the loss of storage and flow capacity that results from the road widening.

9.0 RECOMMENDATIONS

Based on the stormwater management study presented in this report, it is recommended that:

1. The City of Vaughan continues to require all redevelopments in the study area to provide on-site stormwater quantity and quality management.
2. The storm runoff produced by the road improvements to Huntington Road be treated using a combination of bio-retention, catch basin controls, grassed swales, and oil/grit separators.
3. The existing culverts be replaced by larger culverts, as noted in Table 7.
4. The watercourse be realigned as recommended in the Waters Edge fluvial geomorphology report, and that the floodplain be excavated on the east side to compensate for conveyance and flood storage losses.

APPENDIX A
Hydrologic Model Results

City of Vaughan

Table 4.1 Rainfall Intensity

Return Frequency	Intensity
2 years	$I = 647.7 \ (T+4.0)^{-0.784}$ mm/hr
5 years	$I = 929.6 \ (T+4.0)^{-0.798}$ mm/hr
10 years	$I = 1021 \ (T+3.0)^{-0.787}$ mm/hr
25 years	$I = 1100 \ (T+2.0)^{-0.776}$ mm/hr
50 years	$I = 1488 \ (T+3.0)^{-0.803}$ mm/hr
100 years	$I = 1770 \ (T+4.0)^{-0.820}$ mm/hr

Where T = time of concentration in minutes

MTO IDF Curve

$I = AT^B$ mm/hr

T time in hours

	A	B
2 year	21.8	-0.699
5 year	28.8	-0.699
10 year	33.3	-0.699
25 year	39.1	-0.699
50 year	43.4	-0.699
100 year	47.6	-0.699

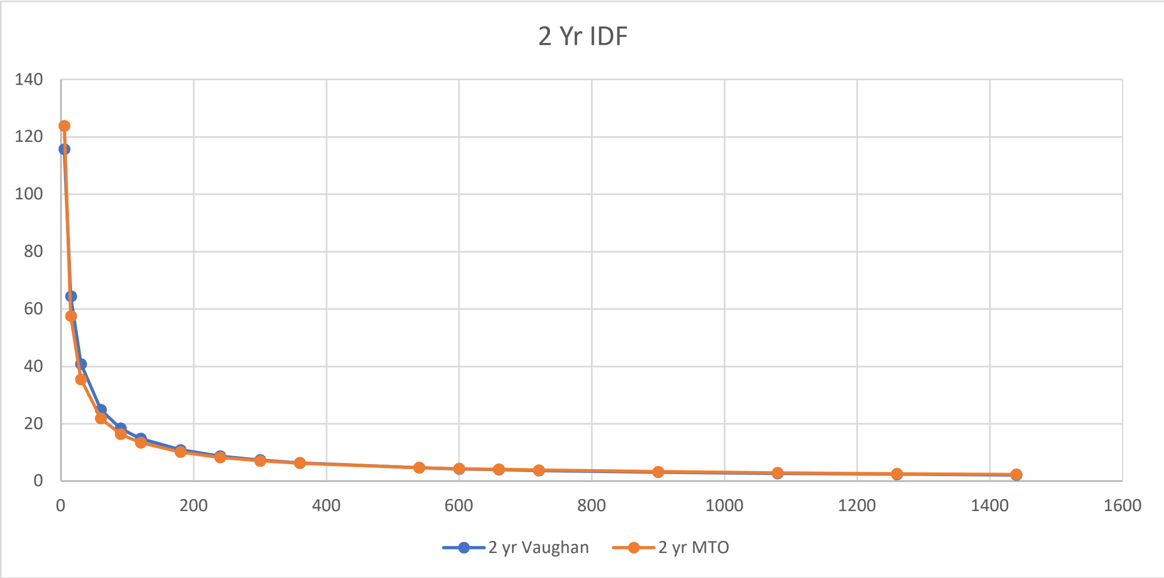
City of Vaughan

Tc (min)	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
5	116	161	199	243	280	292
15	64	89	105	122	146	158
30	41	56	65	75	90	98
60	25	34	39	45	53	58
90	18	25	29	33	39	43
120	15	20	23	26	31	34
180	11	14	17	19	23	25
240	9	12	14	16	18	20
300	7	10	11	13	15	16
360	6	8	10	11	13	14
540	5	6	7	8	9	10
600	4	6	7	8	9	9
660	4	5	6	7	8	9
720	4	5	6	7	8	8
900	3	4	5	6	6	7
1080	3	4	4	5	5	6
1260	2	3	4	4	5	5
1440	2	3	3	4	4	5

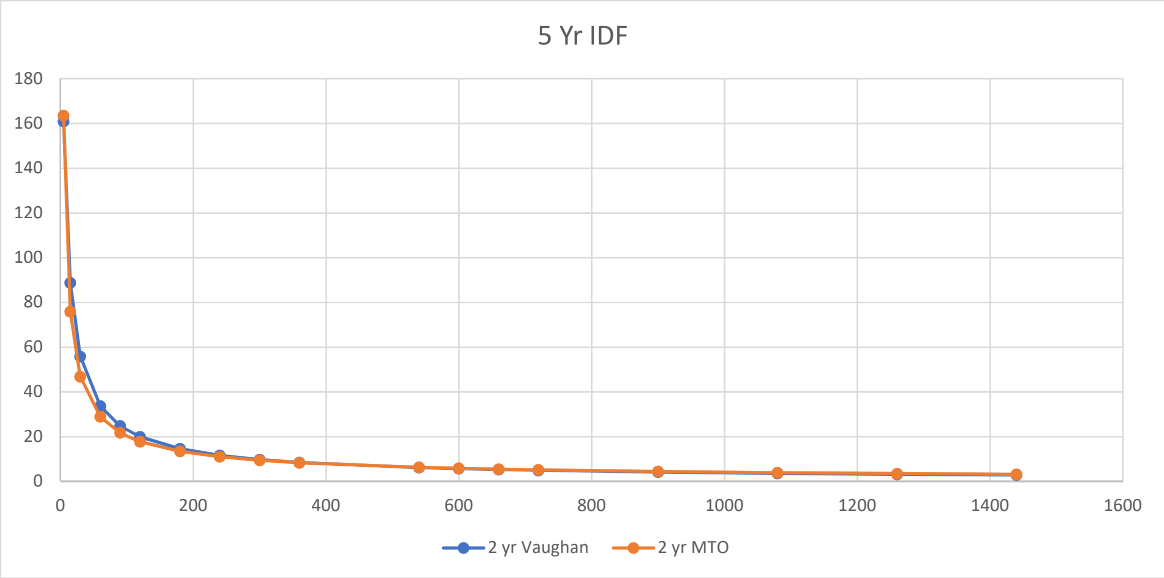
MTO IDF Curve

Tc (min)	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
5	124	164	189	222	247	270
15	57	76	88	103	114	125
30	35	47	54	63	70	77
60	22	29	33	39	43	48
90	16	22	25	29	33	36
120	13	18	21	24	27	29
180	10	13	15	18	20	22
240	8	11	13	15	16	18
300	7	9	11	13	14	15
360	6	8	10	11	12	14
540	5	6	7	8	9	10
600	4	6	7	8	9	10
660	4	5	6	7	8	9
720	4	5	6	7	8	8
900	3	4	5	6	7	7
1080	3	4	4	5	6	6
1260	3	3	4	5	5	6
1440	2	3	4	4	5	5

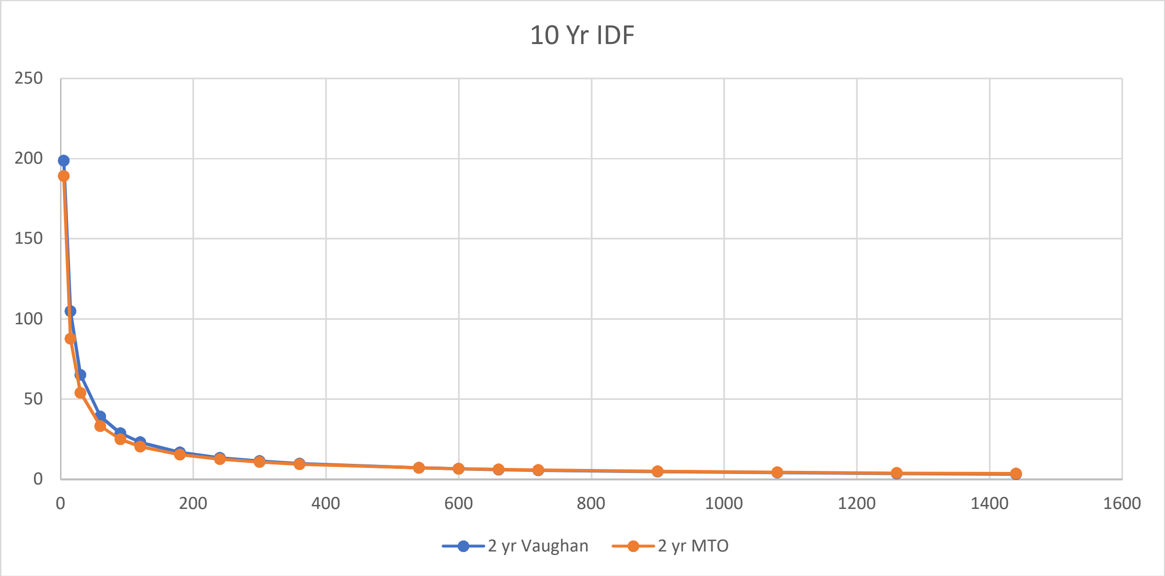
2 yr			
Tc (min)	Vaughan	MTO	% diff
5	116	124	-7.0%
15	64	57	10.8%
30	41	35	13.3%
60	25	22	12.3%
90	18	16	10.7%
120	15	13	9.2%
180	11	10	6.8%
240	9	8	4.9%
300	7	7	3.4%
360	6	6	2.0%
540	5	5	-1.1%
600	4	4	-2.0%
660	4	4	-2.7%
720	4	4	-3.5%
900	3	3	-5.3%
1080	3	3	-6.9%
1260	2	3	-8.3%
1440	2	2	-9.5%



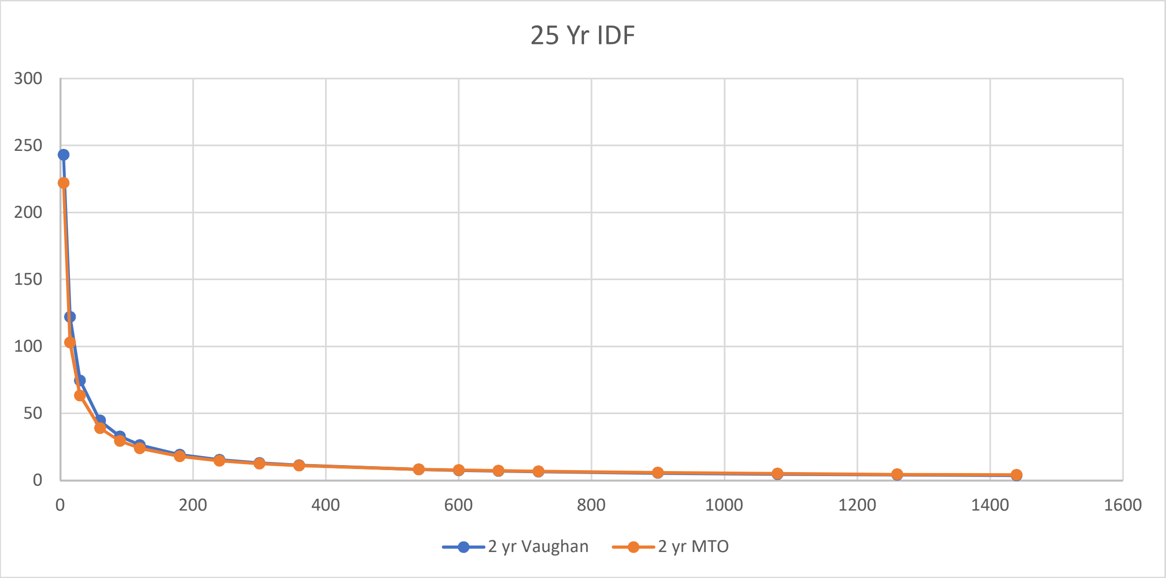
5 yr			
Tc (min)	Vaughan	MTO	% diff
5	161	164	-1.6%
15	89	76	14.4%
30	56	47	16.1%
60	34	29	14.4%
90	25	22	12.4%
120	20	18	10.6%
180	14	13	7.8%
240	12	11	5.5%
300	10	9	3.7%
360	8	8	2.1%
540	6	6	-1.6%
600	6	6	-2.7%
660	5	5	-3.6%
720	5	5	-4.4%
900	4	4	-6.7%
1080	4	4	-8.5%
1260	3	3	-10.2%
1440	3	3	-11.6%



10 yr			
Tc (min)	Vaughan	MTO	% diff
5	199	189	4.8%
15	105	88	16.4%
30	65	54	17.0%
60	39	33	15.0%
90	29	25	13.0%
120	23	21	11.3%
180	17	15	8.7%
240	14	13	6.7%
300	11	11	5.0%
360	10	10	3.6%
540	7	7	0.3%
600	7	7	-0.6%
660	6	6	-1.4%
720	6	6	-2.1%
900	5	5	-4.1%
1080	4	4	-5.7%
1260	4	4	-7.1%
1440	3	4	-8.4%

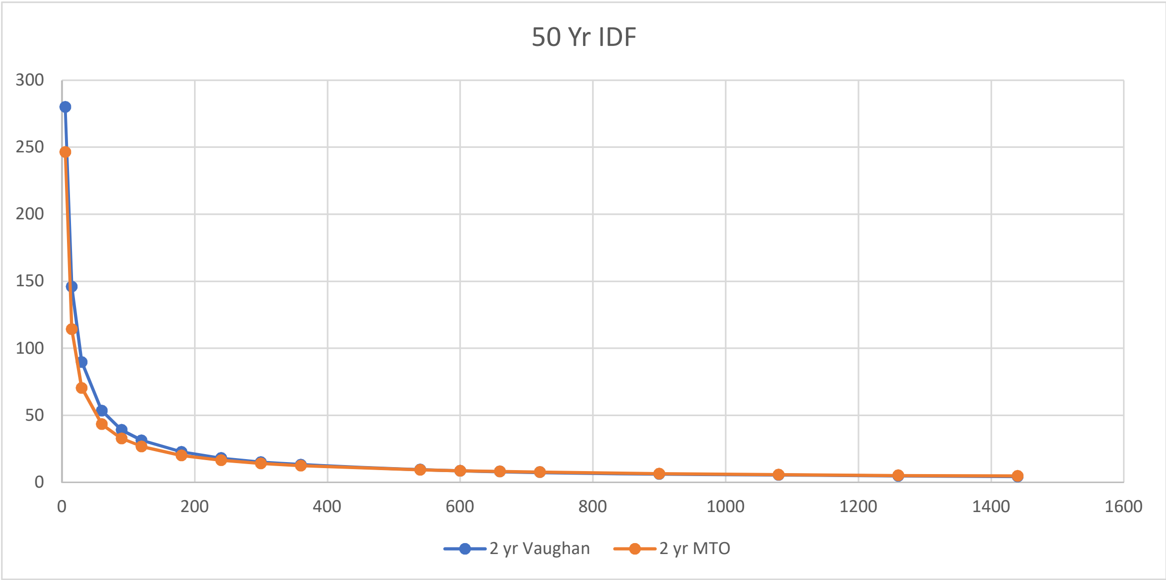


25 yr			
Tc (min)	Vaughan	MTO	% diff
5	243	222	8.6%
15	122	103	15.6%
30	75	63	15.0%
60	45	39	12.6%
90	33	29	10.5%
120	26	24	8.9%
180	19	18	6.4%
240	16	15	4.5%
300	13	13	3.0%
360	11	11	1.7%
540	8	8	-1.2%
600	8	8	-2.0%
660	7	7	-2.8%
720	7	7	-3.4%
900	6	6	-5.2%
1080	5	5	-6.6%
1260	4	5	-7.9%
1440	4	4	-9.0%



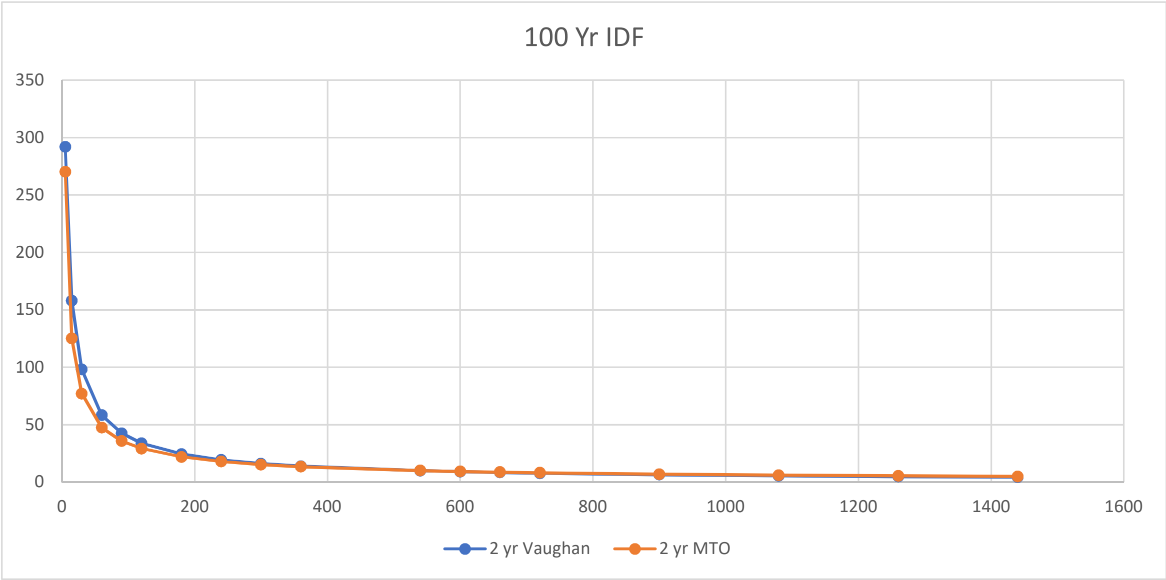
50 yr

Tc (min)	Vaughan	MTO	% diff
5	280	247	12.0%
15	146	114	21.7%
30	90	70	21.5%
60	53	43	18.8%
90	39	33	16.3%
120	31	27	14.4%
180	23	20	11.3%
240	18	16	8.9%
300	15	14	6.9%
360	13	12	5.3%
540	9	9	1.4%
600	9	9	0.3%
660	8	8	-0.6%
720	8	8	-1.5%
900	6	7	-3.8%
1080	5	6	-5.7%
1260	5	5	-7.4%
1440	4	5	-8.9%



100 yr

Tc (min)	Vaughan	MTO	% diff
5	292	270	7.4%
15	158	125	20.7%
30	98	77	21.3%
60	58	48	18.6%
90	43	36	16.0%
120	34	29	13.7%
180	25	22	10.2%
240	20	18	7.4%
300	16	15	5.2%
360	14	14	3.2%
540	10	10	-1.3%
600	9	10	-2.6%
660	9	9	-3.7%
720	8	8	-4.8%
900	7	7	-7.5%
1080	6	6	-9.9%
1260	5	6	-11.9%
1440	5	5	-13.7%



City of Vaughan
Huntington Road Class EA

Hydrograph Parameters

NHYD	NAME	AREA [ha]	DWF [m³/s]	CN	IA [mm]	N	TP [hr]	STORM INDEX	RAIN [mm/hr]
7	Tributary 6	88.4	0	67	5	3	1.14	1	0
19	Tributary 5B	778.7	0	77	10	3	2.5	1	0
38	NasHyd - 38	10.4	0	80	5	3	0.2	1	0

NHYD	NAME	AREA [ha]	TIMP	XIMP	DWF [m³/s]	LOSS	SLPP [%]	LGP [m]	MNP	SCP [hr]	DPSI [mm]	SLPI [%]	LGI Type	LGI [m]	MNI	SCI [hr]
9	Tributary 4	305.1	0.9	0.7	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	1426.11	0.013	0
11	Tributary 3	68.6	0.9	0.7	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	676.32	0.013	0
12	Tributary 2	26.7	0.9	0.7	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	421.61	0.013	0
15	Tributary 1A	97.4	0.9	0.7	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	805.71	0.013	0
16	Tributary 1B	507.8	0.63	0.49	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	1840.01	0.013	0
18	Tributary 5C	195.9	0.9	0.7	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	1142.8	0.013	0
13	Tributary 5A	499.6	0.63	0.49	0	Modified SCS Curve Method	2	40	0.25	0	1	1	Auto	1824.95	0.013	0
27	Road Area 1	4.0	0.61	0.49	0	Horton's Equation	2	40	0.25	0	1	1	Auto	162.69	0.013	0
29	Road Area 2	2.2	0.5	0.35	0	Horton's Equation	2	40	0.25	0	1	1	Auto	121.66	0.013	0
31	Road Area 3	1.2	0.48	0.35	0	Horton's Equation	2	40	0.25	0	1	1	Auto	88.69	0.013	0
33	Road Area 4	1.8	0.54	0.35	0	Horton's Equation	2	40	0.25	0	1	1	Auto	110.45	0.013	0
35	Road Area 5	5.3	0.42	0.3	0	Horton's Equation	2	40	0.25	0	1	1	Auto	188.5	0.013	0
37	Tributary 1C	8.0	0.5	0.35	0	Horton's Equation	2	40	0.25	0	1	1	Auto	231.43	0.013	0

Huntington Road EA
Flow Comparison

Summary of Flows - TRCA Models

Culvert Station	1+330	2+460	3+320	3+768	4+687
River	4	3	3	1	6
Reach	1	1	1	2	1
XS	24.14	24.69	24.81		22.145
2-yr	4.0	6.2	3.9	12.6	0.5
5-yr	7.7	12.3	7.6	19.5	0.9
10-yr	11.0	17.3	11.1	24.3	1.2
25-yr	15.9	23.7	15.2	31.4	1.8
50-yr	20.2	28.8	18.9	37.0	2.5
100-yr	24.3	34.3	23.1	42.9	3.2
Regional	56.2	60.0	40.1	149.9	11.1

Summary of Flows - SEI Model

Culvert Station	1+330	2+460	3+320	3+768	4+687
River	4	3	3	1	6
Reach	1	1	1	2	1
XS	24.14	24.69	24.81		22.145
2-yr	4.3	17.1	7.9	8.1	0.4
5-yr	8.8	23.0	11.1	23.2	3.0
10-yr	11.7	27.1	14.8	30.9	4.0
25-yr	13.4	32.2	17.0	35.6	4.6
50-yr	14.0	36.3	17.7	37.1	4.8
100-yr	18.6	40.0	26.0	46.0	5.3
Regional	52.6	53.5	50.3	150.4	14.2

Summary of Flow Differences TRCA v SEI

Culvert Station	1+330	2+460	3+320	3+768	4+687
River	4	3	3	1	6
Reach	1	1	1	2	1
XS	24.14	24.69	24.81		22.145
2-yr	-0.3	-10.9	-4.0	4.5	0.1
5-yr	-1.1	-10.7	-3.5	-3.7	-2.1
10-yr	-0.7	-9.8	-3.7	-6.6	-2.8
25-yr	2.5	-8.5	-1.8	-4.2	-2.8
50-yr	6.2	-7.5	1.2	-0.1	-2.3
100-yr	5.7	-5.7	-2.9	-3.1	-2.1
Regional	3.6	6.5	-10.2	-0.5	-3.1

Summary of Flow Differences TRCA v SEI in % of TRCA flow

Culvert Station	1+330	2+460	3+320	3+768	4+687
River	4	3	3	1	6
Reach	1	1	1	2	1
XS	24.14	24.69	24.81		22.145
2-yr	-7.5%	-175.6%	-102.6%	35.7%	20.0%
5-yr	-14.3%	-86.9%	-46.1%	-19.0%	-233.3%
10-yr	-6.4%	-56.8%	-33.3%	-27.2%	-233.3%
25-yr	15.7%	-35.7%	-11.8%	-13.4%	-155.6%
50-yr	30.7%	-25.9%	6.3%	-0.3%	-92.0%
100-yr	23.5%	-16.6%	-12.6%	-7.2%	-65.6%
Regional	6.4%	10.9%	-25.4%	-0.3%	-27.9%

Huntington Road EA

Stormwater Management for Road Improvements

The drainage has five catchments

ROW Width (m)	Road Width (m) Existing	Road Width (m) Proposed
26	6	14.4

Area	Station	Station	Total Area		Paved Area		Imperviousness Ratio	
			m2	ha	Existing	Proposed	Existing	Proposed
1	0+220	0+560	9,407	0.94	0.20	0.49	0.22	0.52
2	0+560	2+240	39,654	3.97	1.01	2.42	0.25	0.61
3	2+240	3+380	33,923	3.39	0.68	1.64	0.20	0.48
4	3+380	4+068	18,270	1.83	0.41	0.99	0.23	0.54
5	4+552	6+118	53,343	5.33	0.94	2.25	0.18	0.42

Culvert Flows and Sizes

Distance	Station		Span		Rise	Sum of Areas	
			m		m		
	1+140		0.6				Area 2
191	1+331	West Rainbow Creek	3.2		2.15	39,654	Area 2
194	1+525	Rainbow Creek Tributary	0.75				Area 2
935	2+460	Rainbow Creek Tributary	1.88		1.26	22,169	Area 3
525	2+985		0.6				Area 3
335	3+320	East Rainbow Creek	2.24		1.639	11,754	Area 3
447	3+767	Robinson Creek	3			18,270	Area 4
920	4+687	East Robinson Creek	1.8		1.2	53,343	Area 5
1,106	5+793		0.6				Area 5
187	5+979		0.6				Area 5
247	6+226		0.6				Area 5
				sum		145,190	

Calculation of Culvert Flows

Culvert Flows

Station	Culvert Type	Diameter (mm)	Drainage Area (ha)	C values	t (min)	Q (m3/s)	Culvert diameter	Return period	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
								A	21.8	28.8	33.3	39.1	43.4	47.6
								B	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699
1+140	CSP	600	8.4	0.60	15	1.5	1.2		0.8	1.1	1.2	1.5	1.6	1.8
1+525	CSP	750	27	0.70	45	2.5	1.5		1.4	1.9	2.2	2.5	2.8	3.1
2+985	CSP	600	10.4	0.60	17	1.6	1.5		0.9	1.2	1.4	1.6	1.8	2.0
5+793	CSP	600	3.1	0.40	10	0.5	0.9		0.3	0.3	0.4	0.5	0.5	0.6
5+979	CSP	600	5.4	0.40	10	0.8	0.9		0.5	0.6	0.7	0.8	0.9	1.0
6+226	CSP	600	4.7	0.40	10	0.7	0.9		0.4	0.5	0.6	0.7	0.8	0.9

Calculation of Flows produced by Road - Parameters

Stormwater Management for Road Improvements

The drainage has five catchments

					ROW Width (m)	Road Width (m)								
						Existing	Proposed							
					26	6	14.4	C-value impervious	C-Value Pervious	C values	L (m)	Slope (%)	Tc (min)
													Airport
Area	Station	Station	Total Area		Paved Area		Imperviousness Ratio						
			m2	ha	Existing	Proposed	Existing	Proposed					
1	0+220	0+560	9,407	0.94	0.20	0.49	0.22	0.52	0.90	0.20	0.56	340.00	2.00	26
2	0+560	2+240	39,654	3.97	1.01	2.42	0.25	0.61	0.90	0.20	0.63	1680.00	2.00	50
3	2+240	3+380	33,923	3.39	0.68	1.64	0.20	0.48	0.90	0.20	0.54	1140.00	2.00	49
4	3+380	4+068	18,270	1.83	0.41	0.99	0.23	0.54	0.90	0.20	0.58	688.00	2.00	35
5	4+552	6+118	53,343	5.33	0.94	2.25	0.18	0.42	0.90	0.20	0.50	1565.96	2.00	62

Flows Produced by Road vs. Allowable

Stormwater Management for Road Improvements

							Allowable outflow (l/s)					
The drainag	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
	21.8	28.8	33.3	39.1	43.4	47.6						
Area	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699						
	Litres per second											
1	59	78	90	105	117	128	9	14	17	21	25	28
2	172	227	262	308	342	375	34	52	64	80	94	106
3	128	169	196	230	255	280	29	45	55	70	81	92
4	93	123	143	168	186	204	17	25	31	40	46	52
5	158	208	241	283	314	345	44	67	83	105	123	139

Effect of Bioretention on Flows by Road vs Allowable Flows

Bioretention facilities

Grassed Swales

Stormwater Management for Road Improvements

With LID							Allowable outflow (l/s)					
The drainag	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
	21.8	28.8	33.3	39.1	43.4	47.6						
Area	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699						
	Litres per second											
1	17	22	26	30	33	37	9	14	17	21	25	28
2	42	55	63	74	83	91	34	52	64	80	94	106
3	39	52	60	70	78	86	29	45	55	70	81	92
4	26	34	39	46	51	56	17	25	31	40	46	52
5	54	72	83	98	108	119	44	67	83	105	123	139


19
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24

```
  ************************************
  ** Run 01                         **
  ************************************

  W/E COMMAND              HYD ID   DT       AREA   Qpeak Tpeak   R.V. R.C.    Qbase
                                    min       ha     cms   hrs     mm           cms
 ** CALIB STANDHYD        0009  1  5.0     306.59   13.64  5.33  38.00 0.90   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD        0011  1  5.0      70.92    3.45  5.25  38.00 0.90   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD        0012  1  5.0      26.93    1.30  5.25  37.08 0.88   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD          0007  1  5.0      93.73    0.59  6.42   8.45 0.20   0.000
    [CN=67.0          ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD        0016  1  5.0     512.67   16.98  5.42  31.17 0.74   0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD        0015  1  5.0      97.38    4.60  5.25  37.62 0.90   0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]    0023  1  5.0      97.38    3.72  5.42  37.61  n/a   0.000
*
    ADD [  0016+  0023]   0024  3  5.0     610.04   20.70  5.42  32.20  n/a   0.000
*
 ** CALIB NASHYD          0019  1  5.0     778.65    3.34  8.42   9.49 0.23   0.000
    [CN=77.0          ]
    [ N = 3.0:Tp 2.50]
*
 ** CALIB STANDHYD        0013  1  5.0     499.57   16.58  5.42  31.17 0.74   0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]   0020  3  5.0    1278.22   17.07  5.42  17.97  n/a   0.000
*
    PIPE   [ 2:  0020]    0021  1  5.0    1278.22   16.61  5.58  17.97  n/a   0.000
*
 *  CALIB STANDHYD        0018  1  5.0     197.76    9.00  5.25  37.85 0.90   0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]   0025  3  5.0    1475.98   24.70  5.42  20.63  n/a   0.000
*
```

```
  ************************************
  ** Run 02                         **
  ************************************

  W/E COMMAND              HYD ID   DT        AREA   Qpeak Tpeak   R.V. R.C.    Qbase
                                    min        ha     cms   hrs     mm           cms
*
 ** CALIB STANDHYD       0009  1  5.0     306.59   18.41  5.25  50.19 0.92   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0011  1  5.0      70.92    4.58  5.25  50.19 0.92   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0012  1  5.0      26.93    1.74  5.25  49.13 0.90   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD         0007  1  5.0      93.73    0.98  6.42  13.97 0.26   0.000
    [CN=67.0          ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD       0016  1  5.0     512.67   24.05  5.33  42.39 0.78   0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD       0015  1  5.0      97.38    6.19  5.25  49.76 0.92   0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]   0023  1  5.0      97.38    5.10  5.42  49.75  n/a   0.000
*
    ADD [  0016+  0023]  0024  3  5.0     610.04   29.11  5.42  43.56  n/a   0.000
*
 ** CALIB NASHYD         0019  1  5.0     778.65    5.83  8.25  16.38 0.30   0.000
    [CN=77.0          ]
    [ N = 3.0:Tp 2.50]
*
 ** CALIB STANDHYD       0013  1  5.0     499.57   23.48  5.33  42.39 0.78   0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]  0020  3  5.0    1278.22   24.48  5.42  26.54  n/a   0.000
*
    PIPE   [ 2:  0020]   0021  1  5.0    1278.22   23.80  5.50  26.54  n/a   0.000
*
 *  CALIB STANDHYD       0018  1  5.0     197.76   12.17  5.25  50.02 0.92   0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]  0025  3  5.0    1475.98   35.01  5.42  29.69  n/a   0.000
*
```

```
  ************************************
  ** Run 03                         **
  ************************************

  W/E COMMAND             HYD ID   DT       AREA   Qpeak Tpeak   R.V. R.C.    Qbase
                                   min        ha     cms   hrs     mm          cms
*
 ** CALIB STANDHYD       0009  1  5.0     306.59   21.77  5.25  58.43 0.93   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0011  1  5.0      70.92    5.35  5.25  58.43 0.93   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0012  1  5.0      26.93    2.03  5.25  57.30 0.91   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD         0007  1  5.0      93.73    1.29  6.33  18.22 0.29   0.000
    [CN=67.0         ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD       0016  1  5.0     512.67   28.86  5.33  50.10 0.80   0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD       0015  1  5.0      97.38    7.23  5.25  57.97 0.92   0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]   0023  1  5.0      97.38    6.03  5.42  57.96  n/a   0.000
*
    ADD [  0016+  0023]  0024  3  5.0     610.04   34.79  5.33  51.35  n/a   0.000
*
 *  CALIB NASHYD         0019  1  5.0     778.65    7.73  8.25  21.61 0.34   0.000
    [CN=77.0         ]
    [ N = 3.0:Tp 2.50]
*
 *  CALIB STANDHYD       0013  1  5.0     499.57   28.17  5.33  50.10 0.80   0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]  0020  3  5.0    1278.22   29.58  5.42  32.74  n/a   0.000
*
    PIPE   [ 2:  0020]   0021  1  5.0    1278.22   28.84  5.50  32.74  n/a   0.000
*
 *  CALIB STANDHYD       0018  1  5.0     197.76   14.27  5.25  58.25 0.93   0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]  0025  3  5.0    1475.98   41.94  5.42  36.16  n/a   0.000
*
```

```
  W/E COMMAND                HYD ID   DT        AREA   Qpeak Tpeak    R.V. R.C.    Qbase
                                      min         ha     cms   hrs     mm           cms
*
 ** CALIB STANDHYD          0009  1  5.0     306.59   25.84  5.25   68.73 0.94    0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD          0011  1  5.0      70.92    6.33  5.25   68.73 0.94    0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD          0012  1  5.0      26.93    2.40  5.25   67.52 0.92    0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD            0007  1  5.0      93.73    1.71  6.33   24.00 0.33    0.000
    [CN=67.0         ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD          0016  1  5.0     512.67   35.54  5.33   59.85 0.82    0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD          0015  1  5.0      97.38    8.55  5.25   68.24 0.93    0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]      0023  1  5.0      97.38    7.22  5.42   68.23  n/a    0.000
*
    ADD [  0016+  0023]     0024  3  5.0     610.04   42.67  5.33   61.19  n/a    0.000
*
 *  CALIB NASHYD            0019  1  5.0     778.65   10.30  8.17   28.65 0.39    0.000
    [CN=77.0         ]
    [ N = 3.0:Tp 2.50]
*
 *  CALIB STANDHYD          0013  1  5.0     499.57   34.68  5.33   59.85 0.82    0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]     0020  3  5.0    1278.22   36.59  5.42   40.85  n/a    0.000
*
    PIPE   [ 2:  0020]      0021  1  5.0    1278.22   35.79  5.50   40.85  n/a    0.000
*
 *  CALIB STANDHYD          0018  1  5.0     197.76   17.09  5.25   68.53 0.94    0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]     0025  3  5.0    1475.98   51.21  5.33   44.56  n/a    0.000
*
```

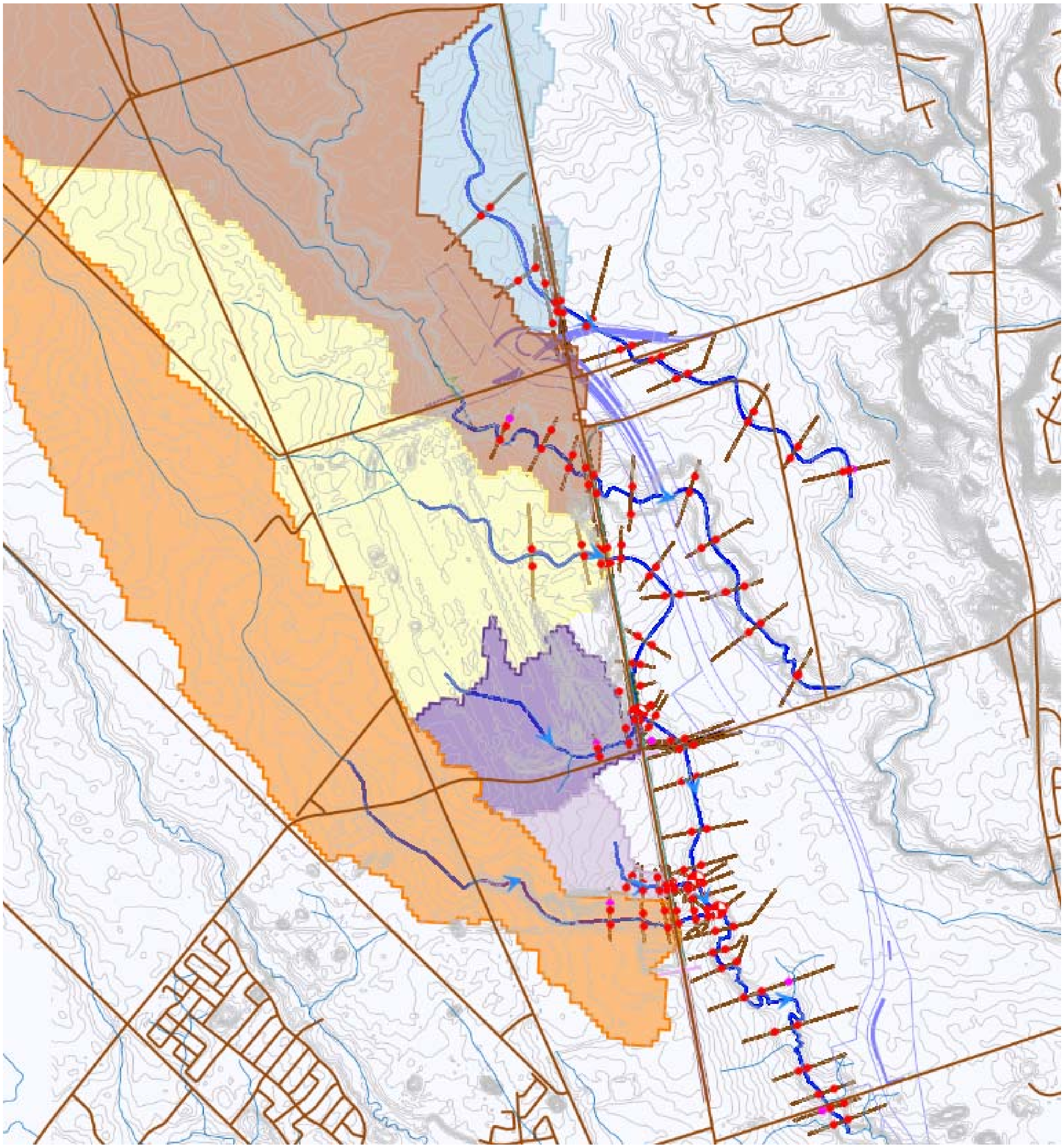
```
  W/E COMMAND             HYD ID   DT        AREA   Qpeak Tpeak    R.V. R.C.    Qbase
                                   min        ha     cms   hrs      mm           cms
*
 ** CALIB STANDHYD       0009  1  5.0     306.59   29.18  5.25   76.39 0.95   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0011  1  5.0      70.92    7.08  5.25   76.39 0.95   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0012  1  5.0      26.93    2.69  5.25   75.14 0.93   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD         0007  1  5.0      93.73    2.04  6.33   28.61 0.35   0.000
    [CN=67.0          ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD       0016  1  5.0     512.67   40.29  5.33   67.18 0.83   0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD       0015  1  5.0      97.38    9.61  5.25   75.89 0.94   0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]   0023  1  5.0      97.38    8.20  5.42   75.88  n/a   0.000
*
    ADD [  0016+  0023]  0024  3  5.0     610.04   48.43  5.33   68.57  n/a   0.000
*
 *  CALIB NASHYD         0019  1  5.0     778.65   12.33  8.08   34.19 0.42   0.000
    [CN=77.0          ]
    [ N = 3.0:Tp 2.50]
*
 *  CALIB STANDHYD       0013  1  5.0     499.57   39.32  5.33   67.18 0.83   0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]  0020  3  5.0    1278.22   41.69  5.33   47.08  n/a   0.000
*
    PIPE   [ 2:  0020]   0021  1  5.0    1278.22   40.59  5.42   47.08  n/a   0.000
*
 *  CALIB STANDHYD       0018  1  5.0     197.76   19.16  5.25   76.19 0.94   0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]  0025  3  5.0    1475.98   58.38  5.33   50.98  n/a   0.000
*
```

```
 ***********************************
 ** Run 06                       **
 ***********************************

 W/E COMMAND              HYD ID   DT        AREA    Qpeak Tpeak    R.V. R.C.     Qbase
                                   min        ha      cms   hrs      mm            cms
*
 ** CALIB STANDHYD       0009  1  5.0     306.59   32.22  5.25   84.07 0.95   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0011  1  5.0      70.92    7.79  5.25   84.07 0.95   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD       0012  1  5.0      26.93    2.95  5.25   82.78 0.93   0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD         0007  1  5.0      93.73    2.39  6.33   33.45 0.38   0.000
    [CN=67.0          ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD       0016  1  5.0     512.67   45.06  5.33   74.55 0.84   0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD       0015  1  5.0      97.38   10.58  5.25   83.55 0.94   0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]   0023  1  5.0      97.38    9.09  5.42   83.54  n/a   0.000
*
    ADD [  0016+  0023]  0024  3  5.0     610.04   54.10  5.33   75.99  n/a   0.000
*
 *  CALIB NASHYD         0019  1  5.0     778.65   14.44  8.08   39.95 0.45   0.000
    [CN=77.0          ]
    [ N = 3.0:Tp 2.50]
*
 *  CALIB STANDHYD       0013  1  5.0     499.57   43.96  5.33   74.55 0.84   0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]  0020  3  5.0    1278.22   46.88  5.33   53.47  n/a   0.000
*
    PIPE   [ 2:  0020]   0021  1  5.0    1278.22   45.71  5.42   53.47  n/a   0.000
*
 *  CALIB STANDHYD       0018  1  5.0     197.76   21.13  5.25   83.86 0.95   0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]  0025  3  5.0    1475.98   65.34  5.33   57.54  n/a   0.000
*
```

```
 ************************************
 ** Run 07                        **
 ************************************

 W/E COMMAND              HYD ID   DT       AREA    Qpeak Tpeak    R.V. R.C.    Qbase
                                   min        ha      cms   hrs      mm           cms
*
 ** CALIB STANDHYD        0009  1  5.0    306.59    43.16 10.00 207.22 0.98     0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD        0011  1  5.0     70.92    10.33 10.00 207.22 0.98     0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB STANDHYD        0012  1  5.0     26.93     3.94 10.00 205.65 0.97     0.000
    [I%=70.0:S%= 2.00]
*
 ** CALIB NASHYD          0007  1  5.0     93.73     7.58 11.33 129.02 0.61     0.000
    [CN=67.0         ]
    [ N = 3.0:Tp 1.14]
*
 ** CALIB STANDHYD        0016  1  5.0    512.67    67.20 10.08 195.43 0.92     0.000
    [I%=49.0:S%= 2.00]
*
 ** CALIB STANDHYD        0015  1  5.0     97.38    14.11 10.00 206.60 0.97     0.000
    [I%=70.0:S%= 2.00]
*
    PIPE   [ 2:  0015]    0023  1  5.0     97.38    12.51 10.17 206.60  n/a     0.000
*
    ADD [  0016+  0023]   0024  3  5.0    610.04    79.59 10.08 197.21  n/a     0.000
*
 *  CALIB NASHYD          0019  1  5.0    778.65    48.53 12.50 146.85 0.69     0.000
    [CN=77.0         ]
    [ N = 3.0:Tp 2.50]
*
 *  CALIB STANDHYD        0013  1  5.0    499.57    65.55 10.08 195.43 0.92     0.000
    [I%=49.0:S%= 2.00]
*
    ADD [  0013+  0019]   0020  3  5.0   1278.22    91.53 10.17 165.83  n/a     0.000
*
    PIPE   [ 2:  0020]    0021  1  5.0   1278.22    90.82 10.33 165.83  n/a     0.000
*
 *  CALIB STANDHYD        0018  1  5.0    197.76    28.26 10.00 206.96 0.98     0.000
    [I%=70.0:S%= 2.00]
*
    ADD [  0018+  0021]   0025  3  5.0   1475.98   116.75 10.17 171.34  n/a     0.000
*
```

APPENDIX B

Hydraulic Model Results



River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
East Robinson Creek	Main Channel	3761.8	2 Year	0.4	207.11
East Robinson Creek	Main Channel	3761.8	5 Year	3.0	207.31
East Robinson Creek	Main Channel	3761.8	10 Year	4.0	207.35
East Robinson Creek	Main Channel	3761.8	25 Year	4.6	207.37
East Robinson Creek	Main Channel	3761.8	50 Year	4.8	207.38
East Robinson Creek	Main Channel	3761.8	100 Year	5.3	207.40
East Robinson Creek	Main Channel	3761.8	H. Hazel	14.2	207.62
East Robinson Creek	Main Channel	3326	2 Year	0.4	204.16
East Robinson Creek	Main Channel	3326	5 Year	3.0	204.36
East Robinson Creek	Main Channel	3326	10 Year	4.0	204.41
East Robinson Creek	Main Channel	3326	25 Year	4.6	204.43
East Robinson Creek	Main Channel	3326	50 Year	4.8	204.44
East Robinson Creek	Main Channel	3326	100 Year	5.3	204.45
East Robinson Creek	Main Channel	3326	H. Hazel	14.2	204.67
East Robinson Creek	Main Channel	3153	2 Year	0.4	203.31
East Robinson Creek	Main Channel	3153	5 Year	3.0	203.68
East Robinson Creek	Main Channel	3153	10 Year	4.0	203.74
East Robinson Creek	Main Channel	3153	25 Year	4.6	203.78
East Robinson Creek	Main Channel	3153	50 Year	4.8	203.79
East Robinson Creek	Main Channel	3153	100 Year	5.3	203.83
East Robinson Creek	Main Channel	3153	H. Hazel	14.2	204.27
East Robinson Creek	Main Channel	3097	2 Year	0.4	203.27
East Robinson Creek	Main Channel	3097	5 Year	3.0	203.62
East Robinson Creek	Main Channel	3097	10 Year	4.0	203.69
East Robinson Creek	Main Channel	3097	25 Year	4.6	203.72
East Robinson Creek	Main Channel	3097	50 Year	4.8	203.73
East Robinson Creek	Main Channel	3097	100 Year	5.3	203.77
East Robinson Creek	Main Channel	3097	H. Hazel	14.2	204.27
East Robinson Creek	Main Channel	3088		Culvert	
East Robinson Creek	Main Channel	3061.7	2 Year	0.4	203.23
East Robinson Creek	Main Channel	3061.7	5 Year	3.0	203.55
East Robinson Creek	Main Channel	3061.7	10 Year	4.0	203.61

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
East Robinson Creek	Main Channel	3061.7	25 Year	4.6	203.64
East Robinson Creek	Main Channel	3061.7	50 Year	4.8	203.65
East Robinson Creek	Main Channel	3061.7	100 Year	5.3	203.67
East Robinson Creek	Main Channel	3061.7	H. Hazel	14.2	203.86
East Robinson Creek	Main Channel	2888.7	2 Year	0.4	202.23
East Robinson Creek	Main Channel	2888.7	5 Year	3.0	202.48
East Robinson Creek	Main Channel	2888.7	10 Year	4.0	202.54
East Robinson Creek	Main Channel	2888.7	25 Year	4.6	202.56
East Robinson Creek	Main Channel	2888.7	50 Year	4.8	202.57
East Robinson Creek	Main Channel	2888.7	100 Year	5.3	202.59
East Robinson Creek	Main Channel	2888.7	H. Hazel	14.2	202.83
East Robinson Creek	Main Channel	2645.5	2 Year	0.4	201.16
East Robinson Creek	Main Channel	2645.5	5 Year	3.0	201.34
East Robinson Creek	Main Channel	2645.5	10 Year	4.0	201.39
East Robinson Creek	Main Channel	2645.5	25 Year	4.6	201.41
East Robinson Creek	Main Channel	2645.5	50 Year	4.8	201.42
East Robinson Creek	Main Channel	2645.5	100 Year	5.3	201.43
East Robinson Creek	Main Channel	2645.5	H. Hazel	14.2	201.64
East Robinson Creek	Main Channel	2457.9	2 Year	0.4	200.20
East Robinson Creek	Main Channel	2457.9	5 Year	3.0	200.43
East Robinson Creek	Main Channel	2457.9	10 Year	4.0	200.48
East Robinson Creek	Main Channel	2457.9	25 Year	4.6	200.50
East Robinson Creek	Main Channel	2457.9	50 Year	4.8	200.51
East Robinson Creek	Main Channel	2457.9	100 Year	5.3	200.53
East Robinson Creek	Main Channel	2457.9	H. Hazel	14.2	200.76
East Robinson Creek	Main Channel	2272.6	2 Year	0.4	199.15
East Robinson Creek	Main Channel	2272.6	5 Year	3.0	199.31
East Robinson Creek	Main Channel	2272.6	10 Year	4.0	199.35
East Robinson Creek	Main Channel	2272.6	25 Year	4.6	199.37
East Robinson Creek	Main Channel	2272.6	50 Year	4.8	199.37
East Robinson Creek	Main Channel	2272.6	100 Year	5.3	199.39
East Robinson Creek	Main Channel	2272.6	H. Hazel	14.2	199.57

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
East Robinson Creek	Main Channel	1740.5	2 Year	0.4	196.20
East Robinson Creek	Main Channel	1740.5	5 Year	3.0	196.42
East Robinson Creek	Main Channel	1740.5	10 Year	4.0	196.47
East Robinson Creek	Main Channel	1740.5	25 Year	4.6	196.50
East Robinson Creek	Main Channel	1740.5	50 Year	4.8	196.50
East Robinson Creek	Main Channel	1740.5	100 Year	5.3	196.52
East Robinson Creek	Main Channel	1740.5	H. Hazel	14.2	196.75
East Robinson Creek	Main Channel	1417	2 Year	0.4	195.18
East Robinson Creek	Main Channel	1417	5 Year	3.0	195.38
East Robinson Creek	Main Channel	1417	10 Year	4.0	195.42
East Robinson Creek	Main Channel	1417	25 Year	4.6	195.45
East Robinson Creek	Main Channel	1417	50 Year	4.8	195.45
East Robinson Creek	Main Channel	1417	100 Year	5.3	195.47
East Robinson Creek	Main Channel	1417	H. Hazel	14.2	195.68
East Robinson Creek	Main Channel	1000	2 Year	0.4	192.16
East Robinson Creek	Main Channel	1000	5 Year	3.0	192.37
East Robinson Creek	Main Channel	1000	10 Year	4.0	192.41
East Robinson Creek	Main Channel	1000	25 Year	4.6	192.43
East Robinson Creek	Main Channel	1000	50 Year	4.8	192.44
East Robinson Creek	Main Channel	1000	100 Year	5.3	192.46
East Robinson Creek	Main Channel	1000	H. Hazel	14.2	192.68
Robinson Creek	Main Reach	3685	2 Year	8.1	198.34
Robinson Creek	Main Reach	3685	5 Year	23.2	198.35
Robinson Creek	Main Reach	3685	10 Year	30.9	198.42
Robinson Creek	Main Reach	3685	25 Year	35.6	198.46
Robinson Creek	Main Reach	3685	50 Year	37.1	198.47
Robinson Creek	Main Reach	3685	100 Year	46.0	198.53
Robinson Creek	Main Reach	3685	H. Hazel	150.4	199.02
Robinson Creek	Main Reach	3325.8	2 Year	8.1	196.31
Robinson Creek	Main Reach	3325.8	5 Year	23.2	197.15
Robinson Creek	Main Reach	3325.8	10 Year	30.9	197.59
Robinson Creek	Main Reach	3325.8	25 Year	35.6	197.85
Robinson Creek	Main Reach	3325.8	50 Year	37.1	197.92

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Robinson Creek	Main Reach	3325.8	100 Year	46.0	198.15
Robinson Creek	Main Reach	3325.8	H. Hazel	150.4	198.93
Robinson Creek	Main Reach	3144.9	2 Year	8.1	196.14
Robinson Creek	Main Reach	3144.9	5 Year	23.2	197.15
Robinson Creek	Main Reach	3144.9	10 Year	30.9	197.59
Robinson Creek	Main Reach	3144.9	25 Year	35.6	197.85
Robinson Creek	Main Reach	3144.9	50 Year	37.1	197.92
Robinson Creek	Main Reach	3144.9	100 Year	46.0	198.14
Robinson Creek	Main Reach	3144.9	H. Hazel	150.4	198.92
Robinson Creek	Main Reach	3019.7	2 Year	8.1	196.12
Robinson Creek	Main Reach	3019.7	5 Year	23.2	197.14
Robinson Creek	Main Reach	3019.7	10 Year	30.9	197.59
Robinson Creek	Main Reach	3019.7	25 Year	35.6	197.84
Robinson Creek	Main Reach	3019.7	50 Year	37.1	197.92
Robinson Creek	Main Reach	3019.7	100 Year	46.0	198.14
Robinson Creek	Main Reach	3019.7	H. Hazel	150.4	198.91
Robinson Creek	Main Reach	2994		Culvert	
Robinson Creek	Main Reach	2982.5	2 Year	8.1	195.86
Robinson Creek	Main Reach	2982.5	5 Year	23.2	196.29
Robinson Creek	Main Reach	2982.5	10 Year	30.9	196.42
Robinson Creek	Main Reach	2982.5	25 Year	35.6	196.48
Robinson Creek	Main Reach	2982.5	50 Year	37.1	196.50
Robinson Creek	Main Reach	2982.5	100 Year	46.0	196.61
Robinson Creek	Main Reach	2982.5	H. Hazel	150.4	197.21
Robinson Creek	Main Reach	2656.6	2 Year	8.1	194.39
Robinson Creek	Main Reach	2656.6	5 Year	23.2	194.55
Robinson Creek	Main Reach	2656.6	10 Year	30.9	194.60
Robinson Creek	Main Reach	2656.6	25 Year	35.6	194.63
Robinson Creek	Main Reach	2656.6	50 Year	37.1	194.64
Robinson Creek	Main Reach	2656.6	100 Year	46.0	194.68
Robinson Creek	Main Reach	2656.6	H. Hazel	150.4	195.01

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Robinson Creek	Main Reach	2306.6	2 Year	8.1	192.50
Robinson Creek	Main Reach	2306.6	5 Year	23.2	192.77
Robinson Creek	Main Reach	2306.6	10 Year	30.9	192.88
Robinson Creek	Main Reach	2306.6	25 Year	35.6	192.93
Robinson Creek	Main Reach	2306.6	50 Year	37.1	192.95
Robinson Creek	Main Reach	2306.6	100 Year	46.0	193.04
Robinson Creek	Main Reach	2306.6	H. Hazel	150.4	193.70
Robinson Creek	Main Reach	1979	2 Year	8.1	191.71
Robinson Creek	Main Reach	1979	5 Year	23.2	192.03
Robinson Creek	Main Reach	1979	10 Year	30.9	192.14
Robinson Creek	Main Reach	1979	25 Year	35.6	192.19
Robinson Creek	Main Reach	1979	50 Year	37.1	192.20
Robinson Creek	Main Reach	1979	100 Year	46.0	192.27
Robinson Creek	Main Reach	1979	H. Hazel	150.4	192.84
Robinson Creek	Main Reach	1677.7	2 Year	8.1	190.43
Robinson Creek	Main Reach	1677.7	5 Year	23.2	190.66
Robinson Creek	Main Reach	1677.7	10 Year	30.9	190.74
Robinson Creek	Main Reach	1677.7	25 Year	35.6	190.80
Robinson Creek	Main Reach	1677.7	50 Year	37.1	190.83
Robinson Creek	Main Reach	1677.7	100 Year	46.0	190.95
Robinson Creek	Main Reach	1677.7	H. Hazel	150.4	191.80
Robinson Creek	Main Reach	1436.2	2 Year	8.1	189.85
Robinson Creek	Main Reach	1436.2	5 Year	23.2	190.26
Robinson Creek	Main Reach	1436.2	10 Year	30.9	190.40
Robinson Creek	Main Reach	1436.2	25 Year	35.6	190.48
Robinson Creek	Main Reach	1436.2	50 Year	37.1	190.50
Robinson Creek	Main Reach	1436.2	100 Year	46.0	190.63
Robinson Creek	Main Reach	1436.2	H. Hazel	150.4	191.50
Robinson Creek	Main Reach	1000	2 Year	8.1	188.80
Robinson Creek	Main Reach	1000	5 Year	23.2	189.25
Robinson Creek	Main Reach	1000	10 Year	30.9	189.40
Robinson Creek	Main Reach	1000	25 Year	35.6	189.47
Robinson Creek	Main Reach	1000	50 Year	37.1	189.50

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Robinson Creek	Main Reach	1000	100 Year	46.0	189.62
Robinson Creek	Main Reach	1000	H. Hazel	150.4	190.30
East Rainbow Creek	Tributary 4	2581.6	2 Year	7.9	200.60
East Rainbow Creek	Tributary 4	2581.6	5 Year	11.1	200.66
East Rainbow Creek	Tributary 4	2581.6	10 Year	14.8	200.72
East Rainbow Creek	Tributary 4	2581.6	25 Year	17.0	200.75
East Rainbow Creek	Tributary 4	2581.6	50 Year	17.7	200.76
East Rainbow Creek	Tributary 4	2581.6	100 Year	26.0	200.85
East Rainbow Creek	Tributary 4	2581.6	H. Hazel	50.3	201.01
East Rainbow Creek	Tributary 4	2255.1	2 Year	7.9	200.59
East Rainbow Creek	Tributary 4	2255.1	5 Year	11.1	200.66
East Rainbow Creek	Tributary 4	2255.1	10 Year	14.8	200.72
East Rainbow Creek	Tributary 4	2255.1	25 Year	17.0	200.75
East Rainbow Creek	Tributary 4	2255.1	50 Year	17.7	200.75
East Rainbow Creek	Tributary 4	2255.1	100 Year	26.0	200.84
East Rainbow Creek	Tributary 4	2255.1	H. Hazel	50.3	200.99
East Rainbow Creek	Tributary 4	2164.6	2 Year	7.9	200.59
East Rainbow Creek	Tributary 4	2164.6	5 Year	11.1	200.66
East Rainbow Creek	Tributary 4	2164.6	10 Year	14.8	200.72
East Rainbow Creek	Tributary 4	2164.6	25 Year	17.0	200.74
East Rainbow Creek	Tributary 4	2164.6	50 Year	17.7	200.75
East Rainbow Creek	Tributary 4	2164.6	100 Year	26.0	200.84
East Rainbow Creek	Tributary 4	2164.6	H. Hazel	50.3	200.98
East Rainbow Creek	Tributary 4	2137	2 Year	7.9	200.59
East Rainbow Creek	Tributary 4	2137	5 Year	11.1	200.66
East Rainbow Creek	Tributary 4	2137	10 Year	14.8	200.72
East Rainbow Creek	Tributary 4	2137	25 Year	17.0	200.74
East Rainbow Creek	Tributary 4	2137	50 Year	17.7	200.75
East Rainbow Creek	Tributary 4	2137	100 Year	26.0	200.83
East Rainbow Creek	Tributary 4	2137	H. Hazel	50.3	200.97
East Rainbow Creek	Tributary 4	2125		Culvert	

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
East Rainbow Creek	Tributary 4	2111.6	2 Year	7.9	199.42
East Rainbow Creek	Tributary 4	2111.6	5 Year	11.1	199.47
East Rainbow Creek	Tributary 4	2111.6	10 Year	14.8	199.52
East Rainbow Creek	Tributary 4	2111.6	25 Year	17.0	199.54
East Rainbow Creek	Tributary 4	2111.6	50 Year	17.7	199.56
East Rainbow Creek	Tributary 4	2111.6	100 Year	26.0	199.64
East Rainbow Creek	Tributary 4	2111.6	H. Hazel	50.3	199.81
East Rainbow Creek	Tributary 4	2048.6	2 Year	7.9	198.39
East Rainbow Creek	Tributary 4	2048.6	5 Year	11.1	198.45
East Rainbow Creek	Tributary 4	2048.6	10 Year	14.8	198.51
East Rainbow Creek	Tributary 4	2048.6	25 Year	17.0	198.54
East Rainbow Creek	Tributary 4	2048.6	50 Year	17.7	198.54
East Rainbow Creek	Tributary 4	2048.6	100 Year	26.0	198.63
East Rainbow Creek	Tributary 4	2048.6	H. Hazel	50.3	198.82
East Rainbow Creek	Tributary 4	1854.9	2 Year	7.9	197.21
East Rainbow Creek	Tributary 4	1854.9	5 Year	11.1	197.30
East Rainbow Creek	Tributary 4	1854.9	10 Year	14.8	197.38
East Rainbow Creek	Tributary 4	1854.9	25 Year	17.0	197.42
East Rainbow Creek	Tributary 4	1854.9	50 Year	17.7	197.42
East Rainbow Creek	Tributary 4	1854.9	100 Year	26.0	197.53
East Rainbow Creek	Tributary 4	1854.9	H. Hazel	50.3	197.75
East Rainbow Creek	Tributary 4	1688.6	2 Year	7.9	196.37
East Rainbow Creek	Tributary 4	1688.6	5 Year	11.1	196.42
East Rainbow Creek	Tributary 4	1688.6	10 Year	14.8	196.47
East Rainbow Creek	Tributary 4	1688.6	25 Year	17.0	196.50
East Rainbow Creek	Tributary 4	1688.6	50 Year	17.7	196.51
East Rainbow Creek	Tributary 4	1688.6	100 Year	26.0	196.59
East Rainbow Creek	Tributary 4	1688.6	H. Hazel	50.3	196.79
East Rainbow Creek	Tributary 4	1393.5	2 Year	7.9	195.44
East Rainbow Creek	Tributary 4	1393.5	5 Year	11.1	195.52
East Rainbow Creek	Tributary 4	1393.5	10 Year	14.8	195.59
East Rainbow Creek	Tributary 4	1393.5	25 Year	17.0	195.63
East Rainbow Creek	Tributary 4	1393.5	50 Year	17.7	195.64

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
East Rainbow Creek	Tributary 4	1393.5	100 Year	26.0	195.76
East Rainbow Creek	Tributary 4	1393.5	H. Hazel	50.3	196.02
East Rainbow Creek	Tributary 4	1260.4	2 Year	7.9	195.36
East Rainbow Creek	Tributary 4	1260.4	5 Year	11.1	195.42
East Rainbow Creek	Tributary 4	1260.4	10 Year	14.8	195.48
East Rainbow Creek	Tributary 4	1260.4	25 Year	17.0	195.51
East Rainbow Creek	Tributary 4	1260.4	50 Year	17.7	195.52
East Rainbow Creek	Tributary 4	1260.4	100 Year	26.0	195.62
East Rainbow Creek	Tributary 4	1260.4	H. Hazel	50.3	195.84
East Rainbow Creek	Tributary 4	1121.8	2 Year	7.9	195.29
East Rainbow Creek	Tributary 4	1121.8	5 Year	11.1	195.34
East Rainbow Creek	Tributary 4	1121.8	10 Year	14.8	195.40
East Rainbow Creek	Tributary 4	1121.8	25 Year	17.0	195.42
East Rainbow Creek	Tributary 4	1121.8	50 Year	17.7	195.43
East Rainbow Creek	Tributary 4	1121.8	100 Year	26.0	195.52
East Rainbow Creek	Tributary 4	1121.8	H. Hazel	50.3	195.71
East Rainbow Creek	Tributary 4	1000	2 Year	7.9	195.08
East Rainbow Creek	Tributary 4	1000	5 Year	11.1	195.11
East Rainbow Creek	Tributary 4	1000	10 Year	14.8	195.12
East Rainbow Creek	Tributary 4	1000	25 Year	17.0	195.14
East Rainbow Creek	Tributary 4	1000	50 Year	17.7	195.14
East Rainbow Creek	Tributary 4	1000	100 Year	26.0	195.19
East Rainbow Creek	Tributary 4	1000	H. Hazel	50.3	195.29
Rainbow Creek	Tributary 3	1364.6	2 Year	3.5	197.89
Rainbow Creek	Tributary 3	1364.6	5 Year	4.6	197.93
Rainbow Creek	Tributary 3	1364.6	10 Year	5.4	197.92
Rainbow Creek	Tributary 3	1364.6	25 Year	6.3	197.95
Rainbow Creek	Tributary 3	1364.6	50 Year	7.1	197.97
Rainbow Creek	Tributary 3	1364.6	100 Year	7.8	197.99
Rainbow Creek	Tributary 3	1364.6	H. Hazel	10.3	198.04
Rainbow Creek	Tributary 3	1185.1	2 Year	3.5	194.77
Rainbow Creek	Tributary 3	1185.1	5 Year	4.6	194.80

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Rainbow Creek	Tributary 3	1185.1	10 Year	5.4	194.87
Rainbow Creek	Tributary 3	1185.1	25 Year	6.3	194.98
Rainbow Creek	Tributary 3	1185.1	50 Year	7.1	195.03
Rainbow Creek	Tributary 3	1185.1	100 Year	7.8	195.08
Rainbow Creek	Tributary 3	1185.1	H. Hazel	10.3	195.16
Rainbow Creek	Tributary 3	1028.3	2 Year	3.5	194.71
Rainbow Creek	Tributary 3	1028.3	5 Year	4.6	194.83
Rainbow Creek	Tributary 3	1028.3	10 Year	5.4	194.90
Rainbow Creek	Tributary 3	1028.3	25 Year	6.3	194.99
Rainbow Creek	Tributary 3	1028.3	50 Year	7.1	195.03
Rainbow Creek	Tributary 3	1028.3	100 Year	7.8	195.08
Rainbow Creek	Tributary 3	1028.3	H. Hazel	10.3	195.16
Rainbow Creek	Tributary 3	1012		Culvert	
Rainbow Creek	Tributary 3	1000	2 Year	3.5	194.66
Rainbow Creek	Tributary 3	1000	5 Year	4.6	194.74
Rainbow Creek	Tributary 3	1000	10 Year	5.4	194.79
Rainbow Creek	Tributary 3	1000	25 Year	6.3	194.84
Rainbow Creek	Tributary 3	1000	50 Year	7.1	194.85
Rainbow Creek	Tributary 3	1000	100 Year	7.8	194.89
Rainbow Creek	Tributary 3	1000	H. Hazel	10.3	194.96
Rainbow Creek	Tributary 2	1248.5	2 Year	1.3	192.65
Rainbow Creek	Tributary 2	1248.5	5 Year	1.7	192.68
Rainbow Creek	Tributary 2	1248.5	10 Year	2.0	192.70
Rainbow Creek	Tributary 2	1248.5	25 Year	2.4	192.73
Rainbow Creek	Tributary 2	1248.5	50 Year	2.7	192.75
Rainbow Creek	Tributary 2	1248.5	100 Year	3.0	192.76
Rainbow Creek	Tributary 2	1248.5	H. Hazel	3.9	192.73
Rainbow Creek	Tributary 2	1169.5	2 Year	1.3	191.78
Rainbow Creek	Tributary 2	1169.5	5 Year	1.7	191.81
Rainbow Creek	Tributary 2	1169.5	10 Year	2.0	191.76
Rainbow Creek	Tributary 2	1169.5	25 Year	2.4	191.75
Rainbow Creek	Tributary 2	1169.5	50 Year	2.7	191.75

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Rainbow Creek	Tributary 2	1169.5	100 Year	3.0	191.76
Rainbow Creek	Tributary 2	1169.5	H. Hazel	3.9	191.90
Rainbow Creek	Tributary 2	1081.8	2 Year	1.3	191.20
Rainbow Creek	Tributary 2	1081.8	5 Year	1.7	191.23
Rainbow Creek	Tributary 2	1081.8	10 Year	2.0	191.34
Rainbow Creek	Tributary 2	1081.8	25 Year	2.4	191.46
Rainbow Creek	Tributary 2	1081.8	50 Year	2.7	191.55
Rainbow Creek	Tributary 2	1081.8	100 Year	3.0	191.62
Rainbow Creek	Tributary 2	1081.8	H. Hazel	3.9	191.90
Rainbow Creek	Tributary 2	1044	2 Year	1.3	191.09
Rainbow Creek	Tributary 2	1044	5 Year	1.7	191.25
Rainbow Creek	Tributary 2	1044	10 Year	2.0	191.34
Rainbow Creek	Tributary 2	1044	25 Year	2.4	191.46
Rainbow Creek	Tributary 2	1044	50 Year	2.7	191.55
Rainbow Creek	Tributary 2	1044	100 Year	3.0	191.62
Rainbow Creek	Tributary 2	1044	H. Hazel	3.9	191.90
Rainbow Creek	Tributary 2	1031		Culvert	
Rainbow Creek	Tributary 2	1019	2 Year	1.3	190.11
Rainbow Creek	Tributary 2	1019	5 Year	1.7	190.13
Rainbow Creek	Tributary 2	1019	10 Year	2.0	190.14
Rainbow Creek	Tributary 2	1019	25 Year	2.4	190.15
Rainbow Creek	Tributary 2	1019	50 Year	2.7	190.16
Rainbow Creek	Tributary 2	1019	100 Year	3.0	190.17
Rainbow Creek	Tributary 2	1019	H. Hazel	3.9	190.19
Rainbow Creek	Tributary 2	1000	2 Year	1.3	189.69
Rainbow Creek	Tributary 2	1000	5 Year	1.7	189.71
Rainbow Creek	Tributary 2	1000	10 Year	2.0	189.73
Rainbow Creek	Tributary 2	1000	25 Year	2.4	189.74
Rainbow Creek	Tributary 2	1000	50 Year	2.7	189.75
Rainbow Creek	Tributary 2	1000	100 Year	3.0	189.76
Rainbow Creek	Tributary 2	1000	H. Hazel	3.9	189.80

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Rainbow Creek	Tributary 2	893	2 Year	1.3	187.37
Rainbow Creek	Tributary 2	893	5 Year	1.7	187.50
Rainbow Creek	Tributary 2	893	10 Year	2.0	187.57
Rainbow Creek	Tributary 2	893	25 Year	2.4	187.65
Rainbow Creek	Tributary 2	893	50 Year	2.7	187.71
Rainbow Creek	Tributary 2	893	100 Year	3.0	187.77
Rainbow Creek	Tributary 2	893	H. Hazel	3.9	187.94
Rainbow Creek	Tributary -DS02	1968	2 Year	17.1	194.65
Rainbow Creek	Tributary -DS02	1968	5 Year	23.0	194.73
Rainbow Creek	Tributary -DS02	1968	10 Year	27.1	194.77
Rainbow Creek	Tributary -DS02	1968	25 Year	32.2	194.82
Rainbow Creek	Tributary -DS02	1968	50 Year	36.3	194.82
Rainbow Creek	Tributary -DS02	1968	100 Year	40.0	194.85
Rainbow Creek	Tributary -DS02	1968	H. Hazel	53.5	194.91
Rainbow Creek	Tributary -DS02	1892.9	2 Year	17.1	194.65
Rainbow Creek	Tributary -DS02	1892.9	5 Year	23.0	194.73
Rainbow Creek	Tributary -DS02	1892.9	10 Year	27.1	194.77
Rainbow Creek	Tributary -DS02	1892.9	25 Year	32.2	194.82
Rainbow Creek	Tributary -DS02	1892.9	50 Year	36.3	194.82
Rainbow Creek	Tributary -DS02	1892.9	100 Year	40.0	194.86
Rainbow Creek	Tributary -DS02	1892.9	H. Hazel	53.5	194.92
Rainbow Creek	Tributary -DS02	1720.5	2 Year	17.1	194.65
Rainbow Creek	Tributary -DS02	1720.5	5 Year	23.0	194.73
Rainbow Creek	Tributary -DS02	1720.5	10 Year	27.1	194.77
Rainbow Creek	Tributary -DS02	1720.5	25 Year	32.2	194.82
Rainbow Creek	Tributary -DS02	1720.5	50 Year	36.3	194.82
Rainbow Creek	Tributary -DS02	1720.5	100 Year	40.0	194.86
Rainbow Creek	Tributary -DS02	1720.5	H. Hazel	53.5	194.92
Rainbow Creek	Tributary -DS02	1686		Culvert	
Rainbow Creek	Tributary -DS02	1663.8	2 Year	17.1	192.19
Rainbow Creek	Tributary -DS02	1663.8	5 Year	23.0	192.31
Rainbow Creek	Tributary -DS02	1663.8	10 Year	27.1	192.38

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Rainbow Creek	Tributary -DS02	1663.8	25 Year	32.2	192.45
Rainbow Creek	Tributary -DS02	1663.8	50 Year	36.3	192.50
Rainbow Creek	Tributary -DS02	1663.8	100 Year	40.0	192.55
Rainbow Creek	Tributary -DS02	1663.8	H. Hazel	53.5	192.68
Rainbow Creek	Tributary -DS02	1495.8	2 Year	17.1	190.75
Rainbow Creek	Tributary -DS02	1495.8	5 Year	23.0	190.85
Rainbow Creek	Tributary -DS02	1495.8	10 Year	27.1	190.90
Rainbow Creek	Tributary -DS02	1495.8	25 Year	32.2	190.97
Rainbow Creek	Tributary -DS02	1495.8	50 Year	36.3	191.02
Rainbow Creek	Tributary -DS02	1495.8	100 Year	40.0	191.06
Rainbow Creek	Tributary -DS02	1495.8	H. Hazel	53.5	191.21
Rainbow Creek	Tributary -DS02	1210	2 Year	17.1	188.99
Rainbow Creek	Tributary -DS02	1210	5 Year	23.0	189.10
Rainbow Creek	Tributary -DS02	1210	10 Year	27.1	189.17
Rainbow Creek	Tributary -DS02	1210	25 Year	32.2	189.25
Rainbow Creek	Tributary -DS02	1210	50 Year	36.3	189.30
Rainbow Creek	Tributary -DS02	1210	100 Year	40.0	189.35
Rainbow Creek	Tributary -DS02	1210	H. Hazel	53.5	189.49
Rainbow Creek	Tributary -DS02	1000	2 Year	17.1	187.59
Rainbow Creek	Tributary -DS02	1000	5 Year	23.0	187.66
Rainbow Creek	Tributary -DS02	1000	10 Year	27.1	187.71
Rainbow Creek	Tributary -DS02	1000	25 Year	32.2	187.76
Rainbow Creek	Tributary -DS02	1000	50 Year	36.3	187.80
Rainbow Creek	Tributary -DS02	1000	100 Year	40.0	187.83
Rainbow Creek	Tributary -DS02	1000	H. Hazel	53.5	187.93
West Rainbow Creek	Tributary 1	1382	2 Year	4.3	191.89
West Rainbow Creek	Tributary 1	1382	5 Year	8.8	192.00
West Rainbow Creek	Tributary 1	1382	10 Year	11.7	192.05
West Rainbow Creek	Tributary 1	1382	25 Year	13.4	192.08
West Rainbow Creek	Tributary 1	1382	50 Year	14.0	192.09
West Rainbow Creek	Tributary 1	1382	100 Year	18.6	192.05
West Rainbow Creek	Tributary 1	1382	H. Hazel	52.6	192.34

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
West Rainbow Creek	Tributary 1	1202.3	2 Year	4.3	189.84
West Rainbow Creek	Tributary 1	1202.3	5 Year	8.8	189.95
West Rainbow Creek	Tributary 1	1202.3	10 Year	11.7	190.00
West Rainbow Creek	Tributary 1	1202.3	25 Year	13.4	190.03
West Rainbow Creek	Tributary 1	1202.3	50 Year	14.0	190.04
West Rainbow Creek	Tributary 1	1202.3	100 Year	18.6	190.44
West Rainbow Creek	Tributary 1	1202.3	H. Hazel	52.6	191.58
West Rainbow Creek	Tributary 1	1063	2 Year	4.3	189.21
West Rainbow Creek	Tributary 1	1063	5 Year	8.8	189.66
West Rainbow Creek	Tributary 1	1063	10 Year	11.7	189.92
West Rainbow Creek	Tributary 1	1063	25 Year	13.4	190.05
West Rainbow Creek	Tributary 1	1063	50 Year	14.0	190.10
West Rainbow Creek	Tributary 1	1063	100 Year	18.6	190.45
West Rainbow Creek	Tributary 1	1063	H. Hazel	52.6	191.58
West Rainbow Creek	Tributary 1	1020		Culvert	
West Rainbow Creek	Tributary 1	1000	2 Year	4.3	188.32
West Rainbow Creek	Tributary 1	1000	5 Year	8.8	188.43
West Rainbow Creek	Tributary 1	1000	10 Year	11.7	188.48
West Rainbow Creek	Tributary 1	1000	25 Year	13.4	188.50
West Rainbow Creek	Tributary 1	1000	50 Year	14.0	188.51
West Rainbow Creek	Tributary 1	1000	100 Year	18.6	188.57
West Rainbow Creek	Tributary 1	1000	H. Hazel	52.6	188.87
West Rainbow Creek	Tributary 1	905	2 Year	4.3	187.17
West Rainbow Creek	Tributary 1	905	5 Year	8.8	187.27
West Rainbow Creek	Tributary 1	905	10 Year	11.7	187.33
West Rainbow Creek	Tributary 1	905	25 Year	13.4	187.41
West Rainbow Creek	Tributary 1	905	50 Year	14.0	187.47
West Rainbow Creek	Tributary 1	905	100 Year	18.6	187.52
West Rainbow Creek	Tributary 1	905	H. Hazel	52.6	187.71
West Rainbow Creek	Tributary 1	835	2 Year	4.3	187.18
West Rainbow Creek	Tributary 1	835	5 Year	8.8	187.27
West Rainbow Creek	Tributary 1	835	10 Year	11.7	187.33

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
West Rainbow Creek	Tributary 1	835	25 Year	13.4	187.41
West Rainbow Creek	Tributary 1	835	50 Year	14.0	187.46
West Rainbow Creek	Tributary 1	835	100 Year	18.6	187.51
West Rainbow Creek	Tributary 1	835	H. Hazel	52.6	187.68
Rainbow Creek	Tributary -DS01	1087	2 Year	18.4	187.30
Rainbow Creek	Tributary -DS01	1087	5 Year	24.7	187.41
Rainbow Creek	Tributary -DS01	1087	10 Year	29.2	187.48
Rainbow Creek	Tributary -DS01	1087	25 Year	34.6	187.56
Rainbow Creek	Tributary -DS01	1087	50 Year	39.0	187.62
Rainbow Creek	Tributary -DS01	1087	100 Year	43.0	187.67
Rainbow Creek	Tributary -DS01	1087	H. Hazel	57.4	187.84
Rainbow Creek	Tributary -DS01	1060.7	2 Year	18.4	187.27
Rainbow Creek	Tributary -DS01	1060.7	5 Year	24.7	187.38
Rainbow Creek	Tributary -DS01	1060.7	10 Year	29.2	187.45
Rainbow Creek	Tributary -DS01	1060.7	25 Year	34.6	187.53
Rainbow Creek	Tributary -DS01	1060.7	50 Year	39.0	187.59
Rainbow Creek	Tributary -DS01	1060.7	100 Year	43.0	187.64
Rainbow Creek	Tributary -DS01	1060.7	H. Hazel	57.4	187.81
Rainbow Creek	Tributary -DS01	1000	2 Year	18.4	187.11
Rainbow Creek	Tributary -DS01	1000	5 Year	24.7	187.18
Rainbow Creek	Tributary -DS01	1000	10 Year	29.2	187.23
Rainbow Creek	Tributary -DS01	1000	25 Year	34.6	187.29
Rainbow Creek	Tributary -DS01	1000	50 Year	39.0	187.34
Rainbow Creek	Tributary -DS01	1000	100 Year	43.0	187.38
Rainbow Creek	Tributary -DS01	1000	H. Hazel	57.4	187.59
Rainbow Creek	Rainbow C-DS01	2904	2 Year	39.1	187.07
Rainbow Creek	Rainbow C-DS01	2904	5 Year	53.8	187.12
Rainbow Creek	Rainbow C-DS01	2904	10 Year	63.9	187.16
Rainbow Creek	Rainbow C-DS01	2904	25 Year	77.2	187.20
Rainbow Creek	Rainbow C-DS01	2904	50 Year	87.4	187.24
Rainbow Creek	Rainbow C-DS01	2904	100 Year	97.1	187.27
Rainbow Creek	Rainbow C-DS01	2904	H. Hazel	137.0	187.41

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Rainbow Creek	Rainbow C-DS01	2778.2	2 Year	39.1	186.21
Rainbow Creek	Rainbow C-DS01	2778.2	5 Year	53.8	186.34
Rainbow Creek	Rainbow C-DS01	2778.2	10 Year	63.9	186.43
Rainbow Creek	Rainbow C-DS01	2778.2	25 Year	77.2	186.52
Rainbow Creek	Rainbow C-DS01	2778.2	50 Year	87.4	186.59
Rainbow Creek	Rainbow C-DS01	2778.2	100 Year	97.1	186.66
Rainbow Creek	Rainbow C-DS01	2778.2	H. Hazel	137.0	186.88
Rainbow Creek	Rainbow C-DS01	2576	2 Year	39.1	185.63
Rainbow Creek	Rainbow C-DS01	2576	5 Year	53.8	185.76
Rainbow Creek	Rainbow C-DS01	2576	10 Year	63.9	185.84
Rainbow Creek	Rainbow C-DS01	2576	25 Year	77.2	185.93
Rainbow Creek	Rainbow C-DS01	2576	50 Year	87.4	186.00
Rainbow Creek	Rainbow C-DS01	2576	100 Year	97.1	186.05
Rainbow Creek	Rainbow C-DS01	2576	H. Hazel	137.0	186.24
Rainbow Creek	Rainbow C-DS01	2474.2	2 Year	39.1	185.06
Rainbow Creek	Rainbow C-DS01	2474.2	5 Year	53.8	185.13
Rainbow Creek	Rainbow C-DS01	2474.2	10 Year	63.9	185.17
Rainbow Creek	Rainbow C-DS01	2474.2	25 Year	77.2	185.23
Rainbow Creek	Rainbow C-DS01	2474.2	50 Year	87.4	185.27
Rainbow Creek	Rainbow C-DS01	2474.2	100 Year	97.1	185.31
Rainbow Creek	Rainbow C-DS01	2474.2	H. Hazel	137.0	185.46
Rainbow Creek	Rainbow C-DS01	2215.5	2 Year	39.1	183.36
Rainbow Creek	Rainbow C-DS01	2215.5	5 Year	53.8	183.49
Rainbow Creek	Rainbow C-DS01	2215.5	10 Year	63.9	183.56
Rainbow Creek	Rainbow C-DS01	2215.5	25 Year	77.2	183.65
Rainbow Creek	Rainbow C-DS01	2215.5	50 Year	87.4	183.71
Rainbow Creek	Rainbow C-DS01	2215.5	100 Year	97.1	183.77
Rainbow Creek	Rainbow C-DS01	2215.5	H. Hazel	137.0	183.99
Rainbow Creek	Rainbow C-DS01	1547.1	2 Year	39.1	182.32
Rainbow Creek	Rainbow C-DS01	1547.1	5 Year	53.8	182.44
Rainbow Creek	Rainbow C-DS01	1547.1	10 Year	63.9	182.52
Rainbow Creek	Rainbow C-DS01	1547.1	25 Year	77.2	182.61
Rainbow Creek	Rainbow C-DS01	1547.1	50 Year	87.4	182.68

River	Reach	River Sta	Profile	Flowrate	Water Level
				(m3/s)	(m)
Rainbow Creek	Rainbow C-DS01	1547.1	100 Year	97.1	182.73
Rainbow Creek	Rainbow C-DS01	1547.1	H. Hazel	137.0	182.95
Rainbow Creek	Rainbow C-DS01	1258.9	2 Year	39.1	180.68
Rainbow Creek	Rainbow C-DS01	1258.9	5 Year	53.8	180.81
Rainbow Creek	Rainbow C-DS01	1258.9	10 Year	63.9	180.89
Rainbow Creek	Rainbow C-DS01	1258.9	25 Year	77.2	180.98
Rainbow Creek	Rainbow C-DS01	1258.9	50 Year	87.4	181.04
Rainbow Creek	Rainbow C-DS01	1258.9	100 Year	97.1	181.09
Rainbow Creek	Rainbow C-DS01	1258.9	H. Hazel	137.0	181.29
Rainbow Creek	Rainbow C-DS01	1092.1	2 Year	39.1	180.10
Rainbow Creek	Rainbow C-DS01	1092.1	5 Year	53.8	180.23
Rainbow Creek	Rainbow C-DS01	1092.1	10 Year	63.9	180.30
Rainbow Creek	Rainbow C-DS01	1092.1	25 Year	77.2	180.39
Rainbow Creek	Rainbow C-DS01	1092.1	50 Year	87.4	180.45
Rainbow Creek	Rainbow C-DS01	1092.1	100 Year	97.1	180.51
Rainbow Creek	Rainbow C-DS01	1092.1	H. Hazel	137.0	180.72
Rainbow Creek	Rainbow C-DS01	1000.108	2 Year	39.1	179.91
Rainbow Creek	Rainbow C-DS01	1000.108	5 Year	53.8	180.03
Rainbow Creek	Rainbow C-DS01	1000.108	10 Year	63.9	180.09
Rainbow Creek	Rainbow C-DS01	1000.108	25 Year	77.2	180.17
Rainbow Creek	Rainbow C-DS01	1000.108	50 Year	87.4	180.22
Rainbow Creek	Rainbow C-DS01	1000.108	100 Year	97.1	180.27
Rainbow Creek	Rainbow C-DS01	1000.108	H. Hazel	137.0	180.46
Rainbow Creek	Rainbow C-DS01	1000	2 Year	39.1	179.10
Rainbow Creek	Rainbow C-DS01	1000	5 Year	53.8	179.19
Rainbow Creek	Rainbow C-DS01	1000	10 Year	63.9	179.24
Rainbow Creek	Rainbow C-DS01	1000	25 Year	77.2	179.30
Rainbow Creek	Rainbow C-DS01	1000	50 Year	87.4	179.35
Rainbow Creek	Rainbow C-DS01	1000	100 Year	97.1	179.39
Rainbow Creek	Rainbow C-DS01	1000	H. Hazel	137.0	179.55