REGIONAL STORMWATER MANAGEMENT PLAN
WATERCOURSE 7, 8, 9 & 10

CRAIGLEITH
RESIDENTIAL DEVELOPMENT
TOWN OF THE BLUE MOUNTAINS

PARKBRIDGE LIFESTYLE COMMUNITIES INC. &
MACPHERSON BUILDERS LTD.

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1.0 INTRODUCTION

CF Crozier & Associates Inc. has been retained by Parkbridge Lifestyle Communities Inc. (Parkbridge) and MacPherson Builders Ltd. (MacPherson) to complete a Regional Stormwater Management Plan to supplement the development applications for both developers in the Craigleith area.

The Town of The Blue Mountains has requested that a Regional Stormwater Management Plan be prepared to provide a comprehensive analysis of the local watercourses, as the local watercourses experience significant flow interactions with each other. This report has been prepared to document the existing flooding conditions of the local watercourses and analyze possible flooding mitigation options and is based on the Terms of Reference developed in consultation with the Town of the Blue Mountains. The Terms of Reference is included in **Appendix A**.

This study will build upon the findings the Grey Sauble Conservation Authorities Subwatershed Study (1993) recognizing the historic identification of a flood damage center for Watercourse 7 at its outlet downstream of Highway 26.

1.1 Study Area

This report will analyse four watercourses located within the study area of the Craigleith Camperdown Subwatershed Study (prepared for GSCA by Gore & Storrie, November 1993). The watercourse naming convention has been adopted in this report to be consistent with that study. Watercourses 7, 8, 9 and 10 and their associated drainage areas are the subject of this report.

The study area has been outlined in **Appendix B**.

1.2 Study Purpose

Numerous residential properties are located adjacent to Watercourses 7, 8, 9 and 10 in the area of Highway 26 and Lakeshore Road. Some of these residential properties have been identified to experience historical flooding from these watercourses during high rainfall events and spring freshet conditions.

As such, the Town has requested that a comprehensive analysis of Watercourses 7, 8, 9 and 10 be prepared to assess the existing flooding conditions of the watercourses and make recommendations to improve upon the existing flood conveyance infrastructure downstream of the Parkbridge Craigleith Ridge and MacPherson Home Farm developments.

1.3 Proposed Developments

This report has been prepared to supplement the development applications of two proposed developments in the Craigleith area, the Parkbridge Craigleith Ridge Development and MacPherson Home Farm Development.

<u>Parkbridge Craigleith Ridge</u>

The Parkbridge Craigleith Ridge development is comprised of three separate property parcels. The largest 25 ha (62 acre) property is located south of Lakeshore Road and is legally described

as Plan 529, Part Lot 161, Town of The Blue Mountains, County of Grey. A smaller 1.2 ha (3 acre) property is located south of Lakeshore Road and is legally described as Plan 529, Lot 172 and Part Lot 173 as Part 2 RP 16R-6640, Town of The Blue Mountains, County of Grey. Finally, the third parcel is an approximately 0.6ha undeveloped parcel located north of Lakeshore Road, bounded by Lakeshore Road and Georgian Trail. This parcel is legally described as Plan 529 Part Lot 169, Town of The Blue Mountains, County of Grey.

The Craigleith Ridge development traverses the Nipissing Ridge with a portion of development lands on the upper terrace above the ridge, as well as below the ridge on the lower terrace lands.

The primary drainage features of the site is Watercourse 9 that bisects the site. Watercourse 7 bounds the site at the northeast and Watercourse 10 bounds the site at the west. Watercourse 8 also crosses the site in an ephemeral fashion, as there is no defined valley feature or channel present on the lower terrace lands.

The development concept reflects a total of 211 residential units comprised of a mixture of single detached, townhome and loft townhome units as well as a series of private roads, environmental, open space and stormwater management facility areas. Parkbridge proposes the subject community be developed as a land lease community geared towards the adult lifestyle and recreation markets. In December 2016 a Functional Servicing and Stormwater Management Report was prepared by Crozier in support of planning applications for the Craigleith Ridge development.

MacPherson Home Farm Development

The MacPherson Home Farm development is approximately 60.25ha and is located at the top of Nipissing Ridge to the southeast of the proposed Craigleith Ridge development. The lands are legally described as Lots 2, 3, 4, 5, 6, and 7 – Plan 555, Part of Lot 159 – Plan 529, Part of Lot 20 – Concession 2, and Part of Helen Street ROW – Plan 555.

Home Farm is bounded by County Road 19 to the west, Nipissing Ridge to the north and east, and Tyrolean Lane to the south. The primary drainage features consist of Watercourse 7 crossing the site, and the upper tributary reaches of Watercourse 8 which begins on the Home Farm site.

The Home Farm development is planned to contain 283 residential units that consist of 132 townhouse units and 151 detached units. In February 2015 a Functional Servicing & Stormwater Management Report was prepared by Higgins Engineering Ltd. in support of planning applications for the proposed development.

2.0 BACKGROUND & PREVIOUS STUDY

Watercourses 7, 8, 9 and 10 were first analyzed as part of the Craigleith Camperdown Subwatershed Study (GSCA, 1993). That report analyzed 35 watercourses in the Craigleith/Camperdown area and examined hydrology, hydrogeology, and floodplain characteristics along with other watershed elements.

Subsequent to the Craigleith Camperdown Subwatershed Study, various hydraulic investigations of the Watercourses 7, 8, 9 & 10 have subsequently been undertaken to support development applications within these subwatershed boundaries.

Watercourse 7

Watercourse 7 has been previously assessed by Crozier in support of the Eden Oak Trailshead Development which is located immediately east of the Craigleith Ridge site at the base of the Nipissing Ridge. Hydraulic analysis was completed to determine the development setbacks to the development and summarized in the Functional Servicing and Stormwater Management Report by Crozier (April 2012).

Higgins Engineering Ltd. studied the Watercourse 7 subwatershed further in support of the existing Orchards subdivision located upstream of both the Craigleith Ridge and Home Farm developments. To facilitate subdivision grading within the Orchards development, 8.91ha of lands were diverted from the Watercourse 7 Subwatershed to the Watercourse 9 Subwatershed.

In support of the proposed Home Farm Development, Higgins Engineering Ltd. completed a Functional Servicing and Stormwater Management Report (February 2015) which outlined the pre and post development hydrology of Watercourse 7 with respect to the Home Farm development. Higgins Engineering has proposed to divert 8.48ha from subwatershed 9 to subwatershed 7 to facilitate the subdivision grading of Home Farm, and to restore the subwatershed areas between Watercourse 9 and Watercourse 7. Once completed with the development of the Home Farm lands, the land swap between Subwatershed 7 and Subwatershed 9 will remain at their historic drainage areas.

Crozier has completed various site investigations and surveys to inventory the capacity of the downstream drainage infrastructure (culverts, channels, outlets) of Watercourse 7 between Lakeshore Road and the outlet to Georgian Bay.

Watercourse 8

Given the poorly defined channel and lack of valley feature as well as the relatively small catchment area, Watercourse 8 has not been the subject of considerable formalized study.

Crozier has completed various site investigations and surveys to inventory the capacity of the downstream drainage infrastructure (culverts, channels, outlets) of Watercourse 8 between Lakeshore Road and the outlet to Georgian Bay.

Watercourse 9

Watercourse 9 has been previously assessed by Crozier in 2008 for the Craigleith Ridge development property on behalf of a previous owner. This work was used in support of updates to the planning designations for the subject lands. Crozier completed natural hazards assessments, functional servicing and stormwater management designs for the previous concept plan on the property. The results of the natural hazards assessments were presented and refined with the Town and the Grey Sauble Conservation Authority (GSCA). Through this process, a settlement was reached with the Town and the GSCA that confirmed the hazard land limits for the subject lands.

Watercourse 9, upstream of Grey Road 19, has been assessed by Higgins in support of The Orchards development located west of County Road 19 upstream of the Craigleith Ridge lands. A subwatershed hydrologic model for the catchments upstream of County Road 19 was completed and formalized in the Stormwater Management Design Brief and Functional Servicing Report (Higgins, 2005).

The previous study of Watercourse 9 was expanded upon by Crozier in 2016 to support the planning applications for the proposed Parkbridge Craigleith Ridge development. Summarized in a Functional Servicing and Stormwater Management Report (December 2016), the Parkbridge development proposes Watercourse 9 as the primary outlet for the development lands. A hydrologic model of subwatershed 9 was constructed to guide the design of the proposed stormwater management facilities to achieve "pre to post" peak flow attenuation.

Watercourse 10

The Watercourse 10 watershed was studied in detail by R.J. Burnside & Associates in a report titled Stormwater Management Study for Craigleith Ski Club (2004) to provide future criteria for development at the base of the Craigleith Ski Club.

In 2009, Crozier assessed Watercourse 10 in support of the Bannerman Development located south of Lakeshore Road and directly west of the Craigleith Ride site on the lower terrace lands. At the request of the MTO, Crozier completed a subwatershed-scale hydrologic assessment corresponding hydraulic assessment of the existing Lakeshore Road and Highway 26 culverts for capacity and recommendations of improvements.

3.0 EXISTING CONDITIONS & CURRENT FLOODING CONDITIONS

Watercourses 7, 8, 9 and 10 and the associated watersheds are primarily characterised by the Niagara Escarpment, Nipissing Ridge, and relatively flat lands between the Nipissing Ridge and Georgian Bay.

Downstream of the Nipissing Ridge, all four watercourses lack valley features and exhibit varying levels of channel definition and historic channel alteration. Due to these channel characteristics all four watercourses often experience flows exceeding the channel capacity which spill into the adjacent watershed / lands generally towards Georgian Bay.

The surficial soils within the study area are characterized by the Soil Survey of Grey County (1954). Below the Nipissing Ridge the soil is primarily Granby sand which is generally considered as hydrologic soil Group B and is poor draining. The soils traversing the Nipissing Ridge are characterized as Waterloo sandy loam (Group A) and is considered to be well draining. Above the Nipissing Ridge the soils are characterized as Kemble silty clay (Group C) and is considered to be imperfectly drained.

3.1 Watercourse 7

Watercourse 7 has a catchment area of approximately 180 ha and originates on the Niagara Escarpment. Watercourse 7 traverses the south and east side of the study area passing through the Home Farm lands and along the east side of the Craigleith Ridge lands.

Above the Nipissing Ridge, the drainage patterns are undefined and have varied historically. During high flow events Watercourse 7 has been known to spill out onto the former farm fields of the Home Farm lands which are generally low in relief.

During a site investigation by Crozier staff on March 28, 2016, Watercourse 7 was noted to lack a valley feature across the Home Farm lands and was noted to breach the capacity of the channel and spill in multiple locations. The spill flows observed were generally towards the north of Watercourse 7 to undefined drainage features and large areas of ponding were observed. The low relief of the Home Farm lands was observed and drainage routes were not obvious across the site. It is believed that some spill flow to the north may be directed to Watercourse 8 and possibly spill towards the Parkbridge lands and the residents of Lakeshore Road adjacent Watercourse 8. There are multiple incised ravines that traverse the Nipissing Ridge before converging with the main channel below the ridge. These ravines are believed to drain the large ponding areas on the Home Farm site. A gravel access road was noted to have ditches that conveyed portions of Watercourse 7 spill flows towards the northeast before re-converging with the main channel. Spill flow from these ditches were noted to be redirected to Watercourse 6 to the east. Refer to **Appendix C** for photos from the site investigation.

Below the Nipissing Ridge, Watercourse 7 is contained to a well-defined channel and passes between a number of existing residential units adjacent to Lakeshore Road and Highway 26. The outlet of this watercourse is downstream of Highway 26 and was identified as a flood damage area in the original Craigleith Camperdown Subwatershed Study (GSCA, 1993), due to channel constrictions imposed by existing residences.

The existing conditions peak flows of Watercourse 7 were established by the Grey Sauble Conservation Authority's Subwatershed Study (GSCA, 1993). The watershed hydrology model has

subsequently been refined as development applications have proceeded in the watershed. These flows are summarized in **Table 1** below.

Table 1
Watercourse 7 Existing Hydrologic Flows (GSCA Model)

Location	Return Period	Peak Flow (m^3/s)[1]
Lakeshore Road	2-Year	2.89
	5-Year	3.93
	10-Year	4.95
	25-Year	5.79
	50-Year	6.44
	100-Year	7.39
	Regional	9.72

[1]Peak flows per Craigleith Camperdown Subwatershed Study (GSCA, 1993)

In support of the Eden Oak Development, Crozier completed a hydraulic assessment of Watercourse 7 as it crosses the lower terrace lands. The watercourse 7 hydraulic assessment is summarized in the Functional Servicing & Stormwater Management Report (Crozier, 2012). The hydraulic model was prepared using HEC-RAS modeling software to determine the existing flood conditions of Watercourse 7 and associated development limits for the Eden Oak Development.

The hydraulic assessment identified a spill-flow condition at the base of the Nipissing Ridge at flow rates above 6 m^3/s. It was found that the exceeding flows overtopped the right overbank of Watercourse 7 and spilled eastward into the Eden Oak Lands and ultimately into subwatershed 6. The Eden Oak development applications proceeded under the premise of maintaining this spill flow to Watercourse 6.
Table 2 summarizes the peak flows experienced at Lakeshore Road with and without the spill flow condition at Eden Oak.

Table 2
Watercourse 7 Existing Hydrologic Flows – With Spill Flow

Return Period	Spill Flow at Eden Oak (m^3/s)	Peak Flow at Lakeshore (Spill) (m^3/s)	Peak Flow at Lakeshore (No Spill) (m^3/s)
2-Year	0	2.893	2.89
5-Year	0	3.943	3.94
10-Year	0	4.950	4.95
25-Year	0	5.791	5.79
50-Year	0.442	6.0	6.44
100-Year	1.393	6.0	7.39
Regional	3.720	6.0	9.72

Between the Nipissing Ridge and Georgian Bay, Watercourse 7 has experienced historical flooding affecting various residential properties. In particular, residences upstream Lakeshore Road and downstream of Highway 26 have experienced flooding issues during large storm events

and spring freshet events. Crozier staff have observed the operational conditions of the Watercourse 7 conveyance infrastructure in the Lakeshore Road and Highway 26 area on numerous occasions throughout recent years. Photos from these site investigations are included in **Appendix C** for reference.

Additionally, Crozier has inventoried and assessed the hydraulic capacities of the existing drainage infrastructure on Watercourse 7. The inventory is summarized in **Table 3**.

Table 3
Watercourse 7 Existing Drainage Infrastructure Inventory & Capacities

Location	Description	Maximum Capacity m^3/s	Approx. Return Period	Typical Design Standard
Lakeshore Road	Concrete box culvert (0.95m x 1.9m)	3.8[1]	< 5-Year	25-Year[2]
Georgian Trail	Twin CSP culverts (1.2m diameter)	4.5[1]	< 10-Year	N/A
Highway 26	CSP arch (0.9m x 1.35m)	2.0[1]	< 2-Year	100-Year[3]
Outlet Channel	Channel between dwellings (0.60m x 0.90m)	1.2	< 2-Year	N/A

[1] Maximum capacity of culverts operating under head without overtopping road
[2] Town of the Blue Mountains Design Standard
[3] Ministry of Transportation Design Standard

As outlined in Table 2, there is insufficient capacity of all hydraulic structures on Watercourse 7 based on the typical design criteria. It is noted that the outlet channel is the limiting section of Watercourse 7 as it represents the lowest conveyance capacity. Given the close proximity of the existing residences, 209725 & 209727 Highway 26, this location is highly flood susceptible under existing conditions.

Due to the historical flooding events and poor capacity of the Watercourse 7 outlet channel, there have been improvements previously implemented in an effort to mitigate flooding impacts. These improvements include the construction of a lateral relief pipe located between Highway 26 and Georgian Trail which is intended to divert flow from Watercourse 7 to Watercourse 8. When the Highway 26 culvert reaches capacity the flow is intended to be diverted to Watercourse 8 via the lateral relief structure. The ditch of Highway 26 also serves as additional flow conveyance from Watercourse 7 to Watercourse 8 when the Highway 26 culvert is surcharged.

Table 4 summarizes the existing lateral flow diversion infrastructure and their capacities.

Table 4
Watercourse 7 Existing Flow Diversion Infrastructure Inventory & Capacities

Location	Description	Capacity (m^3/s)
Highway 26	Concrete Arch Pipe (0.9m x 1.5m)	1.5[1]
Highway 26	Highway 26 ditch (0.75m depth, 0.5%)	1.8

[1] Maximum capacity of culvert operating under head without overtopping Highway 26

When accounting for the capacities of the lateral concrete pipe, Highway 26 ditch, and the existing outlet channel, the maximum capacity of the Watercourse 7 conveyance infrastructure is approximately 4.5 m^3/s which equates to less than the 25-Year storm event.

3.2 Watercourse 8

Watercourse 8 is a relatively small subwatershed which originates in the central portion of the Home Farm lands above the Nipissing Ridge. Subwatershed 8 drains approximately 13ha of primarily undeveloped lands consisting of upper terrace lands, the Nipissing Ridge and lower terrace lands. The portion of the subwatershed that is contained within the Home Farm Development consists of the upper terrace lands and is vegetated with a mixture of meadows and trees. Two distinct drainage draws drain the upper terrace and Nipissing Ridge lands, and enter the lower terrace lands. Upon entry to the Craigleith Ridge lands downstream of the ridge, the drainage draws become less defined resulting in runoff being conveyed primarily as sheet flow where it infiltrates back into porous soils and fractures within the shallow bedrock.

Crozier has completed a hydrologic assessment of the Watercourse 8 subcatchment to determine the existing flows experienced at Lakeshore Road.

Table 5
Watercourse 8 Existing Hydrologic Flows (Crozier Hydrologic Model)

Location	Return Period	Peak Flow (m^3/s)[1]
Lakeshore Road	2-Year	0.12
	5-Year	0.25
	10-Year	0.41
	25-Year	0.56
	50-Year	0.65
	100-Year	0.84
	Regional	1.09

Between the Nipissing Ridge and Lakeshore Road, no defined channel exists. Downstream of Lakeshore Road, Watercourse 8 re-develops into a small stream before crossing under Lakeshore Road, Georgian Trail, and Highway 26.

Crozier has inventoried and assessed the hydraulic capacities of the existing drainage infrastructure on Watercourse 8. The inventory is summarized in **Table 6.**

Table 6
Watercourse 8 Existing Drainage Infrastructure Inventory & Capacities

Location	Description	Maximum Capacity m^3/s	Approx. Return Period	Typical Design Standard
Lakeshore Road	CSP culvert (0.35m diameter)	0.11[1]	< 2-Year	25-Year[2]
Georgian Trail	Twin CSP culverts (0.7m diameter)	2.5[1]	< 5-Year	N/A
Highway 26	CSP arch (1.0m x 1.6m)	3.0[1]	> 100-Year	100-Year[3]
Outlet Channel	Channel on Municipally owned land	2.5[4]	< 10-Year	N/A

[1] Maximum capacity of culverts operating under head without overtopping road
[2] Town of the Blue Mountains Design Standard
[3] Ministry of Transportation Design Standard
[4] Outlet channel capacity calculated at 1m depth

3.3 Watercourse 9

Watercourse 9 traverses the Parkbridge Craigleith Ridge Development from Grey Road 19 to Lakeshore Road. Watercourse 9 originates within the tablelands above the Escarpment, and extends across the escarpment face and the subject lands to Georgian Bay.

The portion of the Watercourse 9 watershed, upstream of Grey Road 19 was examined by Higgins Engineering Ltd. in a report titled The Orchards- SWM Implementation Report (Higgins, 2005) to support the proposed The Orchards residential development. Higgins concluded that approximately 116.6 ha of tablelands, escarpment face and development lands contribute to Watercourse 9 upstream of Grey Road 19. Watercourse 9 crosses Grey Road 19 via culvert before entering the Craigleith Ridge development. A portion of the drainage from the Orchards development is conveyed through an existing stormwater management pond before discharging to Watercourse 9 upstream of Grey Road 19.

Through the Parkbridge Development, the Watercourse 9 valley corridor bisects the upper and lower terrace lands into west and east portions. Watercourse 9 is located within a deeply incised ravine across the upper terrace lands and within a defined channel but unconfined valley setting across the lower terrace lands. The upper terrace lands of the Parkbridge development is predominately vegetated with cultural meadows and clusters of trees with series of well-defined drainage draws that discharge to Watercourse 9.

Vegetation across the lower terrace lands of the Parkbridge development consists of cultural meadows and clusters of trees, save and except a more densely treed area located in the vicinity of the toe of the Nipissing Ridge within the western portion of the lower terrace lands.

An existing man made pond is located in the central portion of the Parkbridge site adjacent to the bottom of the ridge. This pond intercepts sheet flow from the ridge and stores the water with a historic outlet structure discharging to Watercourse 9.

Given the lack of a confined valley feature of Watercourse 9 on the lower terrace lands, spill flow escapes the channel overbanks on the east and west side of Watercourse 9 and ponds upstream

of Old Lakeshore Road which is generally low in relief and lacks defined drainage features. This spill flow drains to the south roadside ditches of Old Lakeshore Road which returns the spill water to Watercourse 9. Some of this spill flow to the east is also conveyed to Watercourse 8 and overtops Lakeshore Road in major events.

Downstream of the Nipissing Ridge, Watercourse 9 conveys flows under Lakeshore Road, Georgian Trail, and Highway 26 and ultimately outlets via constructed channel between two dwellings. Downstream of Highway 26 the channel has been constructed with gabion stone and basket walls to Georgian Bay.

In order to support the proposed stormwater strategy for the Parkbridge Development, Crozier completed a hydrologic model of subwatershed 9 existing conditions. The model builds on the hydrologic model of the existing The Orchards development completed by Higgins Engineering covering the upper subwatershed above County Road 19. The existing (pre development) hydrologic flows at Lakeshore Road are summarized in **Table 7**.

Table 7
Watercourse 9 Existing Hydrologic Flows (Crozier Hydrologic Model)

Location	Return Period	Peak Flow (m^3/s)
Lakeshore Road	2-Year	1.37
	5-Year	2.62
	10-Year	4.01
	25-Year	5.26
	50-Year	5.92
	100-Year	7.31
	Regional	9.07

Existing residential buildings are located adjacent to Watercourse 9 downstream of Highway 26. As such, it was requested by the Town to assess the channel hydraulics of Watercourse 9 downstream of Highway 26 as well as through the Parkbridge development. A HEC-RAS hydraulic model was completed by Crozier for Watercourse 9 traversing the Parkbridge development and downstream of Lakeshore Road to the outlet at Georgian Bay. The HEC-RAS model upstream of Lakeshore Road is included in the Functional Servicing and Stormwater Management Report (Crozier, April 2018). The hydraulic model assessing the drainage infrastructure downstream of the Craigleith Ridge development is included in **Appendix D.**

The existing residential buildings are approximately 2.8m from Watercourse 9 on the west and approximately 5 meters to the east. The dwelling on the west was noted to have approximately 0.2-0.4m clearance from ground level to first floor, the dwelling on the east was noted to have approximately 0.6-0.8m clearance from ground level to first floor. As such, flow rates that exceed the channel banks would not necessarily result in residential buildings being flooded as the buildings are higher than the channel banks and extreme flows would be dispersed upstream by overtopping Old Lakeshore Road, the Georgian Trail, and Highway 26.

Table 8
Watercourse 9 Existing Drainage Infrastructure Inventory & Capacities

Location	Description	Maximum Capacity m^3/s	Approx. Return Period	Typical Design Standard
Lakeshore Road	Concrete box culvert (1.55m x 1.05m)	2.6[1]	5-Year	25-Year[2]
Georgian Trail	1.5m diameter CSP culvert	3.7[1]	< 10-Year	N/A
Highway 26	Concrete box culvert (2.45m x 0.9m)	6.0[1]	> 50-Year	100-Year[3]
Outlet Channel	Channel between two existing dwellings	3.5	< 10-Year	N/A

[1] Maximum capacity of culverts operating under head without overtopping road
[2] Town of the Blue Mountains Design Standard
[3] Ministry of Transportation Design Standard

3.4 Watercourse 10

Watercourse 10 is located on the west boundary of the study area. Subwatershed 10 consists of agricultural land across the tablelands, ski hills across the escarpment face, and primarily undeveloped land along the upper and lower terraces of the Nipissing Ridge.

The Watercourse 10 valley corridor is located along the western limits of the lower terrace lands of the Craigleith Ridge Development. As previously stated, along the lower terrace lands, Watercourse 10 is located within a defined channel but unconfined valley setting. An incised drainage draw, located in the western portion of the Craigleith Ridge upper terrace lands, connects into the Watercourse 10 valley corridor.

There are various residential dwellings adjacent to Watercourse 10 downstream of Lakeshore Road on route to Georgian Bay.

Crozier completed a Stormwater Management Implementation Report (2012) supporting the detailed design of the Bannerman Development. Within the report a watershed hydrologic assessment was completed for Watercourse 10. Subsequently, the hydraulic structures for the Lakeshore Road, Georgian Trail, and Highway 26 were assessed for capacity. The results of the hydrologic model and hydraulic capacity assessment are summarized in **Tables 9 & 10,** respectively.

Table 9
Watercourse 10 Existing Hydrologic Flows (Crozier Hydrologic Model)

Location	Return Period	Peak Flow (m^3/s)
Lakeshore Road	2-Year	0.91
	5-Year	1.61
	25-Year	2.97
	100-Year	4.04

Table 10
Summary of Existing Culvert Capacities along Watercourse 10 (Crozier, 2012)

Location	Culvert Dimensions	Maximum Capacity m^3/s	Approx. Return Interval	Typical Design Standard
Lakeshore Road	900 mm Ø CSP	1.4[1]	<5-Year	25-Year[2]
Georgian Trail	750 mm Ø CSP + 900 mm Ø CSP	3.2[1]	25-Year	N/A
Highway 26	1.25 m by 1.50 m Conc. Box	5.0[1]	> 100-Year	100-Year[3]

[1] Maximum capacity of culverts operating under head without overtopping road
[2] Town of the Blue Mountains Design Standard
[3] Ministry of Transportation Design Standard

It is noted that the Georgian Trail and Highway 26 culverts have sufficient capacity to meet typical standards. The Lakeshore Road culvert was noted to be undersized to convey the full 25-Year event. As a result a portion of flow conveyance is split between the Lakeshore Road culvert and overtopping of Lakeshore Road.

4.0 PROPOSED DEVELOPMENT STORMWATER MANAGEMENT STRATEGY & FUTURE CONDITIONS

4.1 Watercourse 7 & 8

4.1.1. Proposed SWM Design - Home Farm Development

In order to address the existing undefined drainage conditions of the Home Farm site as discussed in Section 3.1, Higgins Engineering has proposed to improve the Watercourse 7 drainage channel to safely convey flows through the site and formalize the drainage on the site. This strategy will improve the local drainage conditions which have historically varied and is generally unpredictable. Thus, any spill flow from Watercourse 7 to subwatershed 8 or subwatershed 6 that is currently occurring above the Nipissing Ridge will be captured, which will improve any localized flooding concerns in these adjacent watercourses.

Drainage from developed lands within the Home Farm Subdivision have been proposed to be directed to full quality/quantity control ponds within the development before being discharged to Watercourse 7. To treat stormwater both north and south of Watercourse 7, two Stormwater Management facilities have been provided.

Details of each facility as described in the Higgins Engineering Functional Servicing Report and Stormwater Management Report (2016) have been outlined below:

North Pond
The proposed north stormwater management facility will serve as an end-of-pipe wet pond providing quality, quantity, and erosion control for an area of 11.49ha. The wet pond will be implemented to provide pre to post peak flow attenuation for all storms up to the 100-year.

South Pond
The proposed south stormwater management facility will serve as an end-of-pipe wet pond providing quality, quantity, and erosion control for an area of 9.40ha. The wet pond will be implemented to provide pre to post peak flow attenuation for all storms up to the 100-year.

Infiltration Basin
An infiltration basin is proposed to provide additional stormwater controls for an area of 0.82ha. The infiltration basin is proposed to be sized to store the entire 100-Year storm.

The stormwater management facilities are proposed to achieve pre to post peak flow attenuation up to and including the 100-Year storm which will maintain the existing flow regime of Watercourse 7.

4.1.2. Proposed Development SWM Design - Parkbridge

The Parkbridge site contributes very little drainage areas to Watercourse 7 in the pre-development condition. This has been maintained in the post developed condition as flows from developed portions of the site have been directed toward internal storm sewer and stormwater management facilities that outlet to Watercourse 9. Similarly, the subwatershed 8 area will be reduced in the post development condition due to site grading as developed areas within subwatershed 8 will be directed toward the stormwater management facilities and ultimately Watercourse 9. Subwatershed 8 flows from the upper terrace and the Nipissing Ridge will be conveyed through the Parkbridge site via storm sewer to Lakeshore Road.

4.2 Watercourse 9

4.2.1. Proposed SWM Design – Home Farm Development

As outlined in the Functional Servicing and Stormwater Management Report (Higgins, 2016), the Home Farm development does not propose to contribute any post development runoff to Watercourse 9. Thus, no impacts to Watercourse 9 is expected to result from the Home Farm Development.

4.2.2. Proposed SWM Design – Parkbridge Development

Preliminary hydrologic design was completed and summarized in the Functional Servicing and Stormwater Management Report (Crozier, April 2018).

The proposed Parkbridge SWM facilities have been designed to provide the amount of peak flow quantity control needed in order to maintain or reduce peak flows based on pre-development conditions, with results discussed below. Two Stormwater Management (SWM) Facilities (SWM Facility #1, SWM Facility #2) are proposed to provide quality and quantity control for the Craigleith Ridge development.

SWM Facility #1 is proposed to be constructed from the dugout pond at the base of the Nipissing Ridge and converted into a wetland facility with half of the existing pond being used as the operating SWM facility and half remaining in existing condition. SWM Facility #1 will service the upper terrace lands totaling approximately 3.1ha.

SWM Facility #2 is proposed to be located on the east side of Watercourse 9 and service the majority of the lower terrace lands totaling approximately ha. The pond will be constructed as a wet pond providing quality and quantity control.

A portion of the west lands (2.26ha) will be piped under Watercourse 9 and drain to SWM #2, while a portion of the west lands (1.75ha) drain to Watercourse 9 uncontrolled following stormwater quality treatment.

Post-development hydrologic modeling was completed by Crozier and documented in the Functional Servicing and Stormwater Management Report (Crozier, April 2018). A summary and comparison of pre and post-development peak flows is provided below in **Table 11.**

Table 11
Watercourse 9 Post-Development Hydrologic Flows

Location	Return Period	Peak Flow (m^3/s)		% Difference
		Pre-Development	Post-Development	
Lakeshore Road	2-Year	1.369	1.308	-4.5%
	5-Year	2.617	2.463	-5.9%
	10-Year	4.007	3.730	-6.9%
	25-Year	5.259	5.042	-4.1%
	50-Year	5.917	5.680	-4.0%
	100-Year	7.313	6.976	-4.6%
	Regional	9.070	8.671	-4.4%

As shown above in **Table 11**, post-development peak flows at the Watercourse 9 crossing (and downstream) will be reduced from pre-development levels thereby improving the flood conditions downstream of the subject site, between Lakeshore Road and Georgian Bay.

Further to the reduction of peak flows, comparison of the pre and post development flow hydrographs at Lakeshore Road indicate that the duration of extreme flows (i.e. in excess of the 3.5 m^3/s outlet channel capacity) is not increased, thus, no prolongation of flood conditions will be resulting from the Parkbridge development. The pre and post development flow hydrographs have been included in the Functional Servicing and Stormwater Management Report (Crozier, April 2018).

4.3 Watercourse 10

4.3.1. Proposed SWM Design – Parkbridge Craigleith Ridge Development

Based on the proposed Craigleith Ridge development plan, approximately 0.23ha of drainage area will be redirected from Watercourse 10 to Watercourse 9. Furthermore, no post development drainage areas from the Parkbridge development will discharge to Watercourse 10. As a result, the existing hydrology of Watercourse 10 will be maintained by the Craigleith Ridge development.

4.4 Summary

Under the proposed post-development conditions, the Parkbridge Craigleith Ridge and MacPherson developments will implement on-site stormwater management facilities to maintain existing flow conditions of the receiving watercourses.

The Home Farm development will discharge all development area to Watercourse 7 with no development area from the Home Farm site proposed to discharge to Watercourse 8 or 9.

The Craigleith Ridge development will discharge all development area to Watercourse 7 with no development areas proposed to discharge to Watercourse 7, 8, or 10.

Table 8 outlines the proposed post-development conditions of each subwatershed, outlining the total drainage area of each and the development areas of Home Farm and Craigleith Ridge

developments draining to each watercourse. The subwatershed flow rates are also summarized.

Table 12
Watercourse 9 Post-Development Hydrologic Flows

Watercourse	Drainage Area (ha)	Proposed Craigleith Ridge Developed Area discharging to Watercourse (ha)	Proposed Home Farm Developed Area discharging to Watercourse (ha)	Regional Flow Rate (m^3/s)	100-Year Flow Rate (m^3/s)
7	180.8	0	21.7	7.39	9.72
8	8.2	0	0	1.09	0.84
9	147.9	13.5	0	8.671	6.976
10	61.0	0	0	-	4.04

5.0 FLOOD CONVEYANCE OPPORTUNITIES

Based on the discussion presented in the preceding sections there currently exists various flooding issues with each watercourse including spill flows from Watercourse 7 to 6, Watercourse 7 to 8 and Watercourse 9 to 8. As previously discussed part of the objective of this report is to analysis a comprehensive flood management solution for Watercourse 7, 8, 9, and 10. An evaluation of possible infrastructure improvements to increase conveyance capacity for Watercourses 7, 8 and 9 are presented in section 5.1, 5.2, 5.3 respectively.

5.1 Watercourse 7

As noted earlier, the outlet to Watercourse 7 downstream of Highway 26 passes through a narrow channelized section between two existing residences on private property. Given the constraints of this area it was identified as a flood damage center with the original subwatershed study completed in 1993 (GSCA).

Since that time, efforts to improve the flood susceptible nature of downstream properties have been implemented. These works consist of a flow diversion upstream of Highway 26 which takes flows from Watercourse 7 through an elliptical flow pipe to divert flows from Watercourse 7 westward towards Watercourse 8.

As the downstream reach of Watercourse 7 is within private property and is the most flood susceptible, options to increase conveyance capacities of the multiple roadway and trail crossings upstream would only exacerbate existing issues. Further, the location of existing building structures adjacent the Watercourses effectively limits the opportunities to substantially increase the channel capacities given the proximity of existing structural elements of the buildings.

5.2 Watercourse 8

Watercourse 8 was determined to have good opportunities for increased conveyance of flows to Georgian Bay from upstream areas. Flows from Watercourse 8 upstream of Lakeshore Road are intermittent and conveyed through a small diameter CSP cross culvert. However, downstream of Lakeshore Road both the crossing culverts beneath the Georgian Trail and Highway 26 have considerably larger capacities and no adjacent residential structures. The outlet of Watercourse 8 between Highway 26 and Georgian Bay is also contained within Municipal ownership. There are no existing residences or structures in the vicinity of Watercourse 8 downstream of Lakeshore Road.

Between Lakeshore Road and Highway 26, Watercourse 8 is lowest in elevation compared to Watercourse 7 and Watercourse 9. The existing Lakeshore Road profile falls gradually from Watercourse 9 to Watercourse 8 from the west, as well as from Watercourse 7 to Watercourse 8 from the east. Due to the lower profile of the downstream reaches of Watercourse 8, opportunities for flow diversion from Watercourse 7 and 9 to Watercourse 8 are identified as potential flood relief opportunities.

5.3 Watercourse 9

The outlet of Watercourse 9 passes beneath culverts at Lakeshore Road, the Georgian Trail and Highway 26 as it makes its way towards Georgian Bay. Between Old Lakeshore Road and the Georgian Trail the channel passes through undeveloped lands that have also been purchased by Parkbridge. Downstream of Highway 26 the Watercourse 9 channel passes through private

properties which have historically channelized the Watercourse. This limits opportunities for upsizing of upstream roadway crossings due to the presence of the existing dwellings. However, unlike Watercourse 7 the outlet channelization has maintained a channel width of approximately 2.5m, roughly equivalent to width of the Highway 26 crossing culvert. The channel banks have further been protected from erosion and are constructed of gabion baskets which have been maintained in good repair. Hydraulic analysis of this section has indicated that normal flow, below the 10 year rainfall event are contained within the channel. Flows above the 10 year rainfall event exceed the capacity of the channel, however, the existing structures are setback from the channel with finished floor elevations well above the surrounding ground.

Given the setbacks to the existing Watercourse 9 outlet channel, there is a greater potential for Watercourse 9 outlet improvements as compared to Watercourse 7. Watercourse 9 outlet improvements (i.e. widening, deepening) may be possible to improve the flooding risk of the local private dwellings.

5.4 Watercourse 10

The Parkbridge Craigleith Ridge and MacPherson Home Farm developments do not propose to discharge the stormwater flows of any development areas to Watercourse 10. As a result, the existing flow conditions of Watercourse 10 will be maintained. As discusses in Section 3.4 the downstream drainage infrastructure has sufficient capacity to meet relevant design criteria. As such, further assessment of Watercourse 10 is not included within the subject report.

6.0 PROPOSED FLOOD CONVEYANCE STRATEGY

With the implementation of development on the subject MacPherson and Parkbridge lands exists the opportunity to address and formalize a solution to a number of existing watershed issues. These strategies are outlined further in the respective sections below.

6.1 Watercourse 7 & 8

6.1.1. Proposed Flood Management Strategy

The potential of improving the Watercourse 7 outlet channel was determined to be of low probability given the proximity of the existing residences. Any improvements to the channel dimensions would pose constructability constraints due to limited access and would likely require structural alteration to one or both of the existing residences. As a result, this approach was determined to have a low overall benefit to the regional flooding conditions.

Given the downstream conveyance constraints within Watercourse 7 below Highway 26 and the distinct opportunity to improve capacity in the outlet of Watercourse 8, an examination on the feasibility of directing a majority of flows from Watercourse 7 to Watercourse 8 was undertaken.

A hydraulic assessment of all structures within each watercourse was completed to determine the available capacities and reasonable amount of flow that could be diverted. On the Watercourse 8 system it was determined that the available capacity of the Highway 26 cross culvert of under MTO jurisdiction was approximately 3.0 m^3/s. Additional capacity at this crossing could be obtained by supplementing the capacity with a second culvert barrel or replacement with a larger culvert. However, utilizing overtopping flow conveyance across the Highway would be a much more cost effective alternative.

It was noted that in existing conditions Watercourse 7 overtops Highway 26 when flows greater than 5 m^3/s are experienced. This overtopping is broad and would direct flows to lower yard areas of existing residences downstream of the Highway 26 adjacent Watercourse 7 contributing to the flood risk of these properties. The properties downstream of Highway 26 are lower in elevation than the Highway 26 road profile, thus, any overtopping at this location is not favorable.

Conversely, in the vicinity of Watercourse 8 the overland flow route across the Highway is not constrained by existing residences which are flood susceptible. Flows overtopping Highway 26 adjacent Watercourse 8 would be collected by the downstream roadside ditch and pass through a Municipally owned property where the current outlet channel is located. Further, future residential development areas downstream of the Highway should be constructed higher than the Highway 26 roadway elevation to keep flows out of private property and within the public corridors. The current fill levels of residential properties on both sides of Watercourse 8 are favorable to this approach.

As such, by limiting the flow within Watercourse 7 to the outlet capacity (1.2 m^3/s) and moving the surplus flows of Watercourse 7 to Watercourse 8 would assist with improvements to flood conditions in the general area.

To accomplish this task, a conceptual relief flow channel design was developed to remove extreme flows from Watercourse 7 and its susceptible downstream reaches and divert them to the adjacent Watercourse 8. The proposed diversion consists of a secondary flood relief channel to

be established at the southeast limits of the Parkbridge property and convey flows north and westward toward Watercourse 8. At Lakeshore Road, the southern ditch would be formalized and the existing 350mm diameter CSP cross culvert would be upgraded to allow for more conveyance beneath the roadway while surplus flows would be conveyed overtopping Old Lakeshore Road at the existing low point and directed towards Watercourse 8.

Downstream of Old Lakeshore Road, Watercourse 8 flows through a very narrow section of private property (213 Old Lakeshore Road) that is part of an existing single family lot with the dwelling located east of Watercourse 7. This property owner would need to be consulted regarding the drainage redirection which would benefit them and lessen the flood / erosion risk of the existing residence adjacent to Watercourse 7. Alternatively, the Municipality may consider obtaining the lands to improve drainage conditions to the local area.

Once flows have been redirected to Watercourse 8 they can use the existing conveyance infrastructure and overland flow route to Georgian Bay without impact to surrounding residences. Refer to **Figure 8** for a conceptual design of the flow diversion route. Capacity calculations for various infrastructure components are included in **Appendix D**.

6.2 Watercourse 9

6.2.1. Proposed Flood Management Strategy

As outlined in Section 4.2.2., the proposed stormwater management facilities on the Parkbridge site will reduce the severity and frequency of flows experienced by the downstream channel. The water quantity control criteria for the Parkbridge site will be to match or reduce pre development flows rates for all storms including the Regional Timmins storm so that downstream flooding conditions will be maintained or be improved.

Given the unconfined nature of the existing Watercourse 9 channel and various spill flow conditions across the lower terrace lands of the Craigleith Ridge site, the proposed site grading will confine the spill flow to the Watercourse 9 channel. This has the potential of increasing the peak flows experienced downstream if left unmitigated.

To help mitigate the downstream flooding risk of Watercourse 9, a flow relief channel may be designed to divert flows to Watercourse 8. A channel may be graded between Old Lakeshore Road and Georgian Trail to convey flow to Watercourse 8 within Parkbridge owned lands and the Georgian Trail right-of-way. This channel will capture flows overtopping Lakeshore Road during extreme flow events and convey this flow westward to Watercourse 8. Given the existing culvert inverts there is approximately 0.6% slope from Watercourse 9 invert downstream of Old Lakeshore Road to the Watercourse 8 invert upstream of Georgian Trail. The magnitude of flow diversion chosen is dependent on the preferred level of flood proofing for the downstream channel.

Additional flood mitigation strategies can be employed downstream of Highway 26 where opportunities for outlet channel improvements are present. Assessment of the Watercourse 9 outlet channel configuration determined that a channel widening of 0.5m on both sides will result in a capacity of approximately 6.0m^3/s, which is equivalent to the existing Highway 26 culvert. A widening of 1.0m on both side of the channel will result in a capacity which meets the Regional peak flow (9.1m^3/s). Unlike Watercourse 7, the outlet to Watercourse 9 presents a greater opportunity for improvements due to the available space between existing residential dwellings.

7.0 EVALUATION OF OPTIONS

Based on the preceding discussion and historic correspondence with the Town of the Blue Mountains and Grey Sauble Conservation Authority, Crozier presents four potential flood relief options for the Craigleith Area for consideration.

7.1 Summary of Options

Option 1: Divert all flows exceeding downstream capacities to Watercourse 8

Option 1 proposes to divert all flows in exceedance of Watercourse 7 & 9 capacities to Watercourse 8 outlet. The Watercourse 7 and Watercourse 9 outlets will be maintained in existing conditions with no capacity improvements to either outlet proposed. Flows exceeding Watercourse 7 outlet capacity (1.2 m^3/s) will be diverted to Watercourse 8 via existing lateral relief pipe and proposed flood relief channel upstream of Old Lakeshore Road. Similarly, the flows from Watercourse 9 that are in exceedance of the Watercourse 9 outlet (3.5 m^3/s) will be diverted to Watercourse 8 via proposed flood relief channel between Old Lakeshore Road and Georgian Trail.

The Highway 26 culvert at Watercourse 8 will be upgraded to convey the 100-Year flow, which is approximately 11m^3/s following the proposed flow diversions from Watercourse 7 and 9. The existing Watercourse 8 outlet channel will be widened to accommodate the diverted Regional flow of 15.2 m^3/s. Given the size of the proposed culvert upgrade, an open cut installation would be required on Highway 26.

Refer to **Figure 5** for the Option 3 Drainage Network.

Option 2: Improve Watercourse 9 outlet to 6.0m^3/s, divert remaining flow to Watercourse 8

Option 2 proposes to widen the Watercourse 9 outlet channel to match the culvert capacity of Highway 26 (6.0m^3/s) and divert the remainder to Watercourse 8. The outlet channel widening is to be 0.5m on both sides of the channel.

The diversion channel from Watercourse 9 to Watercourse 8 remains, however, provides a lower flow conveyance capacity due to the increased outlet capacity downstream.

Flows exceeding Watercourse 7 outlet capacity be diverted to Watercourse 8 via existing lateral relief pipe and proposed flood relief channel as in Option 1.

The Highway 26 culvert at Watercourse 8 will be upgraded to convey the 100-Year flow, which is approximately 8.4m^3/s following the proposed flow diversions from Watercourse 7 and 9. The existing Watercourse 8 outlet channel will be widened to accommodate the diverted Regional flow of 12.7m^3/s. Given the size of the proposed culvert upgrade, an open cut installation would be required on Highway 26.

Refer to **Figure 6** for the Option 3 Drainage Network.

Option 3: Improve Watercourse 9 outlet to convey Regional Flow

Under Option 3 it is proposed to widen the Watercourse 9 outlet channel to convey the Regional

flow (9.1m^3/s). No flow from Watercourse 9 will be diverted to Watercourse 8. The Highway 26 culvert at Watercourse 9 will be upgraded to convey the 100-Year flow (7.3m^3/s). The outlet channel widening is to be 1.0m on both sides of the channel.

Flows exceeding Watercourse 7 outlet capacity be diverted to Watercourse 8 via existing lateral relief pipe and proposed flood relief channel as in Option 1 & 2.

The Highway 26 culvert at Watercourse 8 will be upgraded to convey the 100-Year flow, which is approximately 8.4m^3/s following the proposed flow diversion from Watercourse 7. The existing Watercourse 8 outlet channel will be widened to accommodate the diverted Regional flow of 9.3m^3/s. Given the size of the proposed culvert upgrade, an open cut installation would be required on Highway 26.

Refer to **Figure 7** for the Option 3 Drainage Network.

Option 4: Maintain Existing Spill Flow Conditions

Under Option 4 it is proposed to further quantify and maintain existing spill flow characteristics that are currently occurring throughout the study area. It is noted that both the Home Farm and Craigleith Ridge developments have implemented stormwater management facilities as part of the development concepts, which will provide stormwater quantity control to respect the downstream properties.

Spill flows currently occurring from Watercourse 7 to Watercourse 6 (at Eden Oak), Watercourse 7 to Watercourse 8 (at Home Farm), and Watercourse 9 to Watercourse 8 (at Parkbridge) will be maintained and thus, downstream flow conditions will be maintained under existing conditions. Under Option 4, no downstream flow conveyance infrastructure improvements are proposed.

Refer to **Figure 4** for an outline of the Existing Flow Conditions.

7.2 Evaluation of Alternatives

Resulting from the preceding discussion, an evaluation matrix has been produced to help qualify each of the proposed options. **Table 13** outlines the key opportunities and constraints presented by each option.

Table 13
Evaluation Matrix for Options

Option	Opportunities	Constraints
Option 1 Divert all exceeding flows to WC8	• Extreme flows diverted away from existing residential dwellings in flood susceptible areas (WC7 & WC9). • No disturbance to existing private property at WC7 & WC9 outlets. • Minimal Highway 26 disturbance for culvert upgrade (single location at WC8).	• Largest requirement for flood relief channel from WC9 to WC8. • Largest culvert upgrade for Highway 26 at WC8. • Proposed WC 7 relief channel passes through undeveloped private property between Lakeshore Rd. and Georgian Trail. • Size of culvert improvement under Highway necessitates open cut installation.
Option 2 Improve WC 9 outlet to 6.0m^3/s, divert remaining to WC8	• Extreme flows diverted away from existing residential dwellings in flood susceptible areas (WC7 & WC8). • Reduced flows diverted to WC8 decreases culvert upgrade requirement. • Reduced conveyance requirement for flood relief channel from WC9 to WC8. • Minimal Highway 26 disturbance for culvert upgrade (single location at WC8).	• Requires disturbance to private property at WC9 outlet. • Proposed WC 7 relief channel passes through undeveloped private property between Lakeshore Rd. and Georgian Trail. • Size of culvert improvement under Highway necessitates open cut installation.
Option 3 Improve WC 9 outlet to convey Regional Flow	• Extreme flows diverted away from existing residential dwellings in flood susceptible areas (WC7). • Reduced flows diverted to WC8 decreases culvert upgrade requirement. • Eliminates flood relief channel from WC9 to WC8.	• Requires disturbance to private property at WC9 outlet. • Requires most instances of Highway 26 disturbance for culvert upgrades (at WC8 & WC9). • Size of culvert improvement under Highway necessitates open cut installation.
Option 4 Maintain Existing Spill Flow Conditions	• No downstream improvements to Lakeshore Road, Highway 26, or outlets (maintain existing).	• Does not improve to existing flooding conditions downstream.

8.0 CONCLUSIONS & RECOMMENDATIONS

The analysis presented in this report outlines the existing flooding conditions as wells as possible opportunities and options to address the existing flooding conditions. Based on this analysis, our conclusion and recommendations include the following.

- Conveyance of flows from Watercourse 7 to 8 and Watercourse 9 to 8 should continue to occur, by way of designed channels or maintaining existing spill flows.
- Existing Infrastructure should be improved between the Base of the Nippissing Ridge and Georgian Bay to improve conveyance capacities.
- Both the Parkbridge and Home Farm developments should continue to provide post-development to pre-development quantity control to avoid increasing downstream flows.
- Option 1 is the recommended alternative as Watercourse 8 was deemed to have the highest opportunity for improving conveyance infrastructure and can accommodate high flows from Watercourse 7 and 9. Furthermore Option 1 involves the least disturbance to private land owners.
- As flood waters are conveyed through various landowners (Town, MTO and Private lands) obtain permission/approval to construct the required infrastructure requirements is a significant aspect to implementing any of the Option, including Option 1. As such it is recommended to initiate dialogue and obtain feedback form the Town, MTO and private landowners in the area regarding the proposed flood mitigation plan. Based on responses from the land owners the proposed options may be re-evaluated.

Respectfully submitted,

C.F. CROZIER & ASSOCIATES INC.

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JP/BH/bd
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APPENDIX A

Terms of Reference

DECEMBER 4TH, 2017

PROJECT NO: 1046-4031

SENT VIA: EMAIL
BWORSLEY@THEBLUEMOUNTAINS.CA

Town of The Blue Mountains
32 Mill Street, Box 310
Thornbury, ON N0H 2P0

Attention: **Brian Worsley, P.Eng.**
Manager of Engineering

RE: **UPDATED TERMS OF REFERENCE – STORMWATER MANAGEMENT STUDY**
PARKBRIDGE LIFESTYLE COMMUNITIES INC.
TOWN OF THE BLUE MOUNTAINS, GREY COUNTY

Dear Brian,

For your consideration, we are pleased to submit our Terms-of-Reference (TOR) for engineering services related to the preparation of a Stormwater Management Regional Plan examining combined solutions for Stormwater / Floodplain management for watercourse 7, 8, 9, & 10.

1. Purpose

It is understood that the Town of The Blue Mountain has requested a supplemental analysis as part of the Parkbridge Craigleith and MacPherson Home Farm properties illustrating the optimal floodplain management solution for the local area. This study will build upon the finding the Grey Sauble Conservation Authorities Subwatershed Study (1993) recognizing the historic identification of a flood damage center for Watercourse 7 at its outlet downstream of Highway 26.

2. Work Plan

Study Limits

This assessment will focus on Watercourse 7, 8, 9, & 10 in, through and downstream of the Parkbridge and Home Farm developments. As it is recognized the proposed developments are proposing full stormwater management quality / quantity controls onsite the study will focus on assessing identified flooding areas and/or capacity constraints downstream of the noted developments to Georgian Bay such that post development outlet rates consider the downstream capacity.

2.1. Phase 1 – Flood Risk Identification

Crozier and Associates has completed various hydraulic floodplain studies along the subject tributaries. It is generally known that a number of these watercourses spill along

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their downstream limits on route to Georgian Bay expanding flood potential to the general area. The proposed Study will review and identify these documented flood / spill susceptible areas with and downstream of proposed developments.

2.2. Phase 2 – Flood Conveyance Opportunity Review

Given the downstream position of the subject lands in the respective watersheds, providing safe conveyance of flows to Georgian Bay is the optimal method to address flooding concerns. The study will examine opportunities to increase and / or optimize the conveyance of flows to Georgian Bay to eliminate chronic flooding areas. As such, capacity of conveyance routes / infrastructure adjacent areas identified within Phase 1 will be analysed for performance in major storm events (100 year, Regional). Additionally, consideration will be given to the pre and post development erosion power versus hours (probability density function).

Typical conveyance routes / infrastructure that will be addressed includes:

- Natural Watercourse Channels
- Municipal Ditches / Drains
- Culverts
- Overland Flow Routes

Based on the above assessment, the feasibility of utilizing existing watercourses / infrastructure for conveyance of flows will be assessed.

2.3. Phase 3 – Flood Conveyance Improvements

Based on the results of the above noted phases, recommendations for floodplain improvements within the subject Watershed will be analysed. This may include the recommendation to optimize existing conveyance and infrastructures capacities or to provide additional capacity to promote more efficient and safe conveyance to Georgian Bay.

Preliminary designs and calculations of proposed flood improvements will be included to demonstrate the design principle behind each solution to alleviate flood susceptible areas. Figures which demonstrate the preliminary design intent will be prepared. Lastly, timing and triggers for implementation of improvements will also be assessed.

2.4. Phase 4 – Documentation

Documentation of all findings and recommendations will be included in a Stormwater Regional Plan Report for submission to the Town of The Blue Mountains, County and Grey Sauble Conservation Authority for review. This report will include relevant figures and calculations supporting the proposed study.

3. Conclusions

We look forward to working with you on this project. Should you have any questions, please do not hesitate to contact the undersigned.

Yours truly,

C.F. CROZIER & ASSOCIATES INC.

Jon Proctor

Jon Proctor, P.Eng
Associate

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APPENDIX B

Study Area Existing Conditions

Study Area - Existing Conditions

Legend

- Parcels
- Large Scale Roads
  - Provincial Highway
  - County Road
  - Township Road
  - Seasonal Road

0.32 0 0.16 0.32 Kilometers

WGS_1984_Web_Mercator_Auxiliary_Sphere

This map is a user generated static output from an Internet mapping site and is for reference only. Data layers that appear on this map may or may not be accurate, current, or otherwise reliable.

Printed: April 27, 2018

THIS MAP IS NOT TO BE USED FOR NAVIGATION

Notes

Study Area - Soils

Grey County

Kitzbuhl Crescent
Sleepy Hollow Road
Grey Road 19
Schoolhouse Court
Highway 26
Lakeshore Road East
Highway 26
Lakeshore Road East
Fraser Crescent
Highway 26
Blue Mountain Drive
Blue Mountain Drive
Highway 26
Kimheather Court
Timmons Street
Lendvay Alley
Grey Road 19
Craigleith Road
Comet Lane
National Drive
Birches Boulevard
Manitou Court
Venture Boulevard
Craigmore Crescent
Arlberg Crescent
Tyrolean Lane
Tyrolean Lane
Kandahar Lane
Birch View Trail
Arlberg Crescent
Brooker Boulevard
Liisa's Lane

Legend

- Parcels
- Large Scale Roads
  - Provincial Highway
  - County Road
  - Township Road
  - Seasonal Road
- SOIL (OMAFRA / MNRF)
  - 0
  - 1
  - 2
  - 3
  - 4
  - 5
  - 6
  - 7

0.32 0 0.16 0.32 Kilometers

WGS_1984_Web_Mercator_Auxiliary_Sphere

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Printed: April 27, 2018

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Notes

APPENDIX C

Site Photos

Watercourse 7

7.1 WC7 Spill Flow on Upper Terrace (Home Farm) 2016.03.31

7.2 WC7 Spill Flow on Upper Terrace (Home Farm) 2016.03.31

7.3 WC7 Spill Location Eden Oak 2016.03.31

7.4 WC7 Upstream Lakeshore Road 2016.03.31

7.5 WC7 Downstream Lakeshore Road 2016.03.31

7.6 WC7 Upstream Georgian Trail 2016.03.31

7.7 WC7 Downstream Georgian Trial 2016.03.31

7.8 WC7 Downstream Georgian Trial 2016.03.31

7.9 WC7 Outlet 2016.03.31

Watercourse 8

8.1 WC8 Upstream of Lakeshore Road 2016.03.31

8.2 WC8 Upstream of Lakeshore Road 2016.03.31

8.3 WC8 Downstream of Lakeshore Road 2016.03.31

8.4 WC8 Downstream of Georgian Trail 2016.03.31

8.5 WC8 Upstream of Highway 26 2016.03.31

8.6 WC8 Downstream of Highway 26 (Outlet) 2016.03.31

Watercourse 9

9.1 WC 9 Upstream of Lakeshore Road (Overbank berm) 2018.04.25

9.2 WC 9 Location of Spill flow to WC8 Upstream Lakeshore 2018.04.25

9.3 WC 9 Lakeshore Road South Ditch (facing east) 2018.04.25

9.4 WC9 Upstream of Lakeshore Road 2016.03.31

9.5 WC9 Upstream of Lakeshore Road 2016.03.31

9.6 WC9 Downstream of Georgian Trail 2016.03.31

9.7 WC9 Downstream of Highway 26 2016.03.31

9.8 WC9 Downstream of Highway 26 2016.03.31

9.9 WC9 Outlet channel between existing houses 2016.03.31

APPENDIX D

Existing Hydraulic Structures Inventory & Calculations

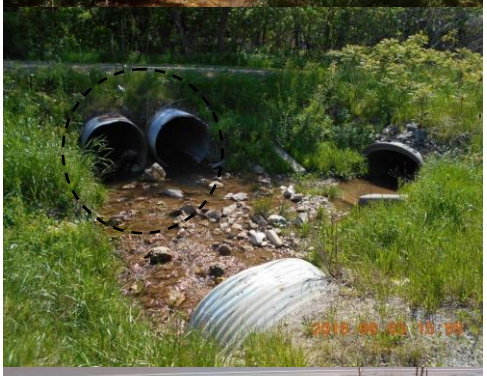
WATERCOURSE 7 – CULVERTS

LAKESHORE ROAD

Description:	Concrete Box Culvert
Size:	0.95m rise x 1.9m span
Upstream invert:	181.51m
Downstream invert:	181.28m
Length:	9m
Slope:	2.5%
Road Deck Elevation:	182.74m

GEORGIAN TRAIL

Description:	Twin CSP Culvert
Size:	Circular 1200mm dia.
Upstream invert:	179.04m
Downstream invert:	179.50m
Length:	10.5m
Slope:	5.14%
Road Deck Elevation:	180.57m



HIGHWAY 26

Description:	CSP Arch
Size:	0.9m rise x 1.35m span
Upstream invert:	178.40m
Downstream invert:	177.89m
Length:	22m
Slope:	2.32%
Road Deck Elevation:	179.64m

LATERAL DIVERSION PIPE (WC 7 to WC8)

Description:	Concrete Pipe Arch
Size:	0.9m rise x 1.5m span
Upstream invert:	178.41m
Downstream invert:	177.70m
Length:	115m
Slope:	0.62%
Road Deck Elevation:	179.64m (Highway 26)

Culvert Calculator Report
WC 7 Georgian Trail Culvert

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	180.57	m	Headwater Depth/Height	1.25	
Computed Headwater Elev	180.57	m	Discharge	4.5422	m³/s
Inlet Control HW Elev.	180.44	m	Tailwater Elevation	179.64	m
Outlet Control HW Elev.	180.57	m	Control Type	Entrance Control	

Grades					
Upstream Invert	179.04	m	Downstream Invert	178.50	m
Length	10.50	m	Constructed Slope	0.051429	m/m

Hydraulic Profile					
Profile	CompositeS1S2		Depth, Downstream	1.14	m
Slope Type	Steep		Normal Depth	0.58	m
Flow Regime	N/A		Critical Depth	0.83	m
Velocity Downstream	2.00	m/s	Critical Slope	0.016487	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	1.22	m
Section Size	1200 mm		Rise	1.22	m
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	180.57	m	Upstream Velocity Head	0.37	m
Ke	0.90		Entrance Loss	0.33	m

Inlet Control Properties					
Inlet Control HW Elev.	180.44	m	Flow Control	Unsubmerged	
Inlet Type	Projecting		Area Full	2.3	m²
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				

Culvert Calculator Report
WC 7 Highway 26 Culvert

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.64	m	Headwater Depth/Height	1.28	
Computed Headwater Elev	179.64	m	Discharge	2.3380	m³/s
Inlet Control HW Elev.	179.64	m	Tailwater Elevation	178.89	m
Outlet Control HW Elev.	179.62	m	Control Type	Inlet Control	

Grades					
Upstream Invert	178.40	m	Downstream Invert	177.89	m
Length	22.00	m	Constructed Slope	0.023182	m/m

Hydraulic Profile					
Profile	CompositePressureProfileS1		Depth, Downstream	0.62	m
Slope Type	N/A		Normal Depth	0.62	m
Flow Regime	Subcritical		Critical Depth	0.68	m
Velocity Downstream	2.93	m/s	Critical Slope	0.019032	m/m

Section					
Section Shape	Arch		Mannings Coefficient	0.025	
Section Material	Steel and Aluminum Var CR		Span	1.45	m
Section Size	1390 x 970 mm		Rise	0.97	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	179.62	m	Upstream Velocity Head	0.37	m
Ke	0.50		Entrance Loss	0.17	m

Inlet Control Properties					
Inlet Control HW Elev.	179.64	m	Flow Control	Transition	
Inlet Type	90° headwall		Area Full	1.1	m²
K	0.00830		HDS 5 Chart	34	
M	2.00000		HDS 5 Scale	1	
C	0.03790		Equation Form	1	
Y	0.69000				

Culvert Calculator Report
WC 7 Lakeshore Culvert

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	182.74	m	Headwater Depth/Height	1.35	
Computed Headwater Elev	182.74	m	Discharge	3.7804	m³/s
Inlet Control HW Elev.	182.74	m	Tailwater Elevation	0.00	m
Outlet Control HW Elev.	182.69	m	Control Type	Inlet Control	

Grades					
Upstream Invert	181.51	m	Downstream Invert	181.28	m
Length	9.00	m	Constructed Slope	0.025556	m/m

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.52	m
Slope Type	Steep		Normal Depth	0.40	m
Flow Regime	Supercritical		Critical Depth	0.76	m
Velocity Downstream	3.98	m/s	Critical Slope	0.004066	m/m

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	1.83	m
Section Size	1830 x 910 mm		Rise	0.91	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	182.69	m	Upstream Velocity Head	0.37	m
Ke	0.20		Entrance Loss	0.07	m

Inlet Control Properties					
Inlet Control HW Elev.	182.74	m	Flow Control	N/A	
Inlet Type	90° headwall w 45° bevels		Area Full	1.7	m²
K	0.49500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	2	
C	0.03140		Equation Form	2	
Y	0.82000				

Culvert Calculator Report
Lateral Concrete Pipe

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.64	m	Headwater Depth/Height	1.35	
Computed Headwater Elev	179.64	m	Discharge	1.4745	m³/s
Inlet Control HW Elev.	179.31	m	Tailwater Elevation	179.31	m
Outlet Control HW Elev.	179.64	m	Control Type	Outlet Control	

Grades					
Upstream Invert	178.41	m	Downstream Invert	177.70	m
Length	115.00	m	Constructed Slope	0.006174	m/m

Hydraulic Profile					
Profile	PressureProfile		Depth, Downstream	1.61	m
Slope Type	N/A		Normal Depth	0.44	m
Flow Regime	N/A		Critical Depth	0.51	m
Velocity Downstream	1.39	m/s	Critical Slope	0.004035	m/m

Section					
Section Shape	Arch		Mannings Coefficient	0.013	
Section Material	Concrete		Span	1.49	m
Section Size	1490 x 910 mm		Rise	0.91	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	179.64	m	Upstream Velocity Head	0.10	m
Ke	0.20		Entrance Loss	0.02	m

Inlet Control Properties					
Inlet Control HW Elev.	179.31	m	Flow Control	N/A	
Inlet Type	Groove end projecting (arch)		Area Full	1.1	m²
K	0.00450		HDS 5 Chart	0	
M	2.00000		HDS 5 Scale	0	
C	0.03170		Equation Form	1	
Y	0.69000				

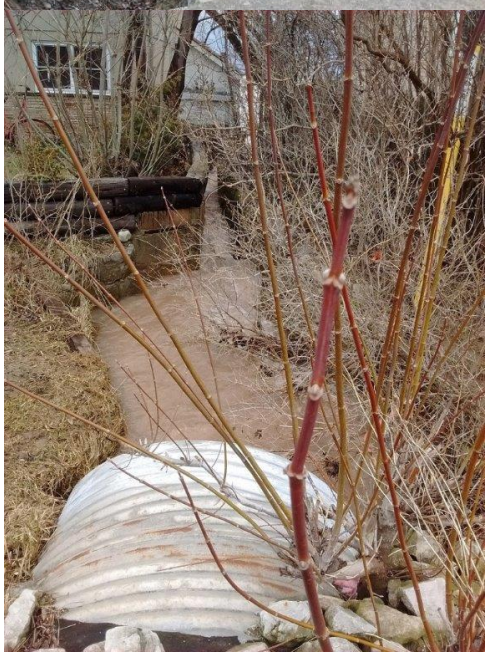
WATERCOURSE 7 – CHANNEL SECTIONS

DOWNSTREAM LAKESHORE ROAD

Description:	Trapezoidal section
Bottom Width:	2.3m
Top Width:	3.8m
Depth:	1.15m
Average Slope:	3.8%

DOWNSTREAM HIGHWAY 26 (OUTLET)

Description:	Rectangular section
Bottom Width:	0.61m
Top Width:	0.91m
Depth:	0.97m
Average Slope:	1.8%



HIGHWAY 26 DITCH (WC 7 to WC8)

Description:	Triangular section
Bottom Width:	-
Top Width:	3.5m
Depth:	0.78m
Average Slope:	1.86%

WC7 Downstream Lakeshore - Existing

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.03768	m/m
Normal Depth	1.18	m

Section Definitions

Station (m)	Elevation (m)
0+00.00	182.43
0+00.92	180.59
0+03.22	180.62
0+03.80	181.77

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.00, 182.43)	(0+03.80, 181.77)	0.045

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Discharge	11.27	m³/s
Elevation Range	180.59 to 182.43 m	
Flow Area	3.36	m²
Wetted Perimeter	4.91	m
Hydraulic Radius	0.68	m
Top Width	3.47	m
Normal Depth	1.18	m
Critical Depth	1.24	m
Critical Slope	0.03203	m/m

WC7 Downstream Lakeshore - Existing

Results

Velocity		3.35	m/s
Velocity Head		0.57	m
Specific Energy		1.75	m
Froude Number		1.09	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	1.18	m
Critical Depth	1.24	m
Channel Slope	0.03768	m/m
Critical Slope	0.03203	m/m

WC7 Downstream Lakeshore - Existing

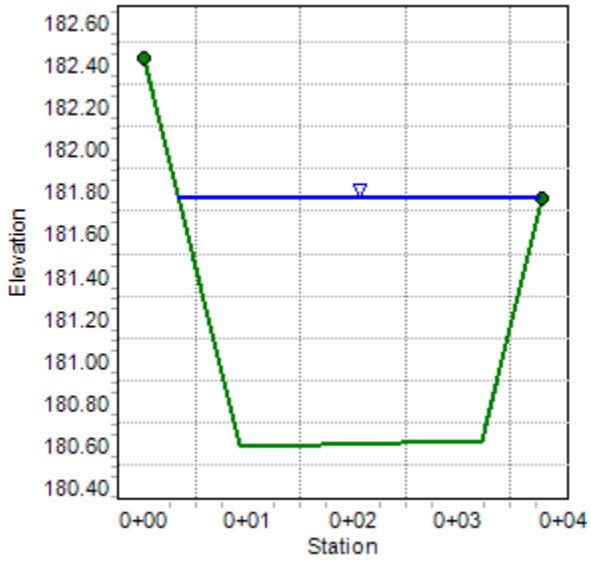
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.03768	m/m
Normal Depth	1.18	m
Discharge	11.27	m³/s

Cross Section Image



Highway 26 Ditch - Existing

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.00500	m/m
Normal Depth	0.78	m

Section Definitions

Station (m)	Elevation (m)
0+00.00	179.37
0+02.50	179.19
0+04.00	179.05
0+06.00	178.59
0+09.00	179.38
0+10.50	179.62
0+14.00	180.14

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.00, 179.37)	(0+14.00, 180.14)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Discharge		1.86	m³/s
Elevation Range	178.59 to 180.14 m		
Flow Area		2.86	m²
Wetted Perimeter		9.13	m
Hydraulic Radius		0.31	m
Top Width		8.96	m

Highway 26 Ditch - Existing

Results

Normal Depth		0.78	m
Critical Depth		0.54	m
Critical Slope		0.04050	m/m
Velocity		0.65	m/s
Velocity Head		0.02	m
Specific Energy		0.80	m
Froude Number		0.37	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth		0.00	m
Length		0.00	m
Number Of Steps		0	

GVF Output Data

Upstream Depth		0.00	m
Profile Description			
Profile Headloss		0.00	m
Downstream Velocity		Infinity	m/s
Upstream Velocity		Infinity	m/s
Normal Depth		0.78	m
Critical Depth		0.54	m
Channel Slope		0.00500	m/m
Critical Slope		0.04050	m/m

Highway 26 Ditch - Existing

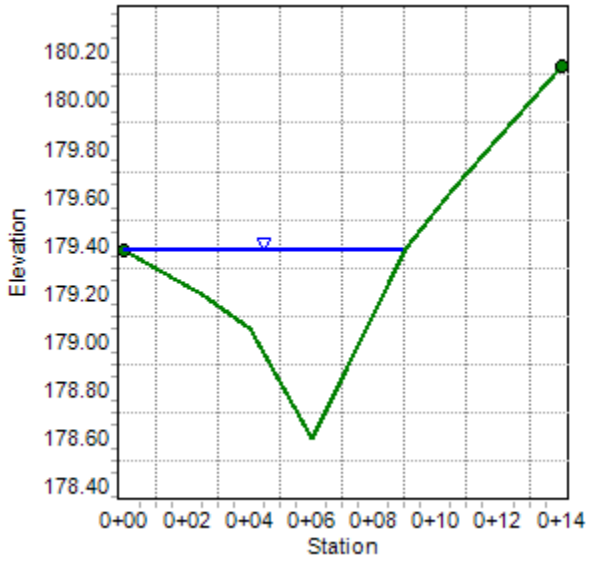
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.00500	m/m
Normal Depth	0.78	m
Discharge	1.86	m³/s

Cross Section Image



WC7 Outlet - Existing

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.01818	m/m
Normal Depth	0.97	m

Section Definitions

Station (m)	Elevation (m)
0+00.00	179.20
0+00.00	177.67
0+00.61	177.67
0+00.91	178.64

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.00, 179.20)	(0+00.91, 178.64)	0.035

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Discharge	1.23	m³/s
Elevation Range	177.67 to 179.20 m	
Flow Area	0.74	m²
Wetted Perimeter	2.60	m
Hydraulic Radius	0.28	m
Top Width	0.91	m
Normal Depth	0.97	m
Critical Depth	0.70	m
Critical Slope	0.04737	m/m

WC7 Outlet - Existing

Results

Velocity		1.66	m/s
Velocity Head		0.14	m
Specific Energy		1.11	m
Froude Number		0.59	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.97	m
Critical Depth	0.70	m
Channel Slope	0.01818	m/m
Critical Slope	0.04737	m/m

WC7 Outlet - Existing

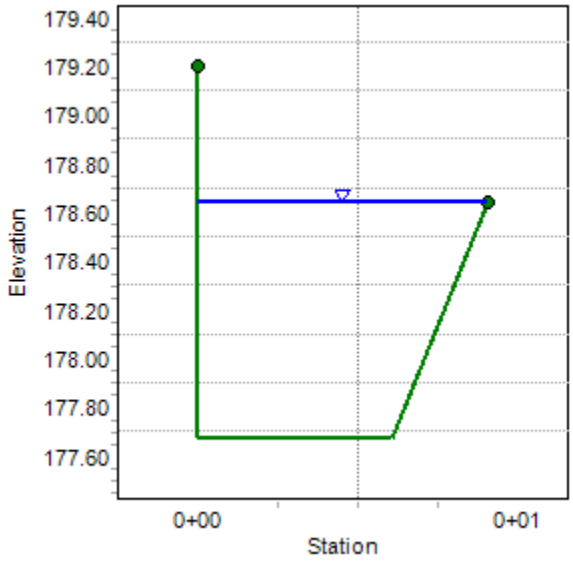
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.01818	m/m
Normal Depth	0.97	m
Discharge	1.23	m³/s

Cross Section Image



WATERCOURSE 8 – CULVERTS

LAKESHORE ROAD

Description:	CSP Culvert
Size:	Circular 350mm dia.
Upstream invert:	180.09m
Downstream invert:	179.71m
Length:	9.5m
Slope:	4%
Road Deck Elevation:	181.01m

GEORGIAN TRAIL

Description:	Twin CSP Culvert
Size:	Circular 750mm dia.
Upstream invert:	178.44m
Downstream invert:	177.76m
Length:	10m
Slope:	6.8%
Road Deck Elevation:	180.43m

HIGHWAY 26 CULVERT

Description:	CSP Arch
Size:	1.0m rise x 1.6m span
Upstream invert:	177.83
Downstream invert:	177.63m
Length:	25m
Slope:	0.8%
Road Deck Elevation:	179.31m

Culvert Calculator Report
WC 8 Lakeshore Culvert

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	181.01	m	Headwater Depth/Height	3.02	
Computed Headwater Elev	181.01	m	Discharge	0.1077	m³/s
Inlet Control HW Elev.	180.64	m	Tailwater Elevation	180.43	m
Outlet Control HW Elev.	181.01	m	Control Type	Outlet Control	

Grades					
Upstream Invert	180.09	m	Downstream Invert	179.71	m
Length	9.50	m	Constructed Slope	0.040000	m/m

Hydraulic Profile					
Profile	PressureProfile		Depth, Downstream	0.72	m
Slope Type	N/A		Normal Depth	0.25	m
Flow Regime	N/A		Critical Depth	0.25	m
Velocity Downstream	1.48	m/s	Critical Slope	0.038022	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	0.30	m
Section Size	300 mm		Rise	0.30	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	181.01	m	Upstream Velocity Head	0.11	m
Ke	0.90		Entrance Loss	0.10	m

Inlet Control Properties					
Inlet Control HW Elev.	180.64	m	Flow Control	Submerged	
Inlet Type	Projecting		Area Full	0.1	m²
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				

Culvert Calculator Report
WC 8 Georgian Trail Culvert

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	180.43	m	Headwater Depth/Height	2.61	
Computed Headwater Elev	180.43	m	Discharge	2.4971	m³/s
Inlet Control HW Elev.	180.19	m	Tailwater Elevation	179.31	m
Outlet Control HW Elev.	180.43	m	Control Type	Outlet Control	

Grades					
Upstream Invert	178.44	m	Downstream Invert	177.76	m
Length	10.00	m	Constructed Slope	0.068000	m/m

Hydraulic Profile					
Profile	PressureProfile		Depth, Downstream	1.55	m
Slope Type	N/A		Normal Depth	0.50	m
Flow Regime	N/A		Critical Depth	0.67	m
Velocity Downstream	2.74	m/s	Critical Slope	0.035269	m/m

Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	0.76	m
Section Size	750 mm		Rise	0.76	m
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	180.43	m	Upstream Velocity Head	0.38	m
Ke	0.90		Entrance Loss	0.34	m

Inlet Control Properties					
Inlet Control HW Elev.	180.19	m	Flow Control	N/A	
Inlet Type	Projecting		Area Full	0.9	m²
K	0.03400		HDS 5 Chart	2	
M	1.50000		HDS 5 Scale	3	
C	0.05530		Equation Form	1	
Y	0.54000				

Culvert Calculator Report
WC 8 Highway 26 Culvert

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.31	m	Headwater Depth/Height	1.36	
Computed Headwater Elev	179.31	m	Discharge	3.0145	m³/s
Inlet Control HW Elev.	179.26	m	Tailwater Elevation	178.63	m
Outlet Control HW Elev.	179.31	m	Control Type	Outlet Control	

Grades					
Upstream Invert	177.83	m	Downstream Invert	177.63	m
Length	25.00	m	Constructed Slope	0.008000	m/m

Hydraulic Profile					
Profile	CompositeM2PressureProfile		Depth, Downstream	0.74	m
Slope Type	Mild		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.74	m
Velocity Downstream	2.80	m/s	Critical Slope	0.017599	m/m

Section					
Section Shape	Arch		Mannings Coefficient	0.025	
Section Material	Steel and Aluminum Var CR		Span	1.63	m
Section Size	1630 x 1120 mm		Rise	1.09	m
Number Sections	1				

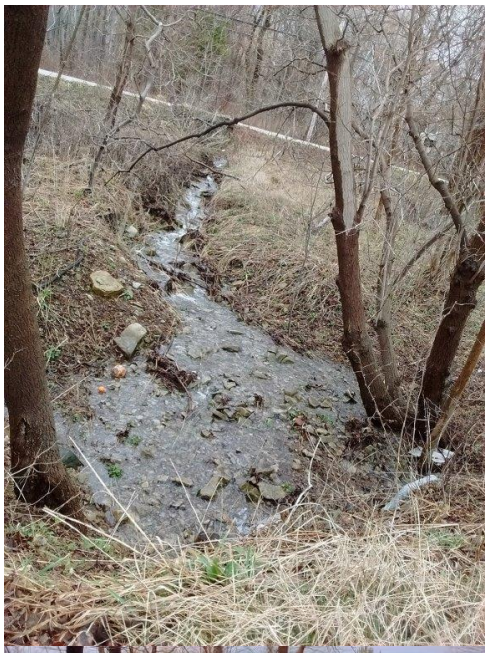
Outlet Control Properties					
Outlet Control HW Elev.	179.31	m	Upstream Velocity Head	0.25	m
Ke	0.90		Entrance Loss	0.23	m

Inlet Control Properties					
Inlet Control HW Elev.	179.26	m	Flow Control	N/A	
Inlet Type	Thin wall projecting		Area Full	1.4	m²
K	0.03400		HDS 5 Chart	34	
M	1.50000		HDS 5 Scale	3	
C	0.04960		Equation Form	1	
Y	0.57000				

WATERCOURSE 8 – CHANNEL SECTIONS

DOWNSTREAM LAKESHORE ROAD

Description:	Trapezoidal section
Bottom Width:	2.3m
Top Width:	3.8m
Depth:	1.15m
Average Slope:	3.8%



DOWNSTREAM HIGHWAY 26 (OUTLET)

Description:	Trapezoidal section
Bottom Width:	1.5m
Top Width:	2.3m
Depth:	0.42m
Average Slope:	1.0%

WC8 Downstream Lakeshore - Existing

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.06400	m/m
Normal Depth	0.43	m

Section Definitions

Station (m)	Elevation (m)
0+00.00	179.09
0+10.00	179.00
0+10.50	178.59
0+12.10	178.57
0+12.60	179.03
0+24.60	179.09

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.00, 179.09)	(0+10.00, 179.00)	0.050
(0+10.00, 179.00)	(0+12.60, 179.03)	0.035
(0+12.60, 179.03)	(0+24.60, 179.09)	0.050

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Discharge		2.86	m³/s
Elevation Range	178.57 to 179.09 m		
Flow Area		0.87	m²
Wetted Perimeter		2.88	m
Hydraulic Radius		0.30	m

WC8 Downstream Lakeshore - Existing

Results

Top Width		2.57	m
Normal Depth		0.43	m
Critical Depth		0.55	m
Critical Slope		0.02551	m/m
Velocity		3.27	m/s
Velocity Head		0.54	m
Specific Energy		0.97	m
Froude Number		1.79	
Flow Type	Supercritical		

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.43	m
Critical Depth	0.55	m
Channel Slope	0.06400	m/m
Critical Slope	0.02551	m/m

WC8 Downstream Lakeshore - Existing

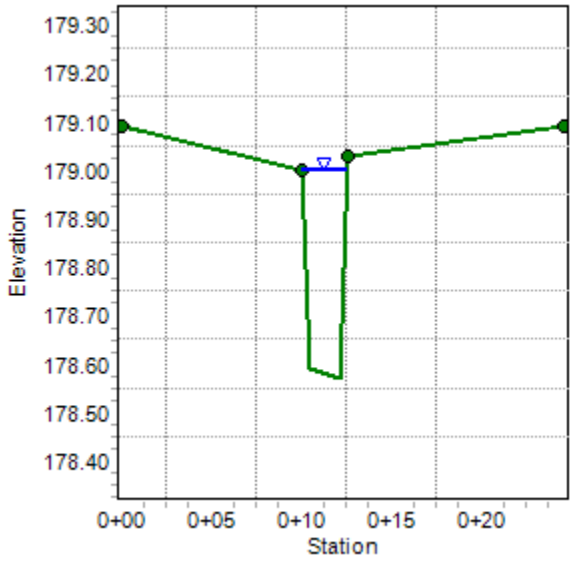
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.06400	m/m
Normal Depth	0.43	m
Discharge	2.86	m³/s

Cross Section Image



WC8 Outlet - Existing

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.01000	m/m
Normal Depth	0.42	m

Section Definitions

Station (m)	Elevation (m)
0+00.00	178.14
0+00.40	177.66
0+01.90	177.57
0+02.30	177.99

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00.00, 178.14)	(0+00.40, 177.66)	0.090
(0+00.40, 177.66)	(0+01.90, 177.57)	0.035
(0+01.90, 177.57)	(0+02.30, 177.99)	0.090

Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

Results

Discharge	0.46	m³/s
Elevation Range	177.57 to 178.14 m	
Flow Area	0.69	m²
Wetted Perimeter	2.51	m
Hydraulic Radius	0.28	m
Top Width	2.18	m
Normal Depth	0.42	m

WC8 Outlet - Existing

Results

Critical Depth		0.25	m
Critical Slope		0.07782	m/m
Velocity		0.67	m/s
Velocity Head		0.02	m
Specific Energy		0.44	m
Froude Number		0.38	
Flow Type	Subcritical		

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.42	m
Critical Depth	0.25	m
Channel Slope	0.01000	m/m
Critical Slope	0.07782	m/m

WC8 Outlet - Existing

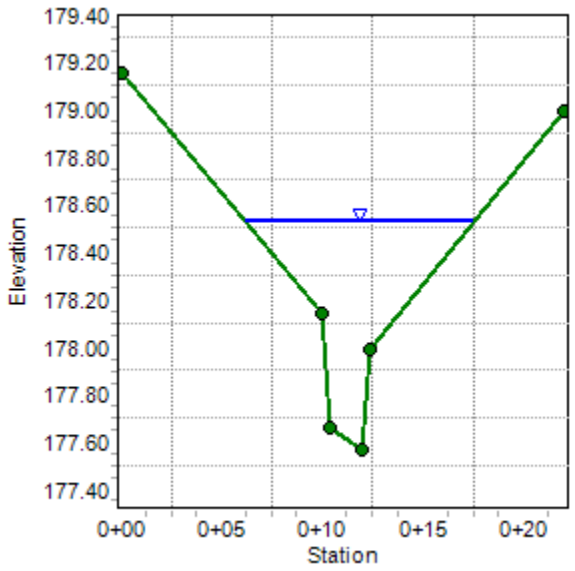
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Channel Slope	0.01000	m/m
Normal Depth	0.96	m
Discharge	2.51	m³/s

Cross Section Image



WATERCOURSE 9 – CULVERTS

LAKESHORE ROAD

Description:	Concrete box culvert
Size:	1.05m rise x 1.5m span
Upstream invert:	180.47m
Downstream invert:	180.33m
Length:	9.25m
Slope:	1.5%
Road Deck Elevation:	181.98m

GEORGIAN TRAIL

Description:	CSP Culvert
Size:	Circular 1500mm dia.
Upstream invert:	178.9m
Downstream invert:	178.1m
Length:	11m
Slope:	6.4%
Road Deck Elevation:	180.45m

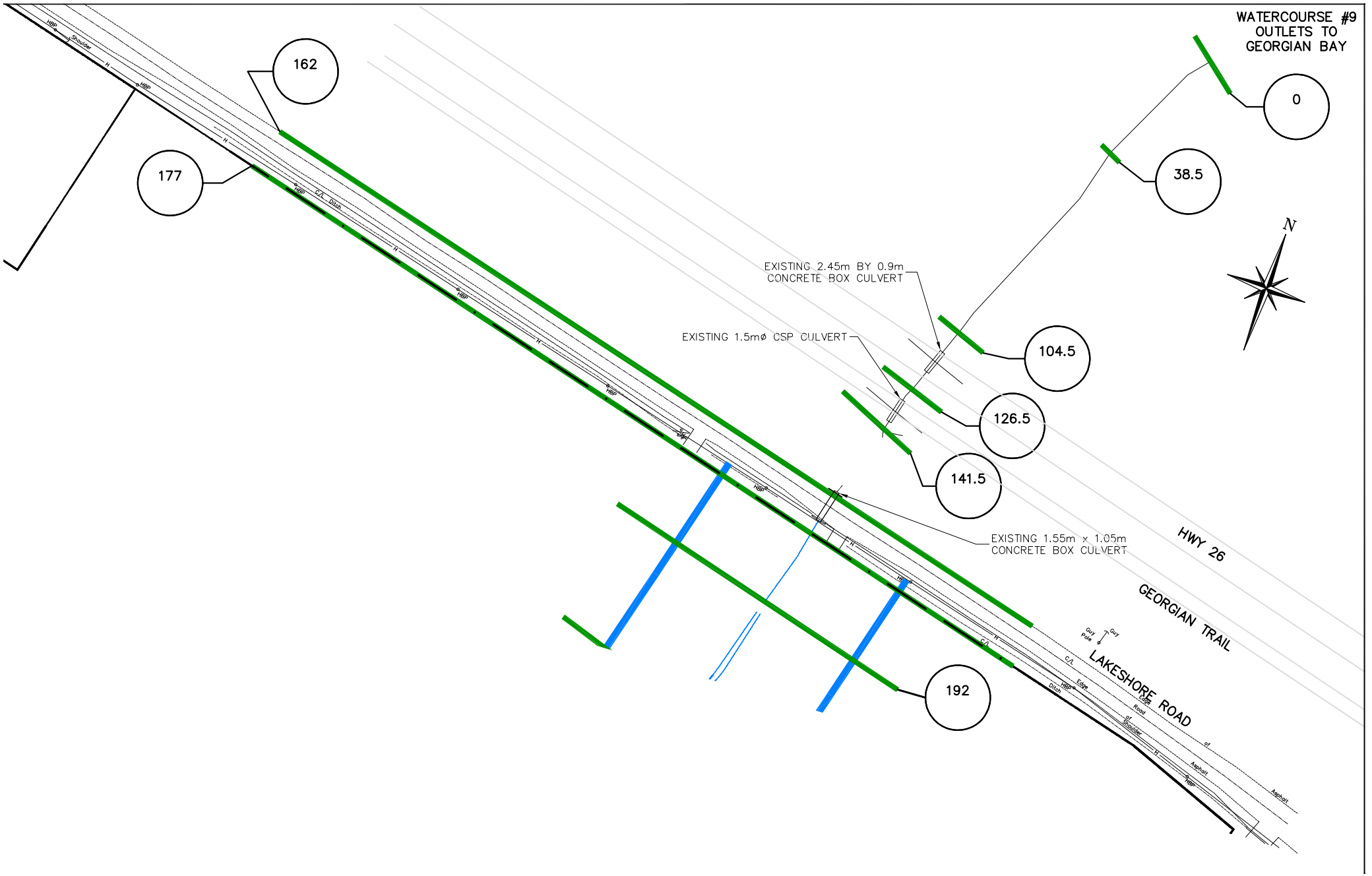
HIGHWAY 26 CULVERT

Description:	Concrete box culvert
Size:	0.9m rise x 2.45m span
Upstream invert:	178.29m
Downstream invert:	178.08m
Length:	11.3m
Slope:	1.8%
Road Deck Elevation:	179.69m

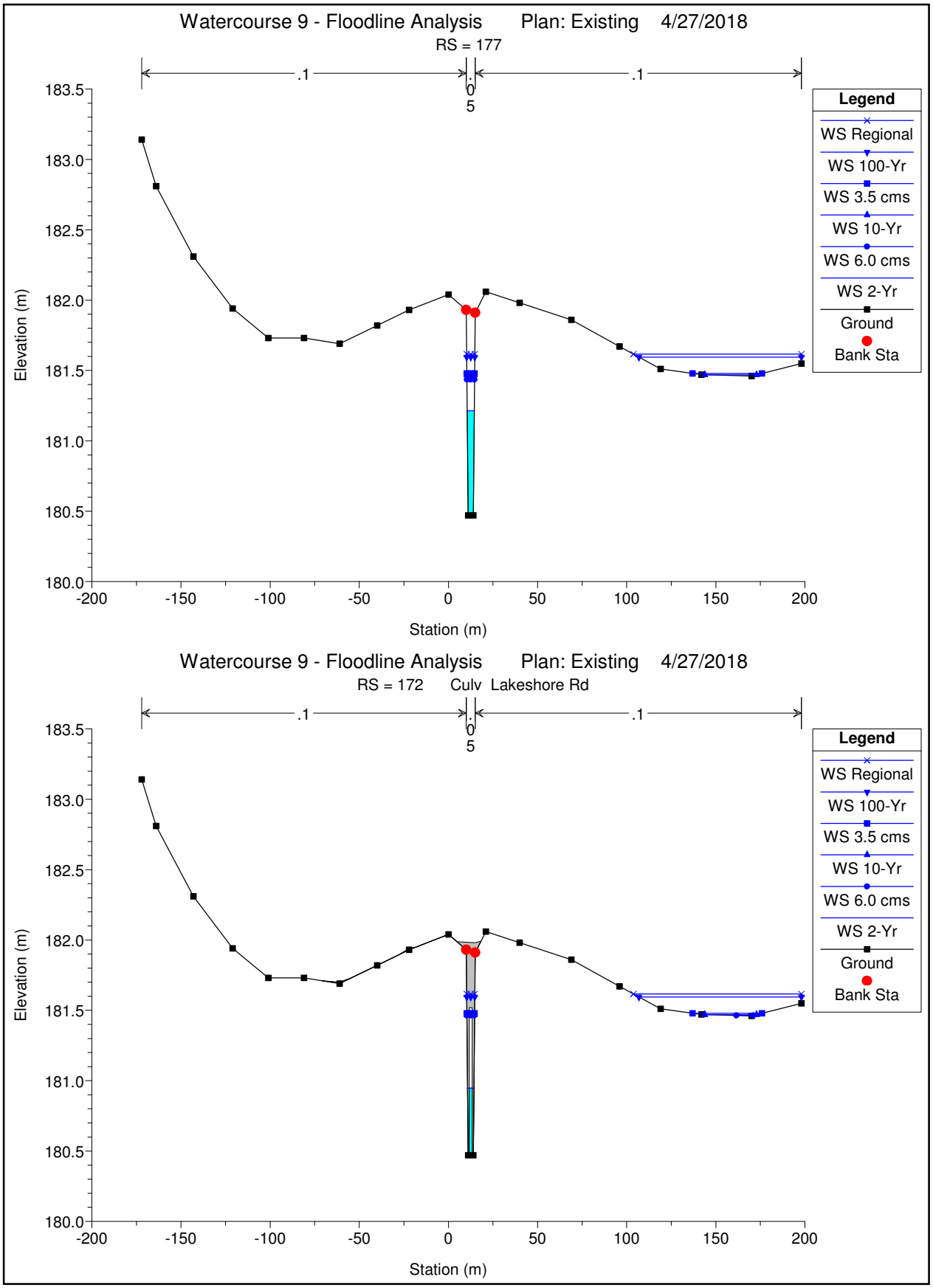
WATERCOURSE 9 – CHANNEL SECTIONS

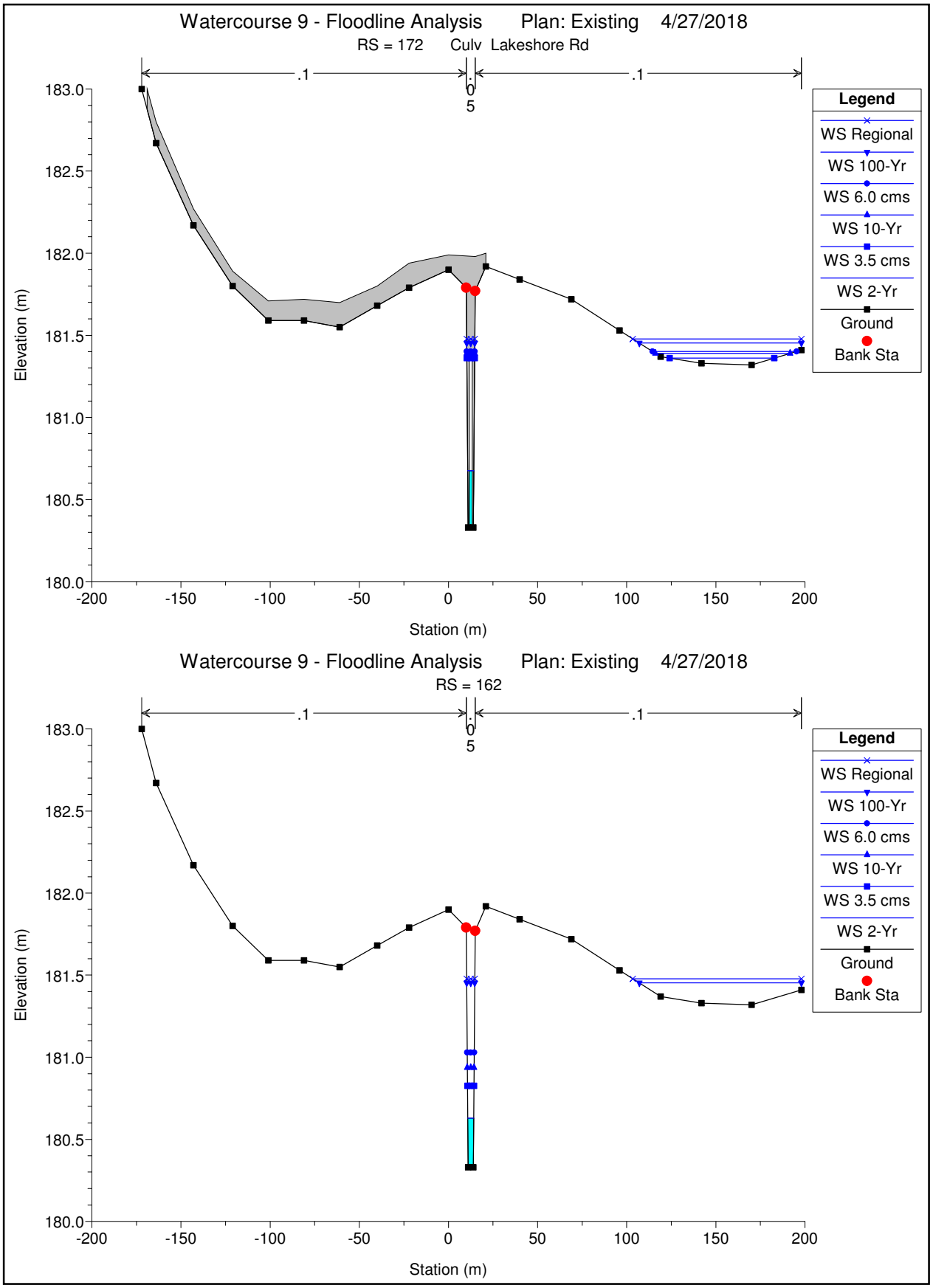
DOWNSTREAM HIGHWAY 26 (OUTLET)

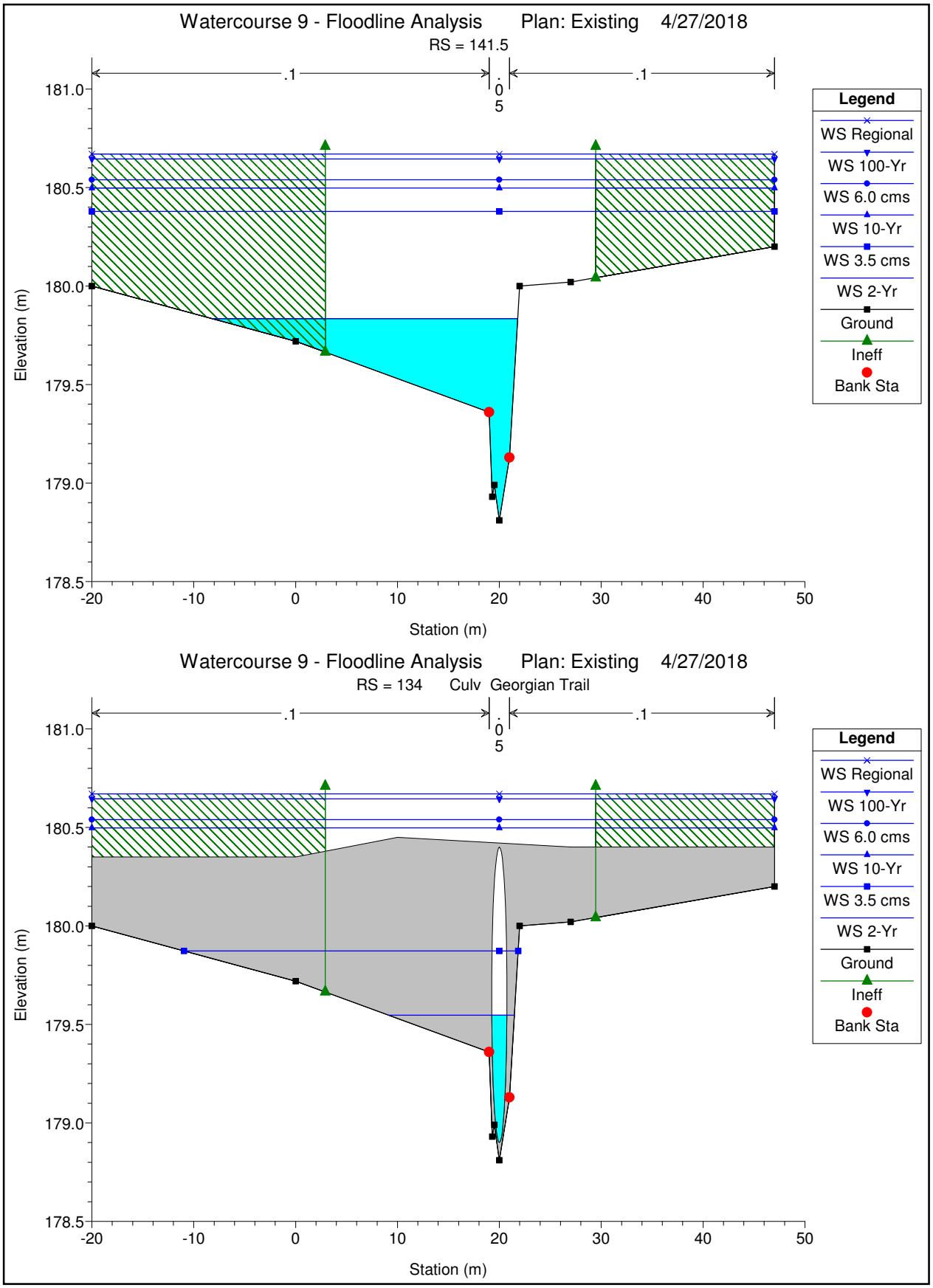
Description:	Trapezoidal section
Bottom Width:	1.5m
Top Width:	2.0m
Depth:	1.0m
Average Slope:	1.3%

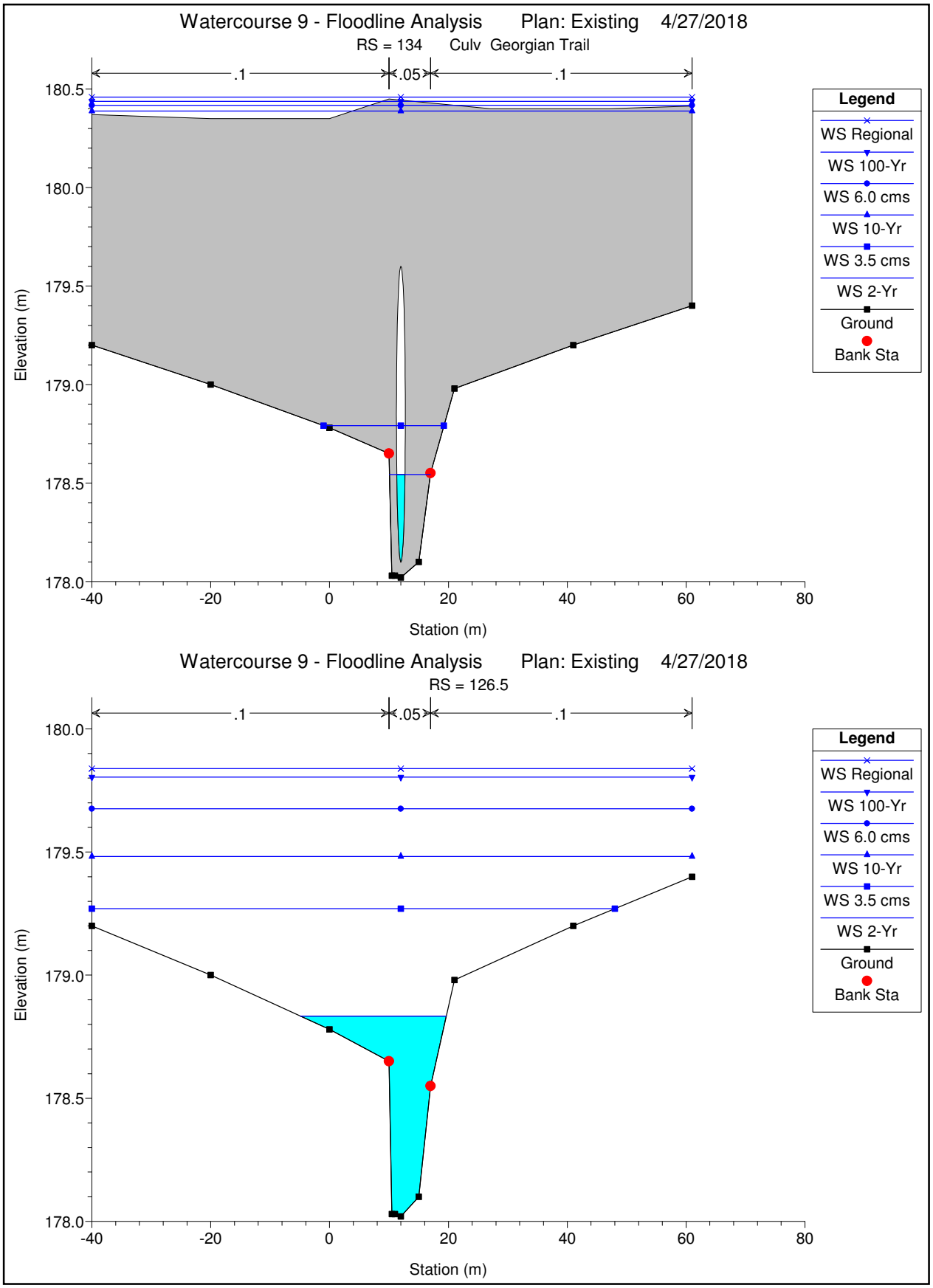


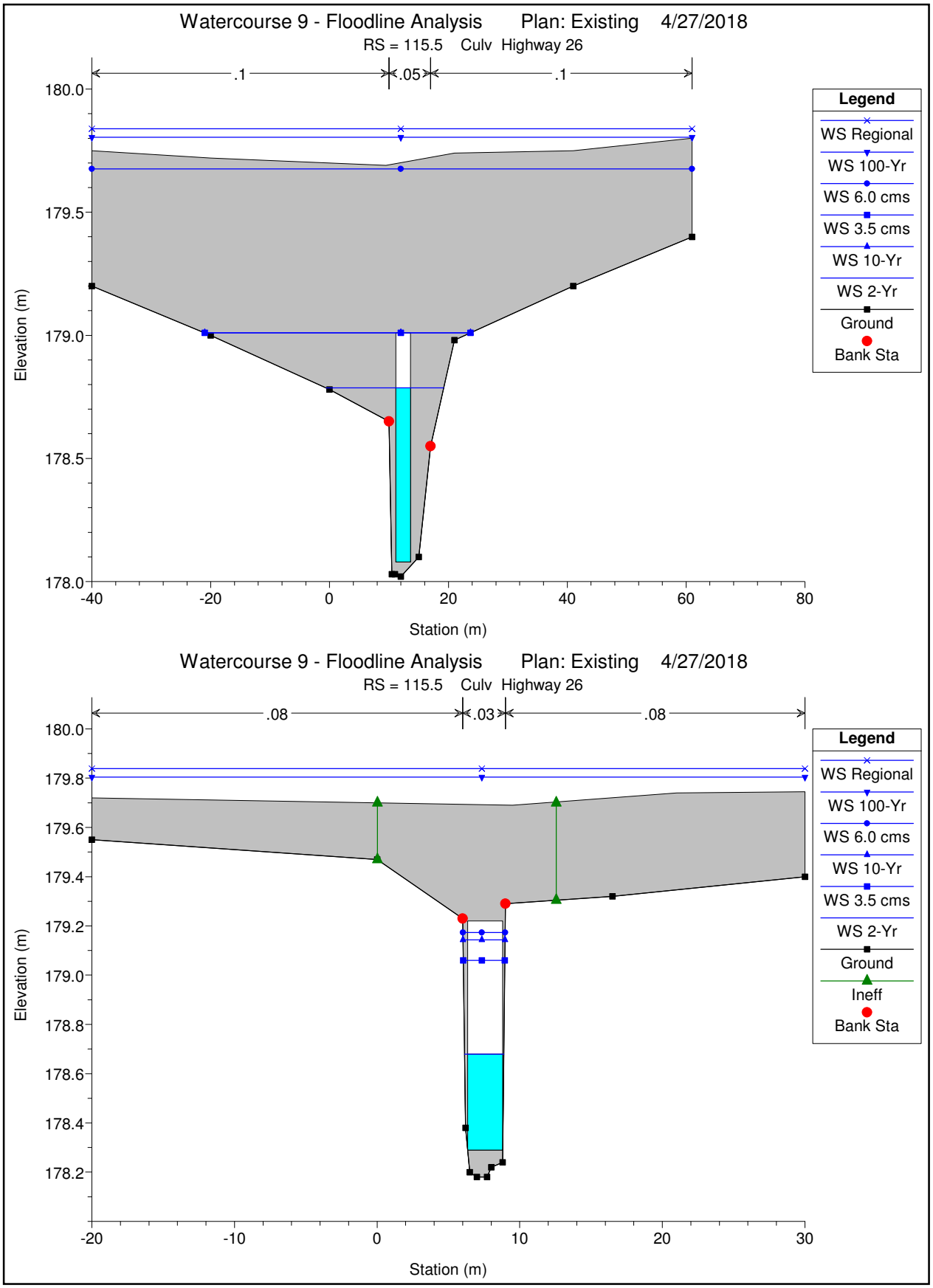
J:\1000\1046-Parkbridge Lifestyle\4031-Craigleith Fogal Land Acq\CAD\CIVIL\1SHEET\4031_215-216.dwg, 210, 4/27/2018 12:07:27 PM,

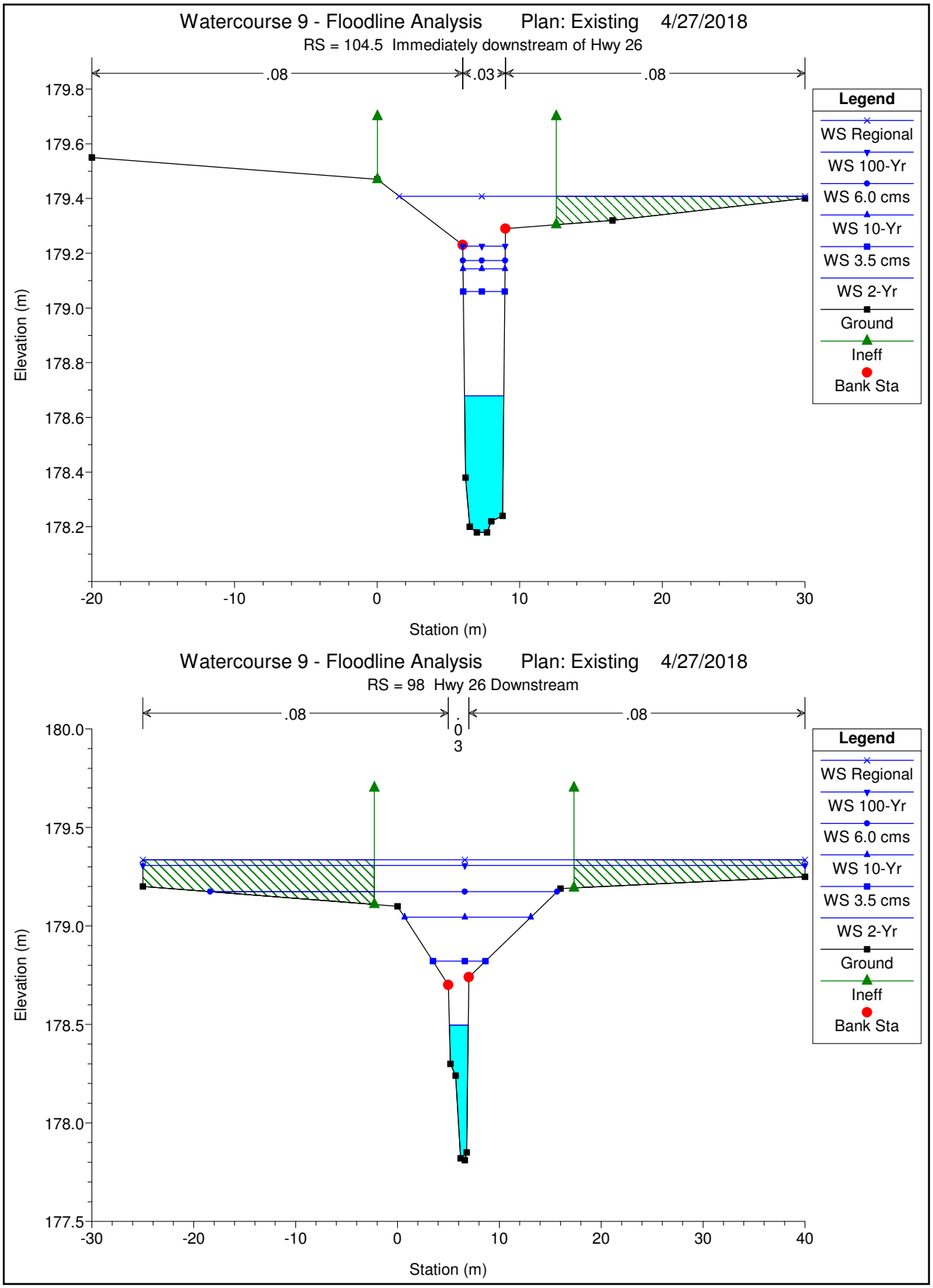

Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 177
.1
.1
.05
Elevation (m)
183.5
183.0
182.5
182.0
181.5
181.0
180.5
180.0
-200
-150
-100
-50
0
50
100
150
200
Station (m)
Legend
WS Regional
WS 100-Yr
WS 3.5 cms
WS 10-Yr
WS 6.0 cms
WS 2-Yr
Ground
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 172 Culv Lakeshore Rd
.1
.1
.05
Elevation (m)
183.5
183.0
182.5
182.0
181.5
181.0
180.5
180.0
-200
-150
-100
-50
0
50
100
150
200
Station (m)
Legend
WS Regional
WS 100-Yr
WS 3.5 cms
WS 10-Yr
WS 6.0 cms
WS 2-Yr
Ground
Bank Sta

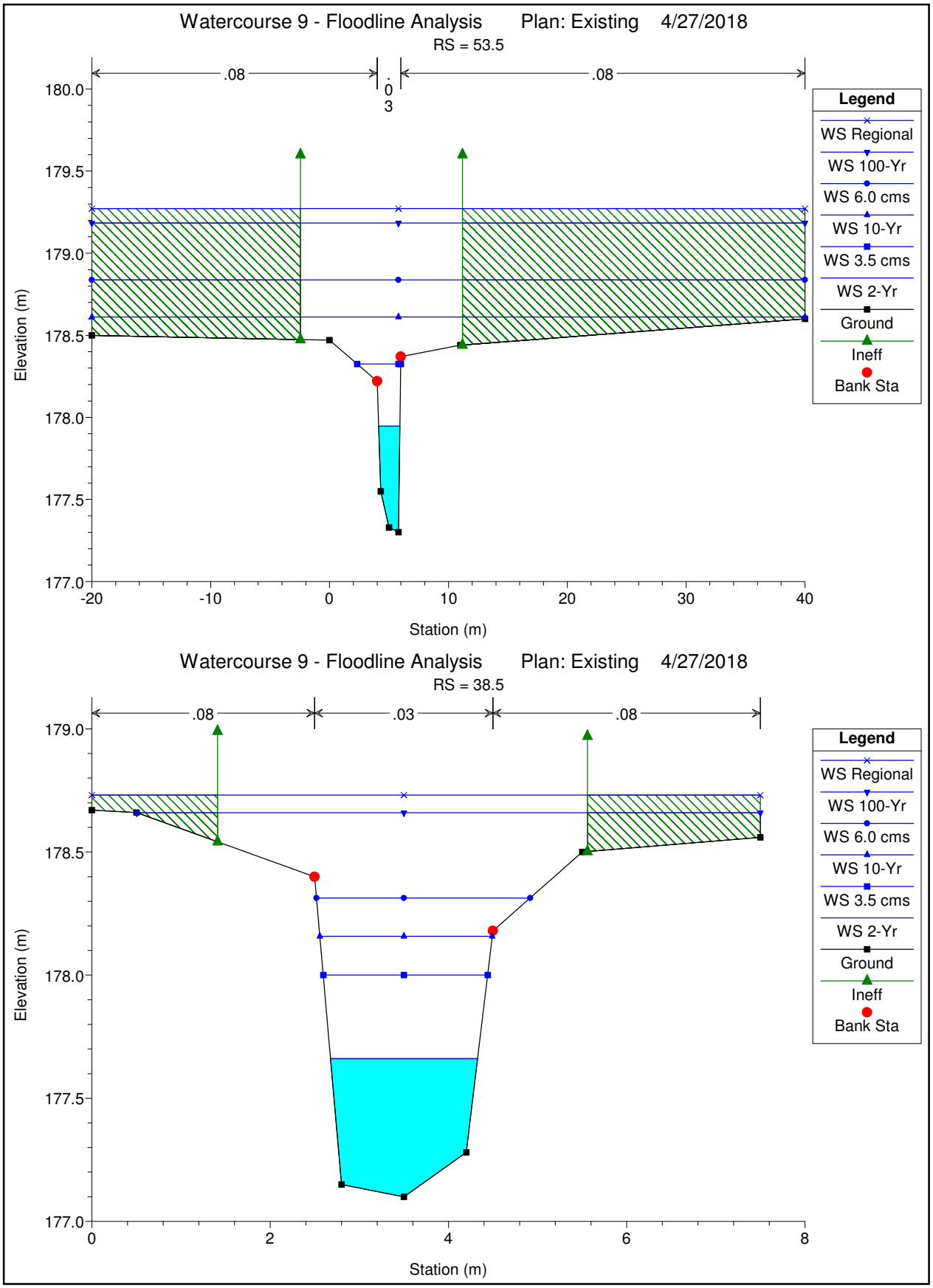

Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 172 Culv Lakeshore Rd
.1
.1
.05
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Bank Sta
Elevation (m)
183.0
182.5
182.0
181.5
181.0
180.5
180.0
-200
-150
-100
-50
0
50
100
150
200
Station (m)
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 162
.1
.1
.05
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Bank Sta
Elevation (m)
183.0
182.5
182.0
181.5
181.0
180.5
180.0
-200
-150
-100
-50
0
50
100
150
200
Station (m)

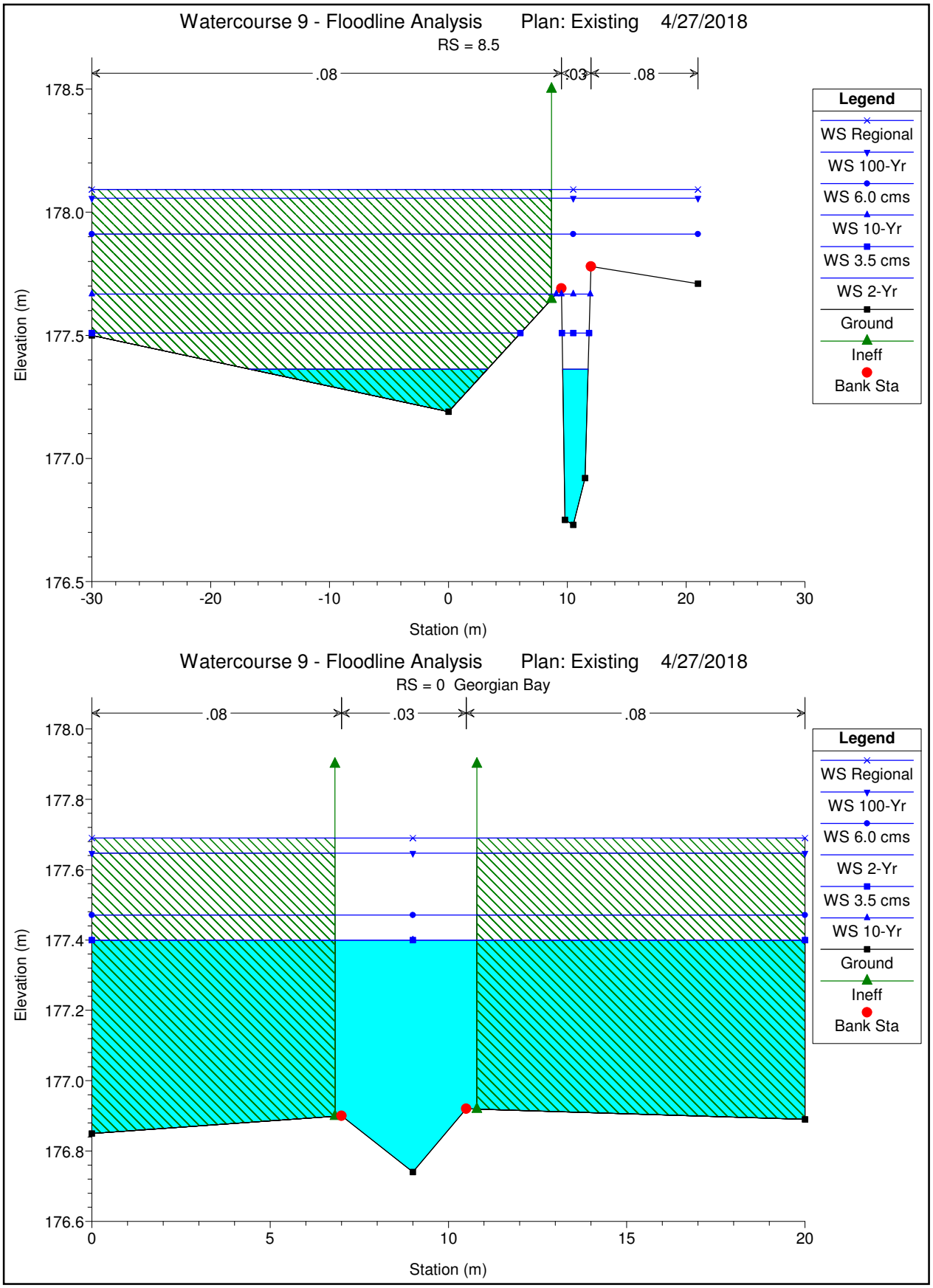

Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 141.5
.1
.05
.1
181.0
180.5
180.0
179.5
179.0
178.5
Elevation (m)
-20
-10
0
10
20
30
40
50
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 134 Culv Georgian Trail
.1
.05
.1
181.0
180.5
180.0
179.5
179.0
178.5
Elevation (m)
-20
-10
0
10
20
30
40
50
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 134 Culv Georgian Trail
.1
.05
.1
180.5
180.0
179.5
179.0
178.5
178.0
Elevation (m)
-40
-20
0
20
40
60
80
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 126.5
.1
.05
.1
180.0
179.5
179.0
178.5
178.0
Elevation (m)
-40
-20
0
20
40
60
80
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 115.5 Culv Highway 26
.1
.05
.1
180.0
179.5
179.0
178.5
178.0
Elevation (m)
-40
-20
0
20
40
60
80
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 3.5 cms
WS 10-Yr
WS 2-Yr
Ground
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 115.5 Culv Highway 26
.08
.03
.08
180.0
179.8
179.6
179.4
179.2
179.0
178.8
178.6
178.4
178.2
Elevation (m)
-20
-10
0
10
20
30
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta

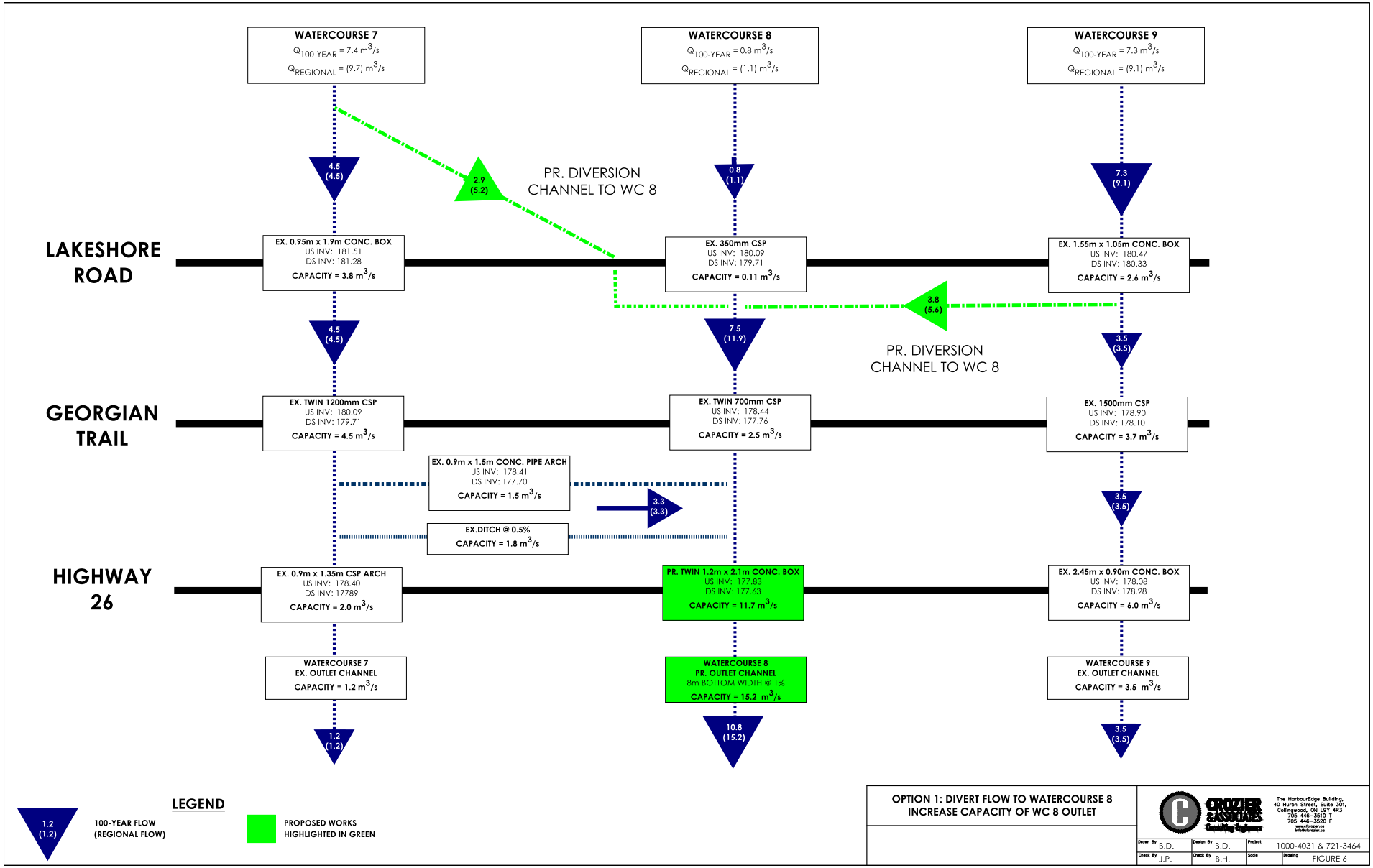

Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 104.5 Immediately downstream of Hwy 26
.08
.03
.08
Elevation (m)
179.8
179.6
179.4
179.2
179.0
178.8
178.6
178.4
178.2
-20
-10
0
10
20
30
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 98 Hwy 26 Downstream
.08
.03
.08
Elevation (m)
180.0
179.5
179.0
178.5
178.0
177.5
-30
-20
-10
0
10
20
30
40
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 53.5
.08
.03
.08
Elevation (m)
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 38.5
.08
.03
.08
Elevation (m)
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 8.5
.08
.03
.08
178.5
178.0
177.5
177.0
176.5
Elevation (m)
-30
-20
-10
0
10
20
30
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Existing 4/27/2018
RS = 0 Georgian Bay
.08
.03
.08
178.0
177.8
177.6
177.4
177.2
177.0
176.8
176.6
Elevation (m)
0
5
10
15
20
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 2-Yr
WS 3.5 cms
WS 10-Yr
Ground
Ineff
Bank Sta

APPENDIX E

Improvement Options & Calculations


WATERCOURSE 7
$Q_{100\text{-YEAR}}$ = 7.4 m^3/s
$Q_{REGIONAL}$ = (9.7) m^3/s
WATERCOURSE 8
$Q_{100\text{-YEAR}}$ = 0.8 m^3/s
$Q_{REGIONAL}$ = (1.1) m^3/s
WATERCOURSE 9
$Q_{100\text{-YEAR}}$ = 7.3 m^3/s
$Q_{REGIONAL}$ = (9.1) m^3/s
4.5 (4.5)
2.9 (5.2)
PR. DIVERSION CHANNEL TO WC 8
0.8 (1.1)
7.3 (9.1)
LAKESHORE ROAD
EX. 0.95m x 1.9m CONC. BOX
US INV: 181.51
DS INV: 181.28
CAPACITY = 3.8 m^3/s
EX. 350mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 0.11 m^3/s
EX. 1.55m x 1.05m CONC. BOX
US INV: 180.47
DS INV: 180.33
CAPACITY = 2.6 m^3/s
3.8 (5.6)
4.5 (4.5)
7.5 (11.9)
PR. DIVERSION CHANNEL TO WC 8
3.5 (3.5)
GEORGIAN TRAIL
EX. TWIN 1200mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 4.5 m^3/s
EX. TWIN 700mm CSP
US INV: 178.44
DS INV: 177.76
CAPACITY = 2.5 m^3/s
EX. 1500mm CSP
US INV: 178.90
DS INV: 178.10
CAPACITY = 3.7 m^3/s
EX. 0.9m x 1.5m CONC. PIPE ARCH
US INV: 178.41
DS INV: 177.70
CAPACITY = 1.5 m^3/s
3.3 (3.3)
3.5 (3.5)
EX.DITCH @ 0.5%
CAPACITY = 1.8 m^3/s
HIGHWAY 26
EX. 0.9m x 1.35m CSP ARCH
US INV: 178.40
DS INV: 17789
CAPACITY = 2.0 m^3/s
PR. TWIN 1.2m x 2.1m CONC. BOX
US INV: 177.83
DS INV: 177.63
CAPACITY = 11.7 m^3/s
EX. 2.45m x 0.90m CONC. BOX
US INV: 178.08
DS INV: 178.28
CAPACITY = 6.0 m^3/s
WATERCOURSE 7
EX. OUTLET CHANNEL
CAPACITY = 1.2 m^3/s
WATERCOURSE 8
PR. OUTLET CHANNEL
8m BOTTOM WIDTH @ 1%
CAPACITY = 15.2 m^3/s
WATERCOURSE 9
EX. OUTLET CHANNEL
CAPACITY = 3.5 m^3/s
1.2 (1.2)
10.8 (15.2)
3.5 (3.5)
LEGEND
1.2 (1.2)
100-YEAR FLOW (REGIONAL FLOW)
PROPOSED WORKS HIGHLIGHTED IN GREEN
OPTION 1: DIVERT FLOW TO WATERCOURSE 8
INCREASE CAPACITY OF WC 8 OUTLET
CROZIER & ASSOCIATES
Consulting Engineers
The HarbourEdge Building,
40 Huron Street, Suite 301,
Collingwood, ON L9Y 4R3
705 446-3510 T
705 446-3520 F
www.crozier.ca
info@crozier.ca
Drawn By B.D.
Design By B.D.
Project 1000-4031 & 721-3464
Check By J.P.
Check By B.H.
Scale
Drawing FIGURE 6

Culvert Calculator Report
WC 8 HWY 26 - Improvement (Option 1)

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.31	m	Headwater Depth/Height	1.21	
Computed Headwater Elev	179.31	m	Discharge	11.7490	m³/s
Inlet Control HW Elev.	179.31	m	Tailwater Elevation	178.43	m
Outlet Control HW Elev.	179.30	m	Control Type	Inlet Control	

Grades					
Upstream Invert	177.83	m	Downstream Invert	177.63	m
Length	25.00	m	Constructed Slope	0.008000	m/m

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.76	m
Slope Type	Steep		Normal Depth	0.71	m
Flow Regime	Supercritical		Critical Depth	0.92	m
Velocity Downstream	3.64	m/s	Critical Slope	0.003902	m/m

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	2.13	m
Section Size	2130 x 1220 mm		Rise	1.22	m
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	179.30	m	Upstream Velocity Head	0.46	m
Ke	0.20		Entrance Loss	0.09	m

Inlet Control Properties					
Inlet Control HW Elev.	179.31	m	Flow Control	Transition	
Inlet Type	90° headwall w 45° bevels		Area Full	5.2	m²
K	0.49500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	2	
C	0.03140		Equation Form	2	
Y	0.82000				

WC8 OUTLET - Improved (Option 1)

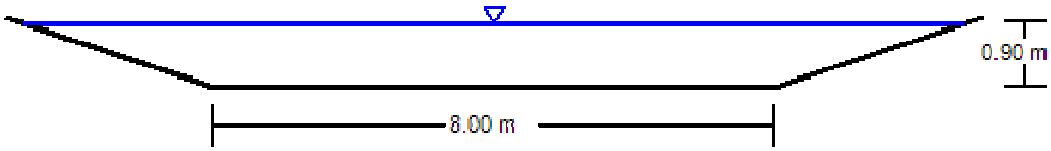
Project Description

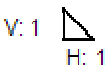
Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.050	
Channel Slope	0.01000	m/m
Normal Depth	0.90	m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	8.00	m
Discharge	15.23	m³/s

Cross Section Image





WC8 OUTLET - IMPROVED

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.050	
Channel Slope	0.01000	m/m
Normal Depth	0.90	m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	8.00	m

Results

Discharge	15.23	m^3/s
Flow Area	9.63	m^2
Wetted Perimeter	13.69	m
Hydraulic Radius	0.70	m
Top Width	13.40	m
Critical Depth	0.66	m
Critical Slope	0.03065	m/m
Velocity	1.58	m/s
Velocity Head	0.13	m
Specific Energy	1.03	m
Froude Number	0.60	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

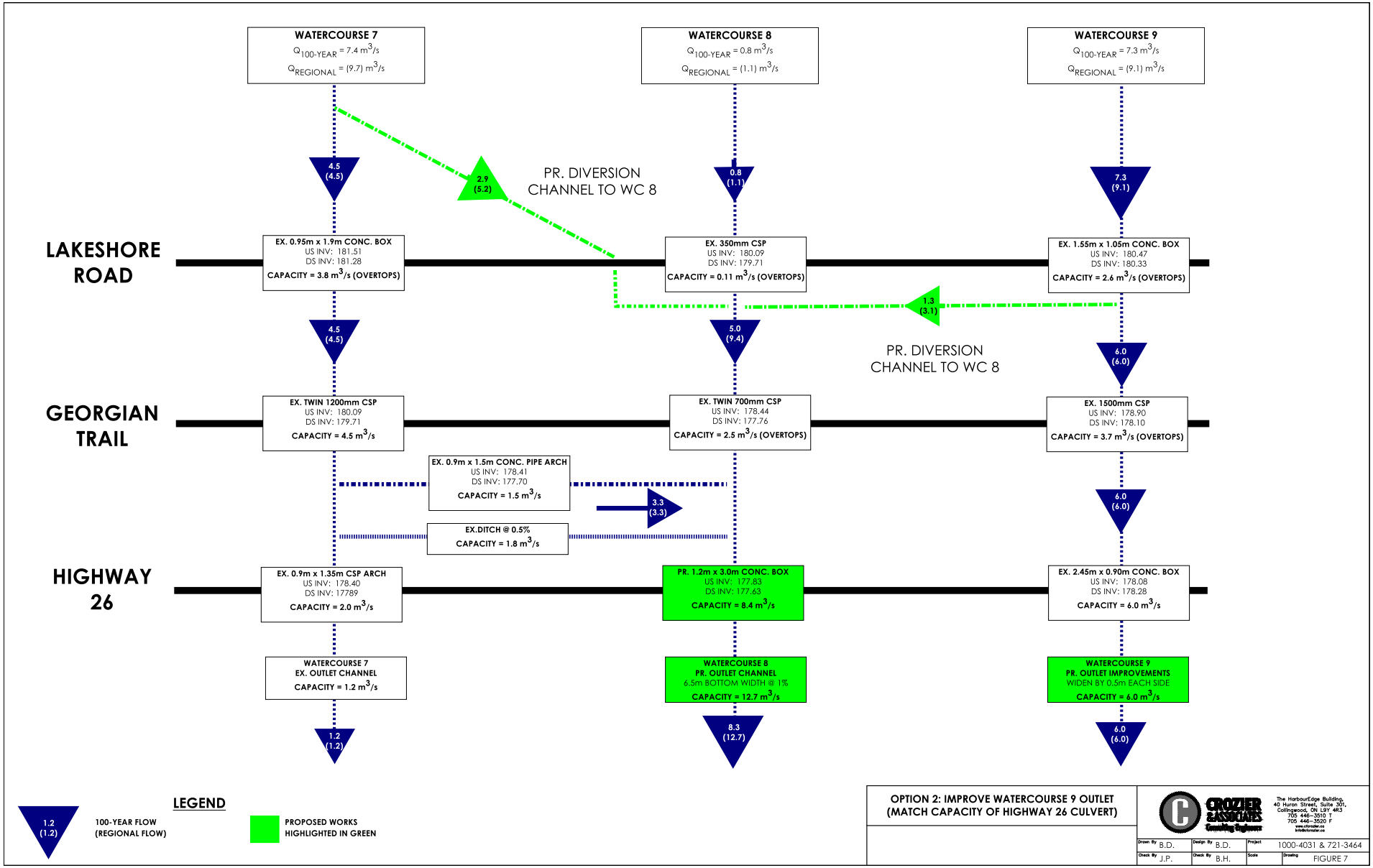
GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.90	m
Critical Depth	0.66	m
Channel Slope	0.01000	m/m

WC8 OUTLET - IMPROVED

GVF Output Data

Critical Slope	0.03065	m/m


WATERCOURSE 7
$Q_{100\text{-YEAR}}$ = 7.4 m^3/s
$Q_{REGIONAL}$ = (9.7) m^3/s
WATERCOURSE 8
$Q_{100\text{-YEAR}}$ = 0.8 m^3/s
$Q_{REGIONAL}$ = (1.1) m^3/s
WATERCOURSE 9
$Q_{100\text{-YEAR}}$ = 7.3 m^3/s
$Q_{REGIONAL}$ = (9.1) m^3/s
4.5 (4.5)
2.9 (5.2)
PR. DIVERSION CHANNEL TO WC 8
0.8 (1.1)
7.3 (9.1)
LAKESHORE ROAD
EX. 0.95m x 1.9m CONC. BOX
US INV: 181.51
DS INV: 181.28
CAPACITY = 3.8 m^3/s (OVERTOPS)
EX. 350mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 0.11 m^3/s (OVERTOPS)
EX. 1.55m x 1.05m CONC. BOX
US INV: 180.47
DS INV: 180.33
CAPACITY = 2.6 m^3/s (OVERTOPS)
1.3 (3.1)
4.5 (4.5)
5.0 (9.4)
PR. DIVERSION CHANNEL TO WC 8
6.0 (6.0)
GEORGIAN TRAIL
EX. TWIN 1200mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 4.5 m^3/s
EX. TWIN 700mm CSP
US INV: 178.44
DS INV: 177.76
CAPACITY = 2.5 m^3/s (OVERTOPS)
EX. 1500mm CSP
US INV: 178.90
DS INV: 178.10
CAPACITY = 3.7 m^3/s (OVERTOPS)
EX. 0.9m x 1.5m CONC. PIPE ARCH
US INV: 178.41
DS INV: 177.70
CAPACITY = 1.5 m^3/s
3.3 (3.3)
EX.DITCH @ 0.5%
CAPACITY = 1.8 m^3/s
6.0 (6.0)
HIGHWAY 26
EX. 0.9m x 1.35m CSP ARCH
US INV: 178.40
DS INV: 17789
CAPACITY = 2.0 m^3/s
PR. 1.2m x 3.0m CONC. BOX
US INV: 177.83
DS INV: 177.63
CAPACITY = 8.4 m^3/s
EX. 2.45m x 0.90m CONC. BOX
US INV: 178.08
DS INV: 178.28
CAPACITY = 6.0 m^3/s
WATERCOURSE 7
EX. OUTLET CHANNEL
CAPACITY = 1.2 m^3/s
WATERCOURSE 8
PR. OUTLET CHANNEL
6.5m BOTTOM WIDTH @ 1%
CAPACITY = 12.7 m^3/s
WATERCOURSE 9
PR. OUTLET IMPROVEMENTS
WIDEN BY 0.5m EACH SIDE
CAPACITY = 6.0 m^3/s
1.2 (1.2)
8.3 (12.7)
6.0 (6.0)
LEGEND
1.2 (1.2)
100-YEAR FLOW (REGIONAL FLOW)
PROPOSED WORKS HIGHLIGHTED IN GREEN
OPTION 2: IMPROVE WATERCOURSE 9 OUTLET (MATCH CAPACITY OF HIGHWAY 26 CULVERT)
C CROZIER & ASSOCIATES Consulting Engineers
The HarbourEdge Building, 40 Huron Street, Suite 301, Collingwood, ON L9Y 4R3
705 446-3510 T
705 446-3520 F
www.crozier.ca
info@crozier.ca
Drawn By B.D.
Design By B.D.
Project 1000-4031 & 721-3464
Check By J.P.
Check By B.H.
Scale
Drawing FIGURE 7

Culvert Calculator Report
WC 8 Hwy. 26 - Improved (Option 2)

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.31	m	Headwater Depth/Height	1.21	
Computed Headwater Elevation	179.31	m	Discharge	8.3921	m³/s
Inlet Control HW Elev.	179.31	m	Tailwater Elevation	178.45	m
Outlet Control HW Elev.	179.30	m	Control Type	Inlet Control	

Grades					
Upstream Invert	177.83	m	Downstream Invert	177.63	m
Length	25.00	m	Constructed Slope	0.008000	m/m

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.74	m
Slope Type	Steep		Normal Depth	0.67	m
Flow Regime	Supercritical		Critical Depth	0.92	m
Velocity Downstream	3.74	m/s	Critical Slope	0.003197	m/m

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	3.05	m
Section Size	3050 x 1220 mm		Rise	1.22	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	179.30	m	Upstream Velocity Head	0.46	m
Ke	0.20		Entrance Loss	0.09	m

Inlet Control Properties					
Inlet Control HW Elev.	179.31	m	Flow Control	N/A	
Inlet Type	90° headwall w 45° bevels		Area Full	3.7	m²
K	0.49500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	2	
C	0.03140		Equation Form	2	
Y	0.82000				

WC8 OUTLET - Improved (Option 2)

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.050	
Channel Slope	0.01000	m/m
Normal Depth	0.90	m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	6.50	m

Results

Discharge	12.79	m^3/s
Flow Area	8.28	m^2
Wetted Perimeter	12.19	m
Hydraulic Radius	0.68	m
Top Width	11.90	m
Critical Depth	0.66	m
Critical Slope	0.03104	m/m
Velocity	1.55	m/s
Velocity Head	0.12	m
Specific Energy	1.02	m
Froude Number	0.59	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.90	m
Critical Depth	0.66	m
Channel Slope	0.01000	m/m

WC8 OUTLET - Improved (Option 2)

GVF Output Data

Critical Slope	0.03104	m/m

WC8 OUTLET - Improvement (Option 2)

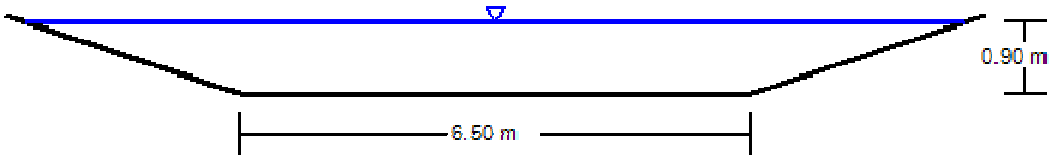
Project Description

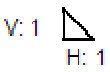
Friction Method	Manning Formula
Solve For	Discharge

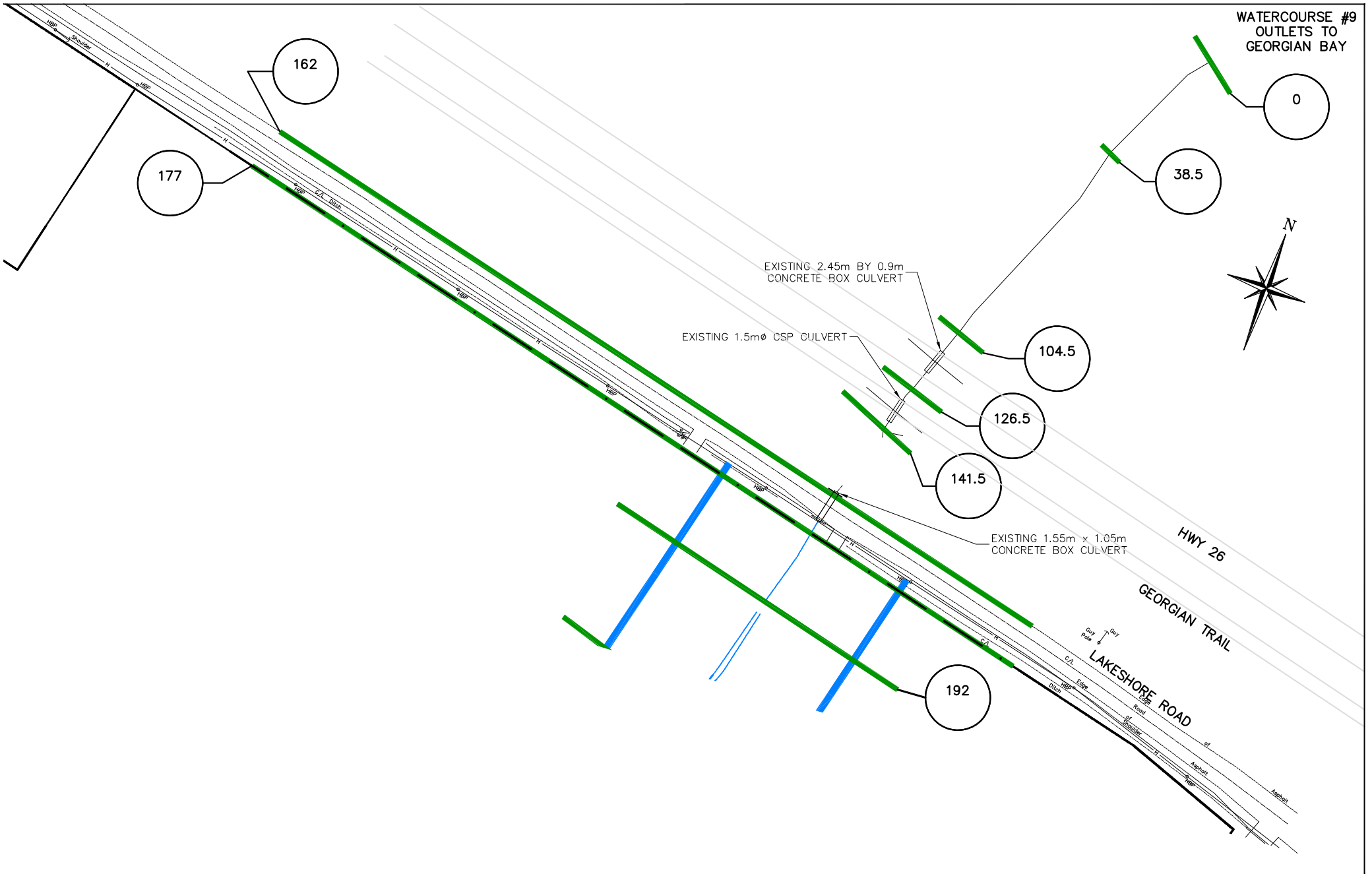
Input Data

Roughness Coefficient	0.050	
Channel Slope	0.01000	m/m
Normal Depth	0.90	m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	6.50	m
Discharge	12.79	m³/s

Cross Section Image







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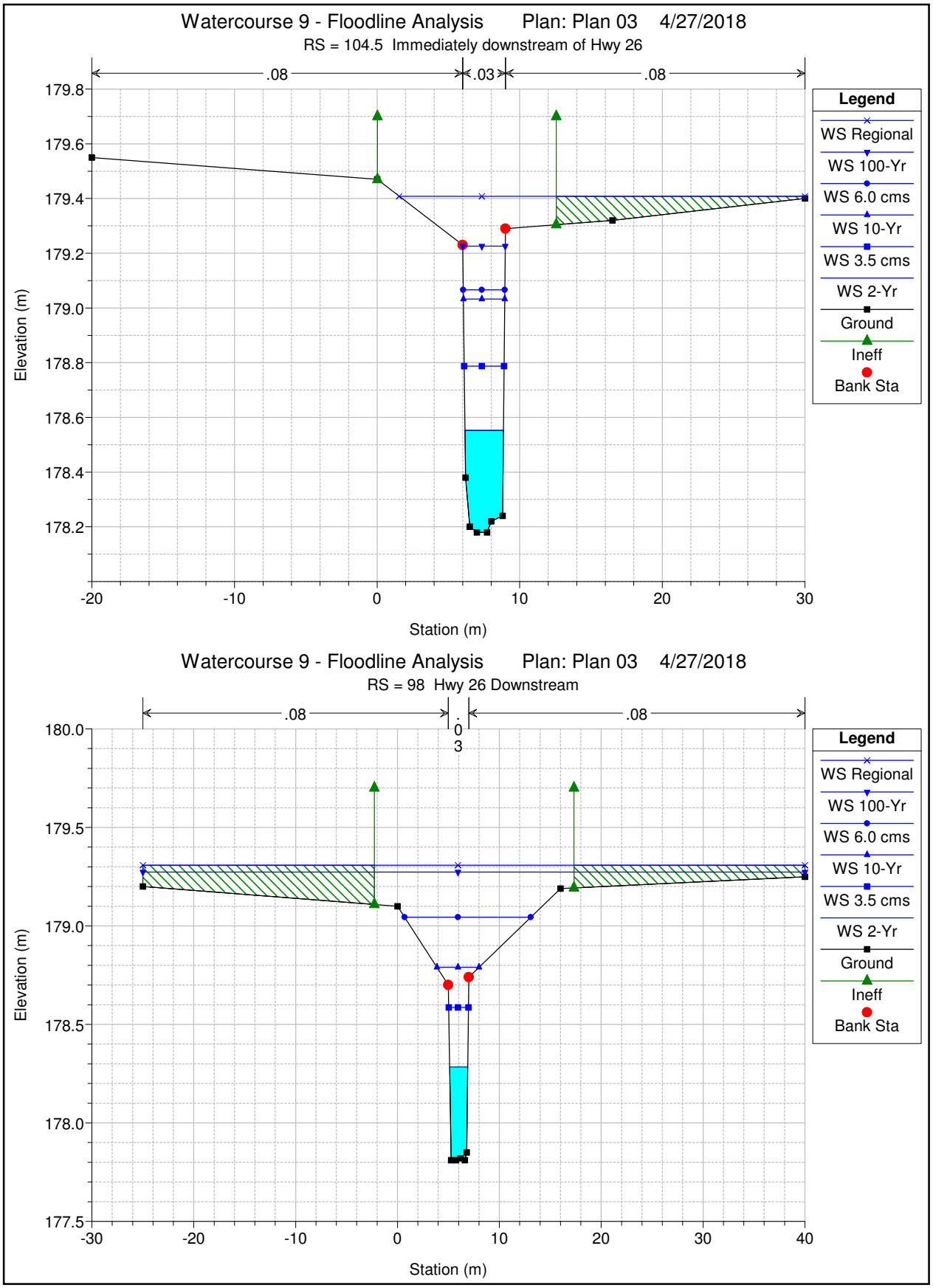
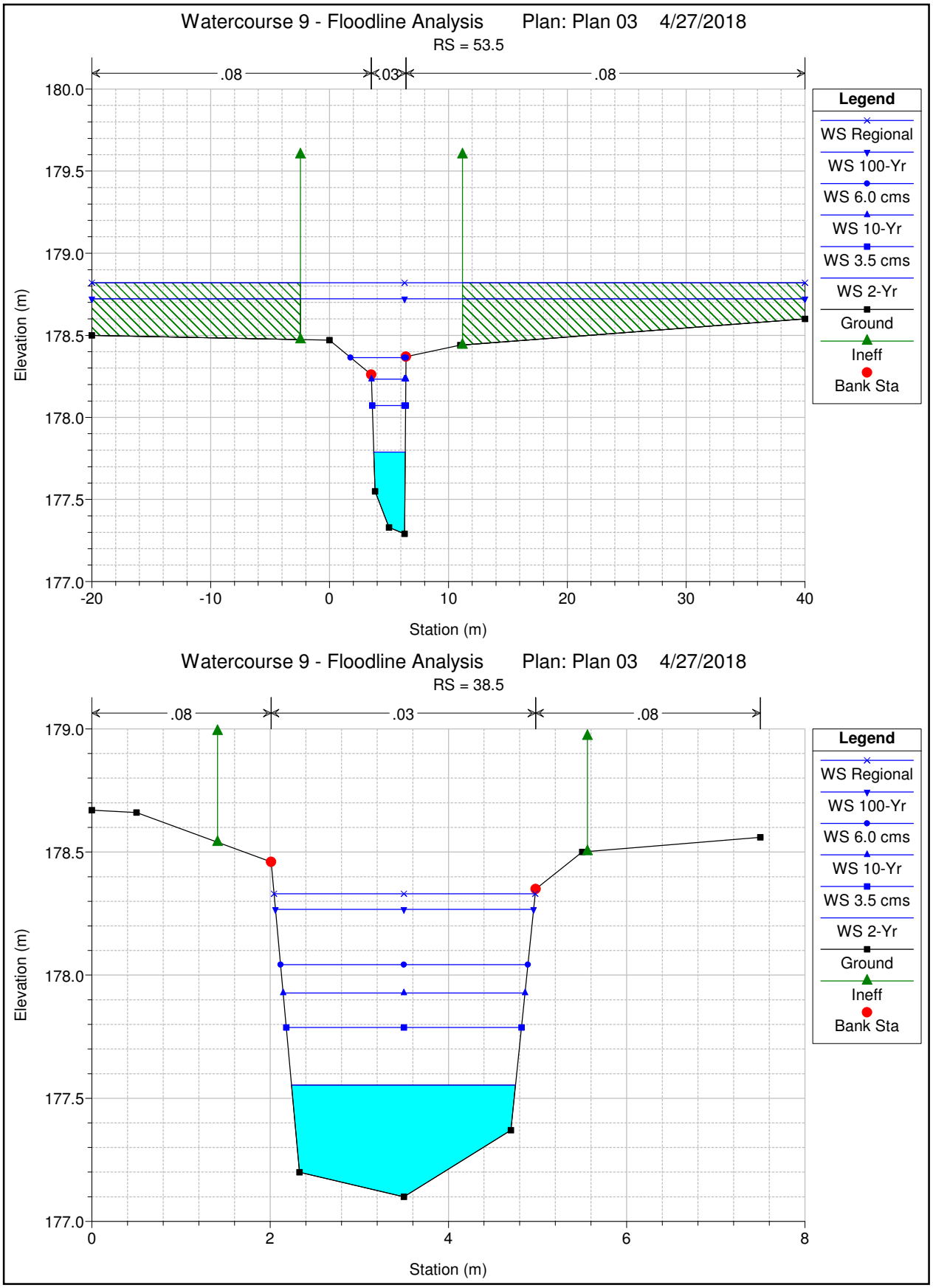
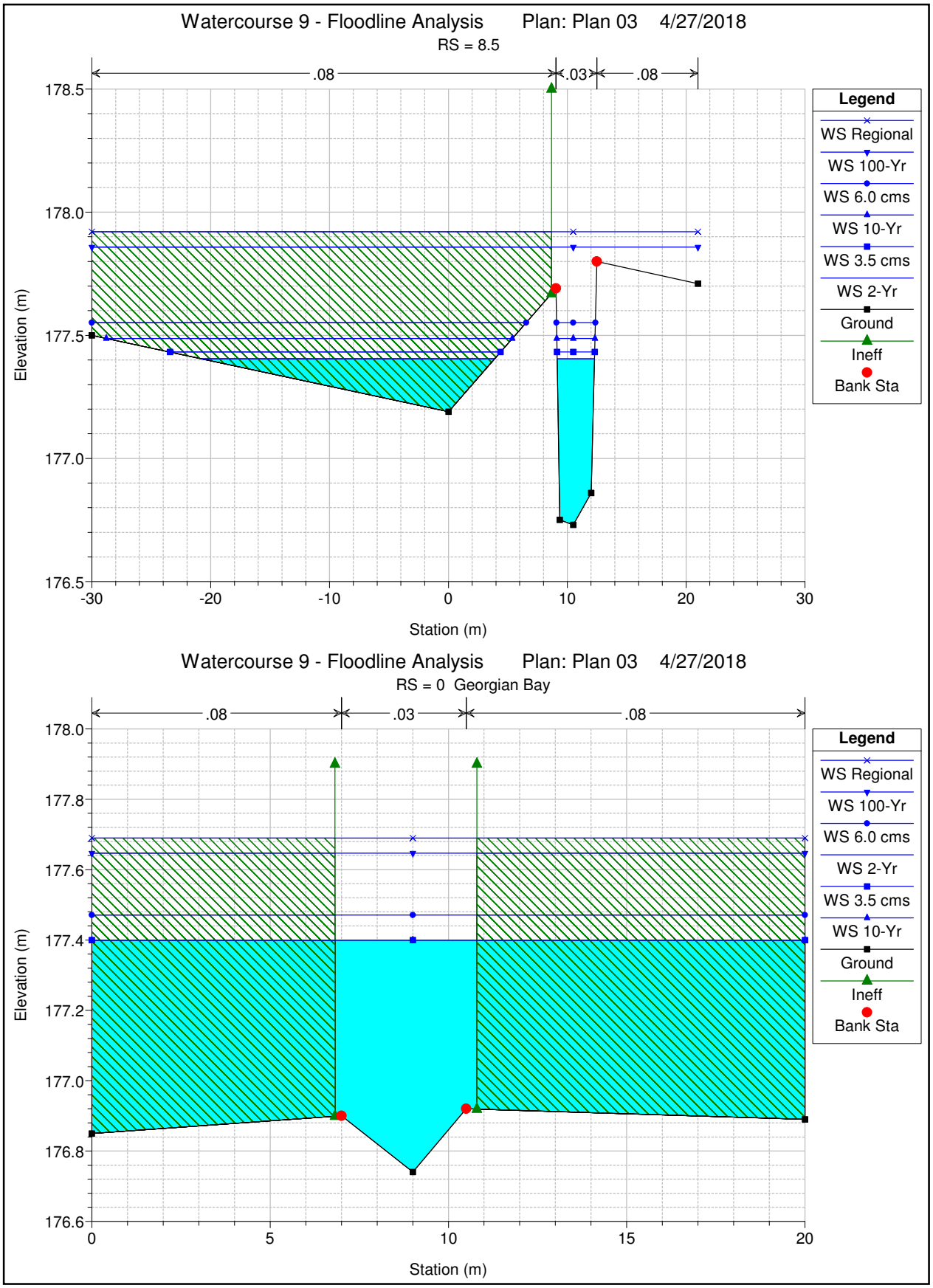
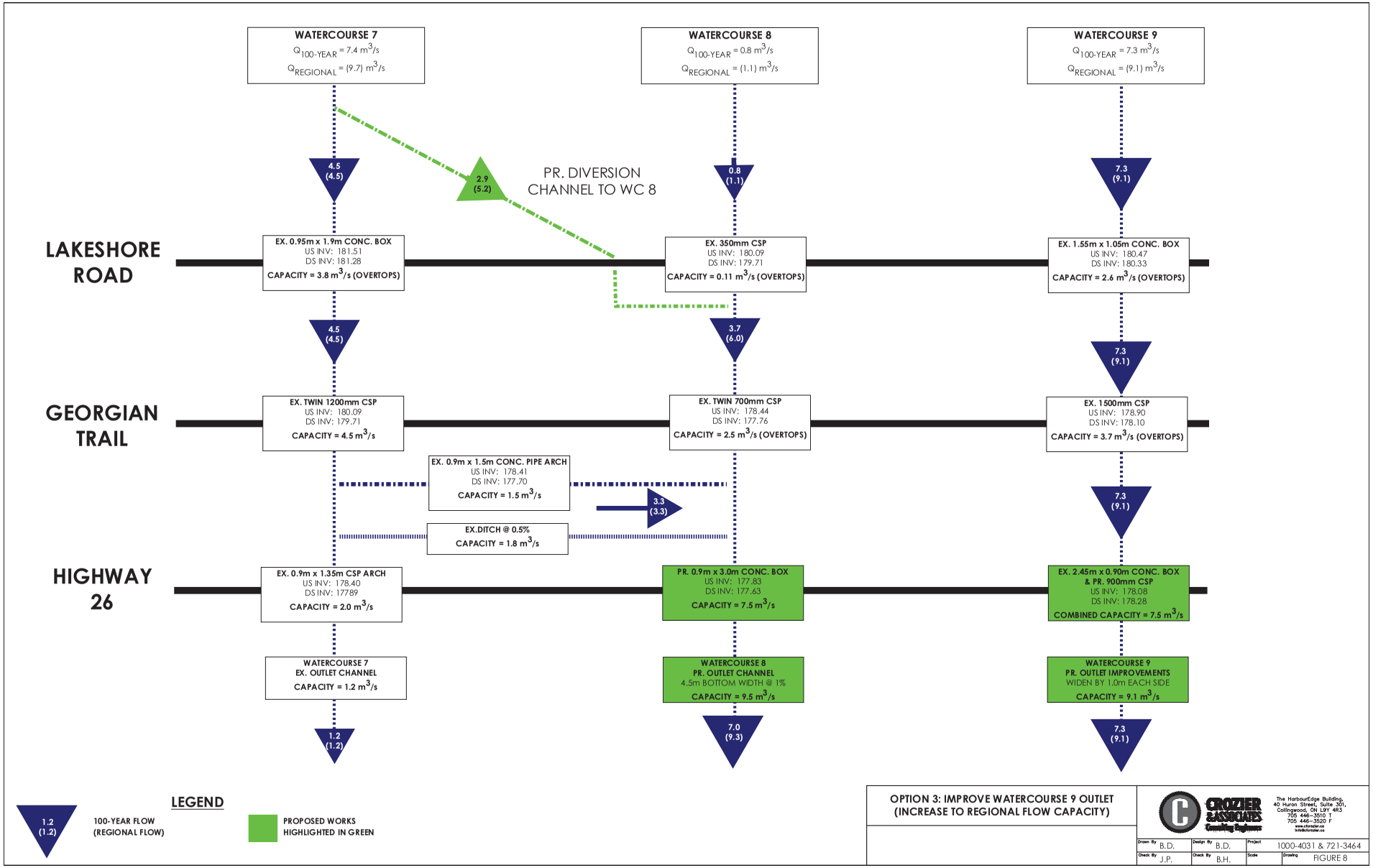

Watercourse 9 - Floodline Analysis Plan: Plan 03 4/27/2018
RS = 104.5 Immediately downstream of Hwy 26
.08
.03
.08
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Elevation (m)
Station (m)
Watercourse 9 - Floodline Analysis Plan: Plan 03 4/27/2018
RS = 98 Hwy 26 Downstream
.08
.03
.08
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Elevation (m)
Station (m)


Watercourse 9 - Floodline Analysis Plan: Plan 03 4/27/2018
RS = 53.5
.08
.03
.08
180.0
179.5
179.0
178.5
178.0
177.5
177.0
Elevation (m)
-20
-10
0
10
20
30
40
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Plan 03 4/27/2018
RS = 38.5
.08
.03
.08
179.0
178.5
178.0
177.5
177.0
Elevation (m)
0
2
4
6
8
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Plan 03 4/27/2018
RS = 8.5
.08
.03
.08
178.5
178.0
177.5
177.0
176.5
Elevation (m)
-30
-20
-10
0
10
20
30
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Plan 03 4/27/2018
RS = 0 Georgian Bay
.08
.03
.08
178.0
177.8
177.6
177.4
177.2
177.0
176.8
176.6
Elevation (m)
0
5
10
15
20
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 2-Yr
WS 3.5 cms
WS 10-Yr
Ground
Ineff
Bank Sta


WATERCOURSE 7
$Q_{100\text{-YEAR}}$ = 7.4 m^3/s
$Q_{REGIONAL}$ = (9.7) m^3/s
WATERCOURSE 8
$Q_{100\text{-YEAR}}$ = 0.8 m^3/s
$Q_{REGIONAL}$ = (1.1) m^3/s
WATERCOURSE 9
$Q_{100\text{-YEAR}}$ = 7.3 m^3/s
$Q_{REGIONAL}$ = (9.1) m^3/s
4.5 (4.5)
2.9 (5.2)
PR. DIVERSION CHANNEL TO WC 8
0.8 (1.1)
7.3 (9.1)
LAKESHORE ROAD
EX. 0.95m x 1.9m CONC. BOX
US INV: 181.51
DS INV: 181.28
CAPACITY = 3.8 m^3/s (OVERTOPS)
EX. 350mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 0.11 m^3/s (OVERTOPS)
EX. 1.55m x 1.05m CONC. BOX
US INV: 180.47
DS INV: 180.33
CAPACITY = 2.6 m^3/s (OVERTOPS)
4.5 (4.5)
3.7 (6.0)
7.3 (9.1)
GEORGIAN TRAIL
EX. TWIN 1200mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 4.5 m^3/s
EX. TWIN 700mm CSP
US INV: 178.44
DS INV: 177.76
CAPACITY = 2.5 m^3/s (OVERTOPS)
EX. 1500mm CSP
US INV: 178.90
DS INV: 178.10
CAPACITY = 3.7 m^3/s (OVERTOPS)
EX. 0.9m x 1.5m CONC. PIPE ARCH
US INV: 178.41
DS INV: 177.70
CAPACITY = 1.5 m^3/s
3.3 (3.3)
7.3 (9.1)
EX.DITCH @ 0.5%
CAPACITY = 1.8 m^3/s
HIGHWAY 26
EX. 0.9m x 1.35m CSP ARCH
US INV: 178.40
DS INV: 17789
CAPACITY = 2.0 m^3/s
PR. 0.9m x 3.0m CONC. BOX
US INV: 177.83
DS INV: 177.63
CAPACITY = 7.5 m^3/s
EX. 2.45m x 0.90m CONC. BOX & PR. 900mm CSP
US INV: 178.08
DS INV: 178.28
COMBINED CAPACITY = 7.5 m^3/s
WATERCOURSE 7
EX. OUTLET CHANNEL
CAPACITY = 1.2 m^3/s
WATERCOURSE 8
PR. OUTLET CHANNEL
4.5m BOTTOM WIDTH @ 1%
CAPACITY = 9.5 m^3/s
WATERCOURSE 9
PR. OUTLET IMPROVEMENTS
WIDEN BY 1.0m EACH SIDE
CAPACITY = 9.1 m^3/s
1.2 (1.2)
7.0 (9.3)
7.3 (9.1)
LEGEND
1.2 (1.2)
100-YEAR FLOW (REGIONAL FLOW)
PROPOSED WORKS HIGHLIGHTED IN GREEN
OPTION 3: IMPROVE WATERCOURSE 9 OUTLET (INCREASE TO REGIONAL FLOW CAPACITY)
C CROZIER & ASSOCIATES Consulting Engineers
The HarbourEdge Building, 40 Huron Street, Suite 301, Collingwood, ON L9Y 4R3
705 446-3510 T
705 446-3520 F
www.crozier.ca
Drawn By B.D.
Design By B.D.
Project 1000-4031 & 721-3464
Check By J.P.
Check By B.H.
Scale
Drawing FIGURE 8

Culvert Calculator Report
WC 8 Hwy. 26 Improvement (Option 3)

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.31	m	Headwater Depth/Height	1.62	
Computed Headwater Elev	179.31	m	Discharge	7.4436	m³/s
Inlet Control HW Elev.	179.31	m	Tailwater Elevation	178.45	m
Outlet Control HW Elev.	179.18	m	Control Type	Inlet Control	

Grades					
Upstream Invert	177.83	m	Downstream Invert	177.63	m
Length	25.00	m	Constructed Slope	0.008000	m/m

Hydraulic Profile					
Profile	S2		Depth, Downstream	0.67	m
Slope Type	Steep		Normal Depth	0.61	m
Flow Regime	Supercritical		Critical Depth	0.85	m
Velocity Downstream	3.62	m/s	Critical Slope	0.003157	m/m

Section					
Section Shape	Box		Mannings Coefficient	0.013	
Section Material	Concrete		Span	3.05	m
Section Size	3050 x 910 mm		Rise	0.91	m
Number Sections	1				

Outlet Control Properties					
Outlet Control HW Elev.	179.18	m	Upstream Velocity Head	0.42	m
Ke	0.20		Entrance Loss	0.08	m

Inlet Control Properties					
Inlet Control HW Elev.	179.31	m	Flow Control	Submerged	
Inlet Type	90° headwall w 45° bevels		Area Full	2.8	m²
K	0.49500		HDS 5 Chart	10	
M	0.66700		HDS 5 Scale	2	
C	0.03140		Equation Form	2	
Y	0.82000				

WC8 OUTLET - Improved (Option 3)

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.050	
Channel Slope	0.01000	m/m
Normal Depth	0.90	m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	4.50	m

Results

Discharge	9.58	m^3/s
Flow Area	6.48	m^2
Wetted Perimeter	10.19	m
Hydraulic Radius	0.64	m
Top Width	9.90	m
Critical Depth	0.66	m
Critical Slope	0.03179	m/m
Velocity	1.48	m/s
Velocity Head	0.11	m
Specific Energy	1.01	m
Froude Number	0.58	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.90	m
Critical Depth	0.66	m
Channel Slope	0.01000	m/m

WC8 OUTLET - Improved (Option 3)

GVF Output Data

Critical Slope	0.03179	m/m

WC8 OUTLET - Improved (Option 3)

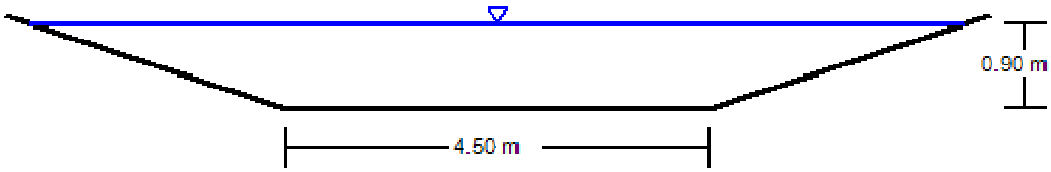
Project Description

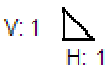
Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.050	
Channel Slope	0.01000	m/m
Normal Depth	0.90	m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	4.50	m
Discharge	9.58	m³/s

Cross Section Image





Culvert Calculator Report
WC 9 HWY. 26 - Improvement (Option 3) - Additional CSP

Solve For: Discharge

Culvert Summary					
Allowable HW Elevation	179.70	m	Headwater Depth/Height	1.77	
Computed Headwater Elev	179.70	m	Discharge	1.5394	m³/s
Inlet Control HW Elev.	179.49	m	Tailwater Elevation	178.92	m
Outlet Control HW Elev.	179.70	m	Control Type	Outlet Control	

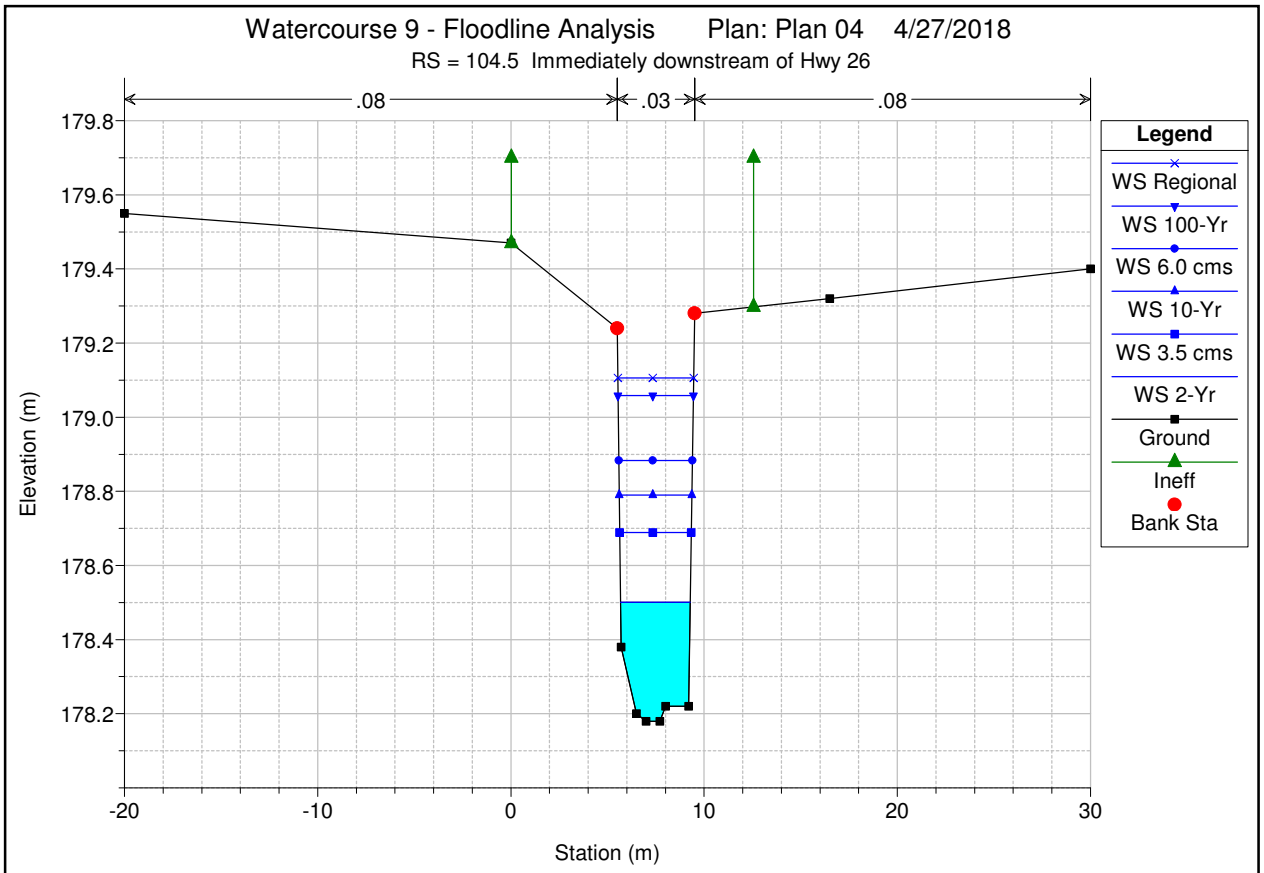
Grades					
Upstream Invert	178.08	m	Downstream Invert	178.28	m
Length	11.30	m	Constructed Slope	-0.017699	m/m

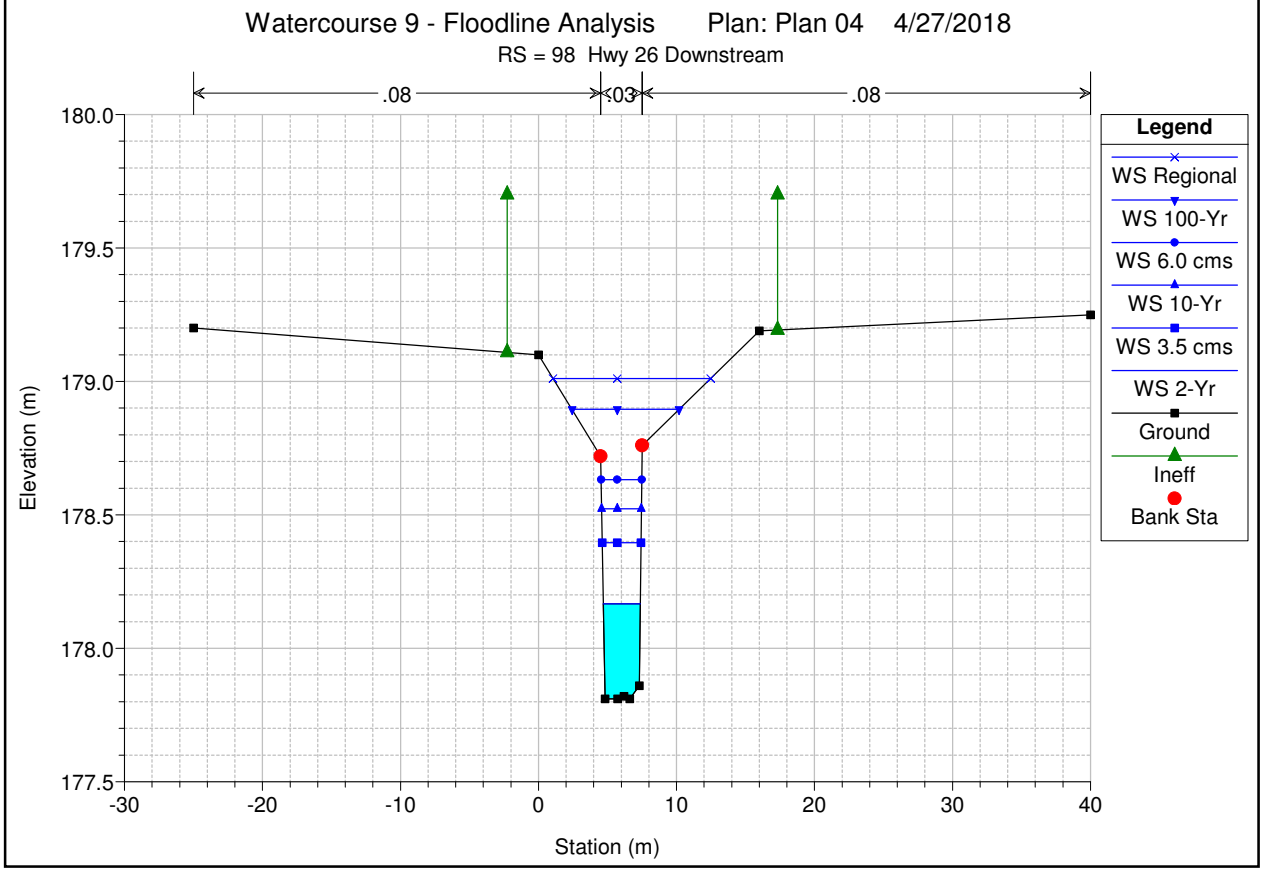
Hydraulic Profile					
Profile	CompositeA2PressureProfile		Depth, Downstream	0.73	m
Slope Type	Adverse		Normal Depth	N/A	m
Flow Regime	Subcritical		Critical Depth	0.73	m
Velocity Downstream	2.74	m/s	Critical Slope	0.023813	m/m

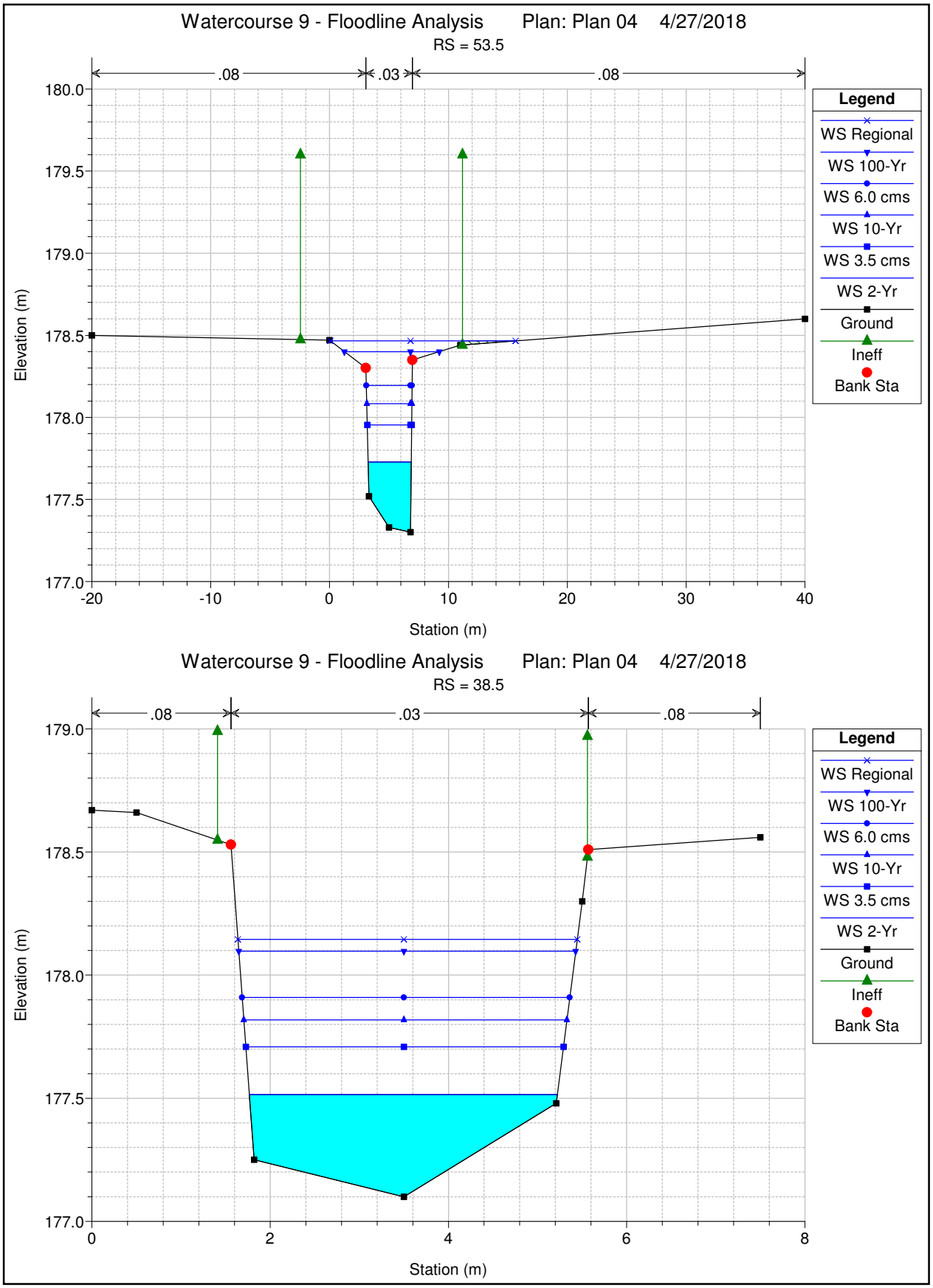
Section					
Section Shape	Circular		Mannings Coefficient	0.024	
Section Material	CMP		Span	0.91	m
Section Size	900 mm		Rise	0.91	m
Number Sections	1				

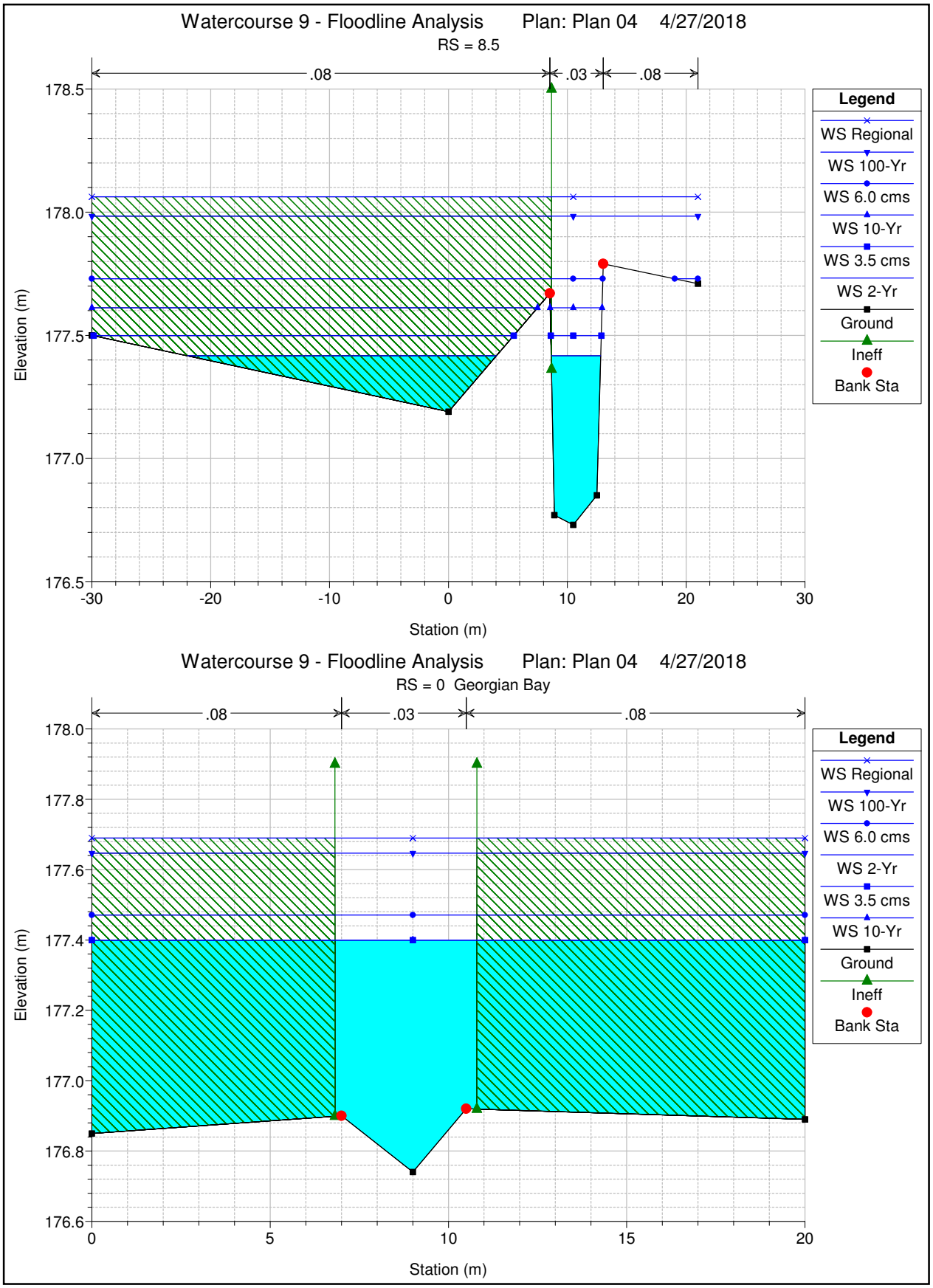
Outlet Control Properties					
Outlet Control HW Elev.	179.70	m	Upstream Velocity Head	0.28	m
Ke	0.20		Entrance Loss	0.06	m

Inlet Control Properties					
Inlet Control HW Elev.	179.49	m	Flow Control	N/A	
Inlet Type	Beveled ring, 33.7° (1.5:1) bevels		Area Full	0.7	m²
K	0.00180		HDS 5 Chart	3	
M	2.50000		HDS 5 Scale	B	
C	0.02430		Equation Form	1	
Y	0.83000				

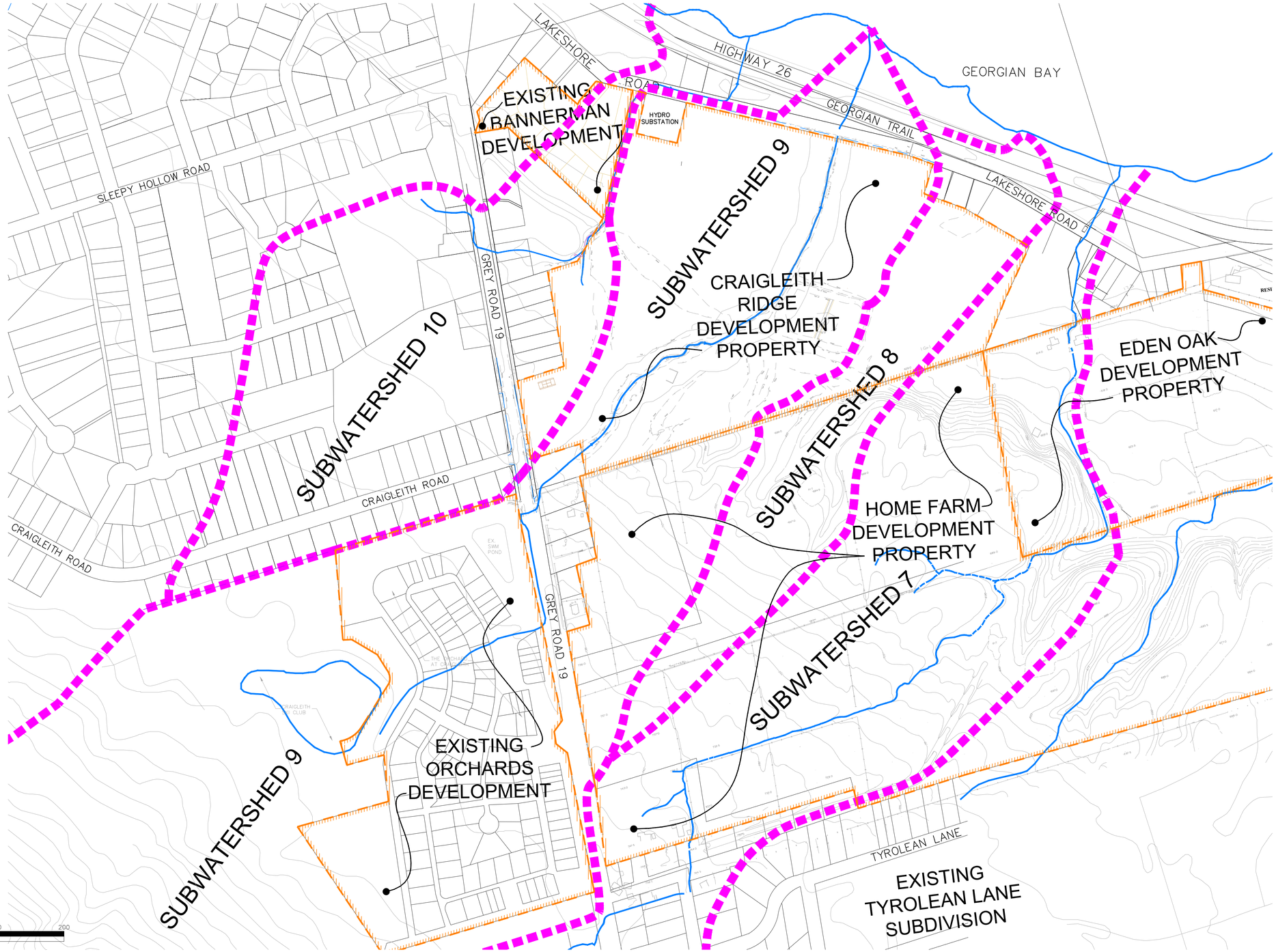

Watercourse 9 - Floodline Analysis Plan: Plan 04 4/27/2018
RS = 104.5 Immediately downstream of Hwy 26
.08
.03
.08
Elevation (m)
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Plan 04 4/27/2018
RS = 98 Hwy 26 Downstream
.08
.03
.08
Elevation (m)
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Plan 04 4/27/2018
RS = 53.5
.08
.03
.08
180.0
179.5
179.0
178.5
178.0
177.5
177.0
Elevation (m)
-20
-10
0
10
20
30
40
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Plan 04 4/27/2018
RS = 38.5
.08
.03
.08
179.0
178.5
178.0
177.5
177.0
Elevation (m)
0
2
4
6
8
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta


Watercourse 9 - Floodline Analysis Plan: Plan 04 4/27/2018
RS = 8.5
.08
.03
.08
178.5
178.0
177.5
177.0
176.5
Elevation (m)
-30
-20
-10
0
10
20
30
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 10-Yr
WS 3.5 cms
WS 2-Yr
Ground
Ineff
Bank Sta
Watercourse 9 - Floodline Analysis Plan: Plan 04 4/27/2018
RS = 0 Georgian Bay
.08
.03
.08
178.0
177.8
177.6
177.4
177.2
177.0
176.8
176.6
Elevation (m)
0
5
10
15
20
Station (m)
Legend
WS Regional
WS 100-Yr
WS 6.0 cms
WS 2-Yr
WS 3.5 cms
WS 10-Yr
Ground
Ineff
Bank Sta

LIST OF FIGURES



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TEMPORARY BENCHMARKS

TBM#1–
TBM#2–
TBM#3–

Town

No.	ISSUE	DATE: MM/DD/YYYY
0	ISSUED FOR CLIENT REVIEW	03/29/2018

Engineer

FOR DISCUSSION
NOT TO BE USED FOR CONSTRUCTION

Engineer

PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

Project

CRAIGLEITH REGIONAL SWM PLAN
TOWN OF THE BLUE MOUNTAINS

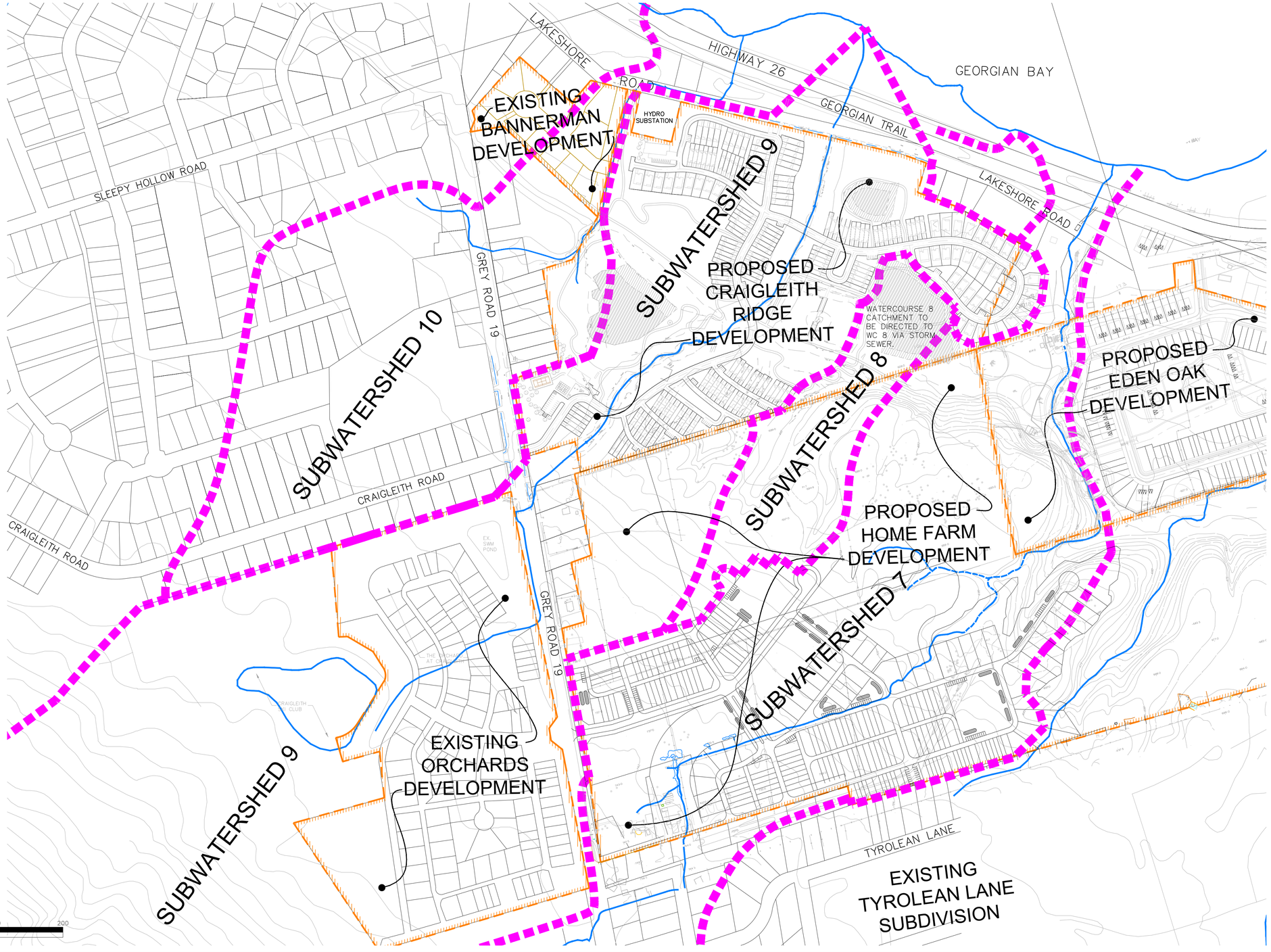
Drawing

EXISTING CONDITIONS
WATERSHED PLAN

CROZIER & ASSOCIATES ENGINEERS
110 Pine Street,
Collingwood, ON L9Y 2N9
705 446–3510 T
705 446–3520 F
www.cfcrozier.ca
info@cfcrozier.ca

Drawn By	Design By	Project
B.K.	B.D./B.R.C.H.	1046–4031
Check By	Check By	Scale / Drawing
B.D.	B.R.C.H.	1:3000 / FIGURE1

J:\1046-Parkbridge Lifestyle\4031-Craigleith Fogal Land Acq\CAD\CIVIL\Figures\FIGURE1.dwg, FIGURE1, 4/27/2018 10:16:36 AM,



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TEMPORARY BENCHMARKS

TBM#1–
TBM#2–
TBM#3–

Town

No.	ISSUE	DATE: MM/DD/YYYY
0	ISSUED FOR CLIENT REVIEW	03/29/2018

Engineer

FOR DISCUSSION
NOT TO BE USED FOR CONSTRUCTION

Engineer

PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

Project

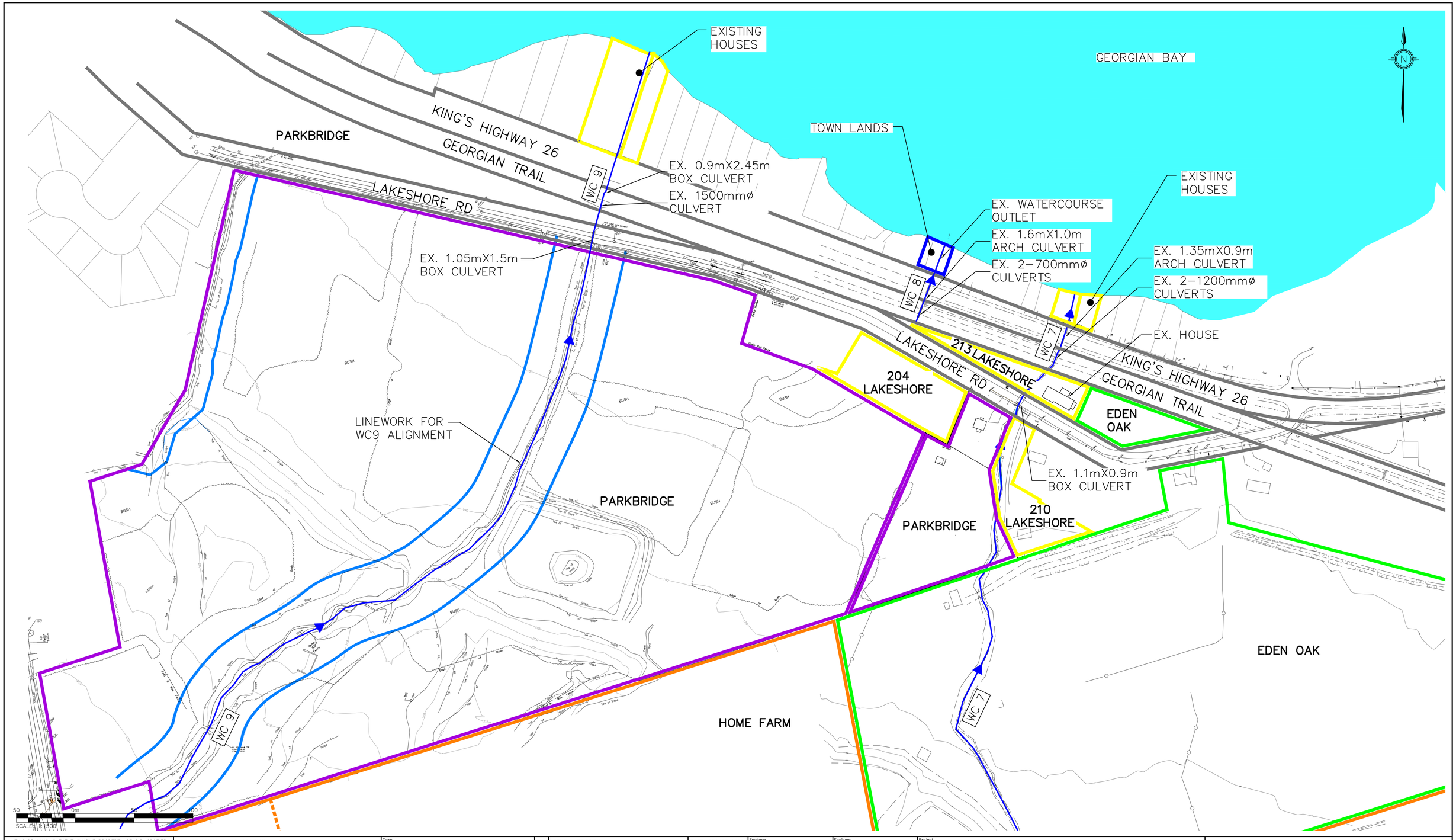
CRAIGLEITH REGIONAL SWM PLAN
TOWN OF THE BLUE MOUNTAINS

Drawing

POST–DEVELOPMENT WATERSHED PLAN

CROZIER & ASSOCIATES ENGINEERS
110 Pine Street,
Collingwood, ON L9Y 2N9
705 446–3510 T
705 446–3520 F
www.cfcrozier.ca
info@cfcrozier.ca

Drawn By	Design By	Project
B.K.	B.D. /B.R.C.H.	1046–4031
Check By	Check By	Scale	Drawing
B.D.	B.R.C.H.	1: 3000	FIGURE2



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TEMPORARY BENCHMARKS

TBM#1–
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Town

No.	ISSUE / REVISION	DATE: MM/DD/YYYY
0	ISSUED FOR CLIENT REVIEW	03/29/2016

Engineer

PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

FOR DISCUSSION
NOT TO BE USED FOR CONSTRUCTION

Engineer

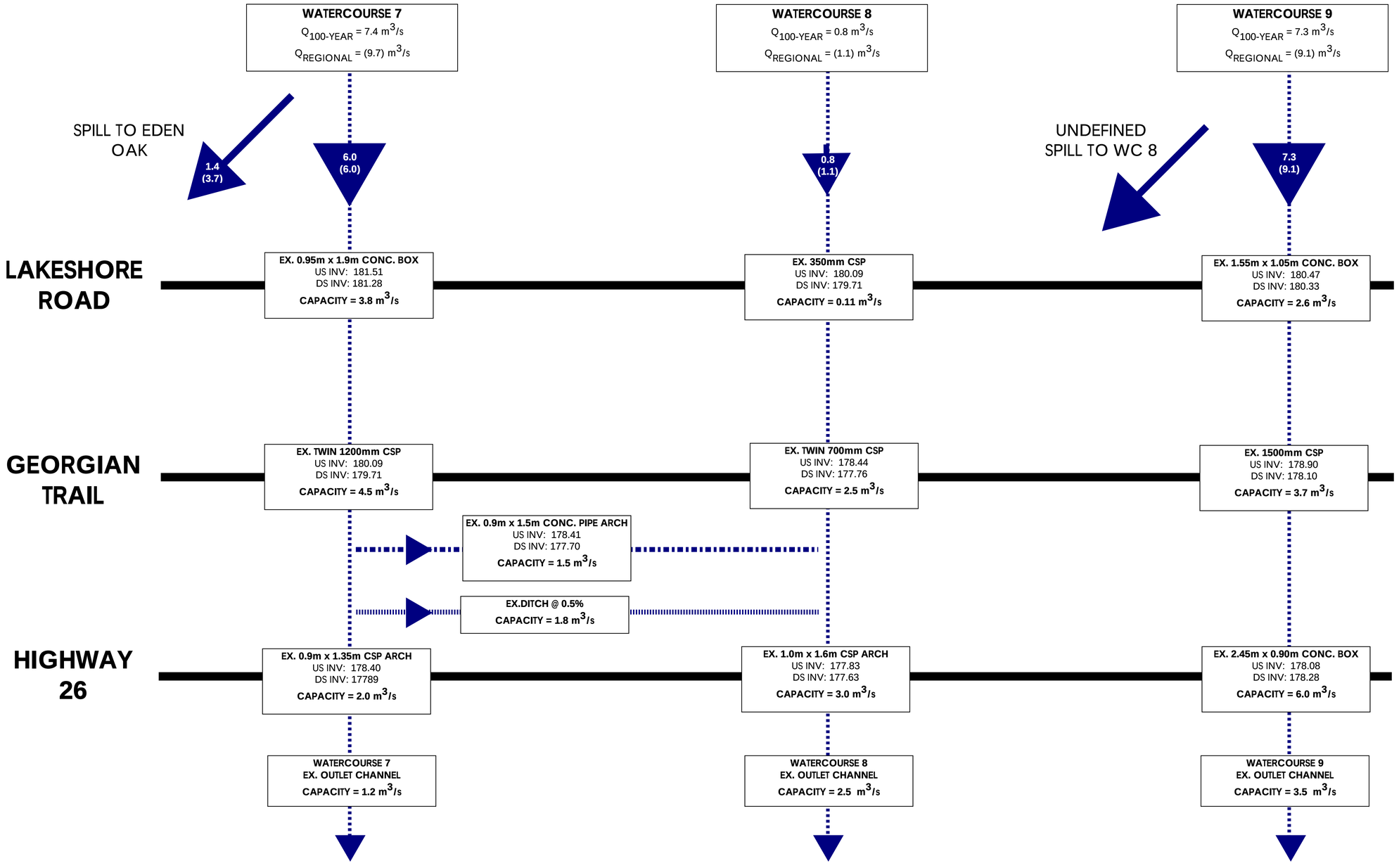
Project: CRAIGLEITH REGIONAL SWM PLAN
TOWN OF THE BLUE MOUNTAINS

Drawing: WATERCOURSE 7, 8 & 9
EXISTING OUTLET INFRASTRUCTURE

CROZIER & ASSOCIATES Consulting Engineers

Drawn By	B.K.	Design By	B.D./B.R.C.H.	Project	1046–4031
Check By	B.D.	Check By	B.R.C.H.	Scale	1:1500
				Drawing	FIGURE3

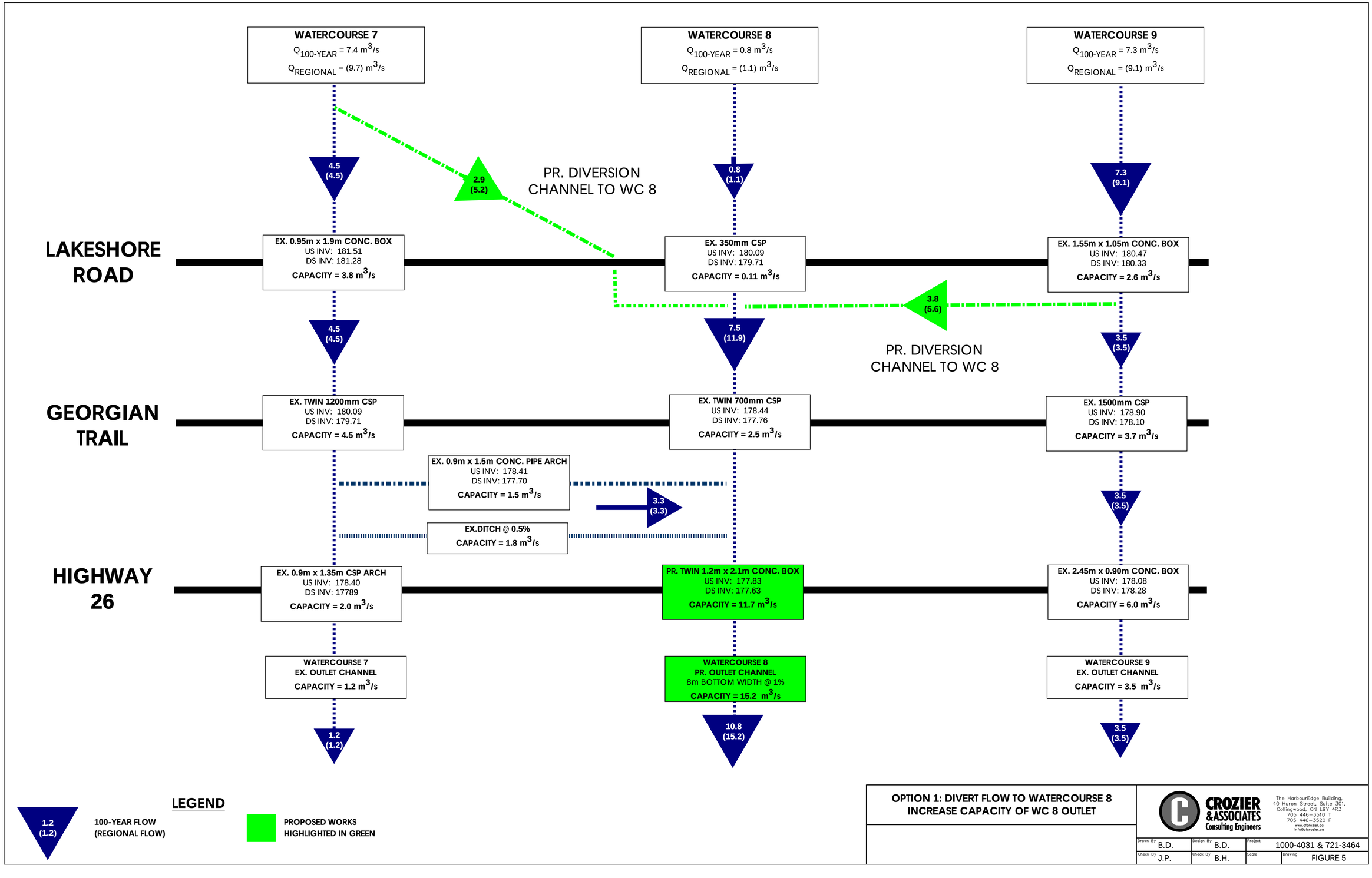
X:\1000\1046-Parkbridge Lifestyle\4031-Craigleith Fogal Land Acq\CAD\CIVIL\Figures\FIGURE4869.dwg, 4/27/2018 1:30:05 PM, DWG To PDF.pc3



EXISTING CONDITIONS

CROZIER & ASSOCIATES Consulting Engineers
The HarbourEdge Building,
40 Huron Street, Suite 301,
Collingwood, ON L9Y 4R3
705 446-3510 T
705 446-3520 F
www.cfcrozier.ca
info@cfcrozier.ca

Drawn By	Design By	Project
B.D.	B.D.	1000-4031 & 721-3464
Check By	Check By	Scale / Drawing
J.P.	B.H.	FIGURE 4



OPTION 1: DIVERT FLOW TO WATERCOURSE 8
INCREASE CAPACITY OF WC 8 OUTLET

CROZIER & ASSOCIATES Consulting Engineers
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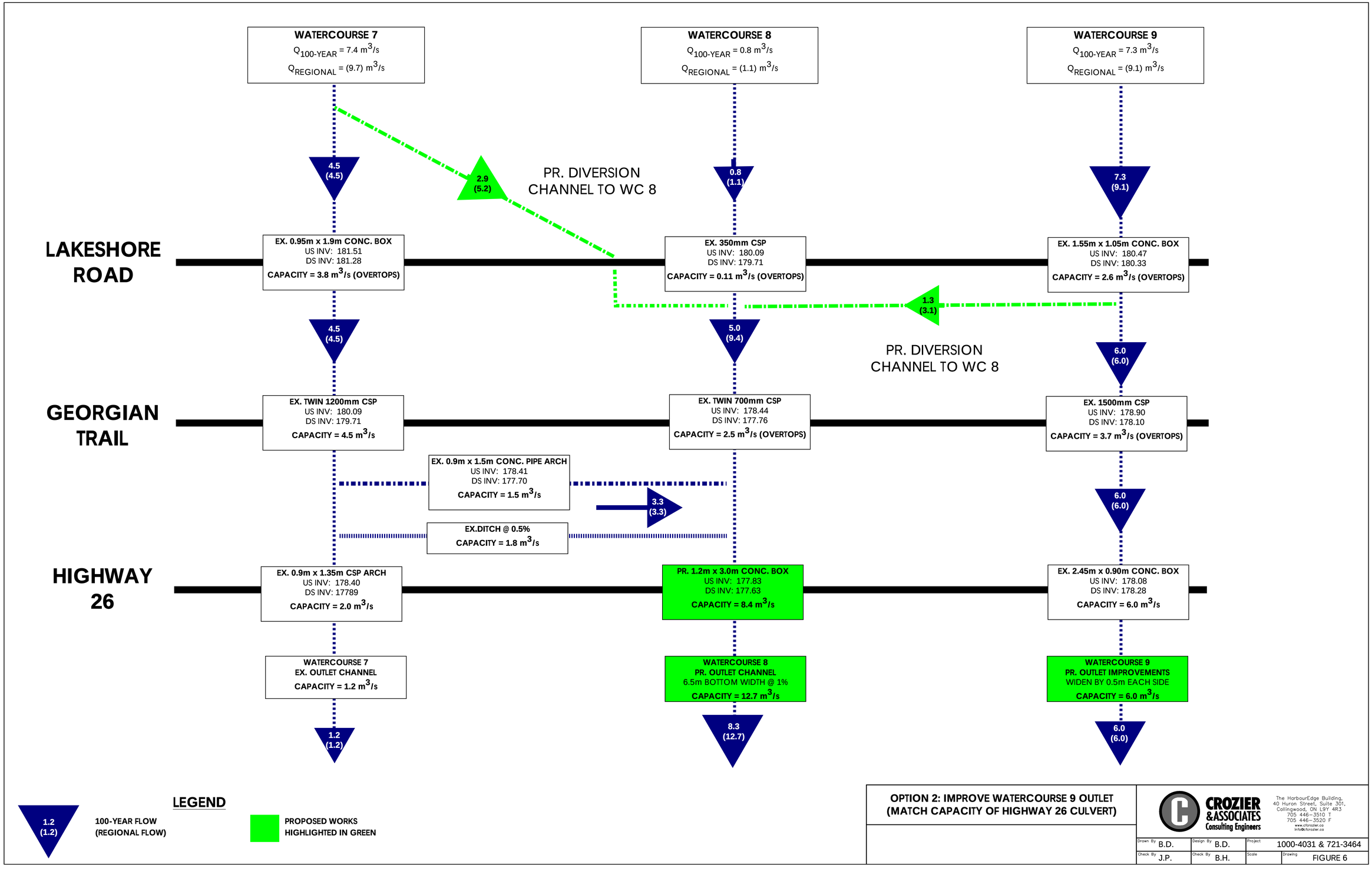
Check By	Check By	Scale / Drawing
J.P.	B.H.	FIGURE 5



OPTION 2: IMPROVE WATERCOURSE 9 OUTLET (MATCH CAPACITY OF HIGHWAY 26 CULVERT)

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Check By	Check By	Scale / Drawing
J.P.	B.H.	FIGURE 6

WATERCOURSE 7
$Q_{100\text{-YEAR}}$ = 7.4 m^3/s
$Q_{REGIONAL}$ = (9.7) m^3/s

WATERCOURSE 8
$Q_{100\text{-YEAR}}$ = 0.8 m^3/s
$Q_{REGIONAL}$ = (1.1) m^3/s

WATERCOURSE 9
$Q_{100\text{-YEAR}}$ = 7.3 m^3/s
$Q_{REGIONAL}$ = (9.1) m^3/s

4.5 (4.5)

2.9 (5.2)

PR. DIVERSION CHANNEL TO WC 8

0.8 (1.1)

7.3 (9.1)

LAKESHORE ROAD

EX. 0.95m x 1.9m CONC. BOX
US INV: 181.51
DS INV: 181.28
CAPACITY = 3.8 m^3/s (OVERTOPS)

EX. 350mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 0.11 m^3/s (OVERTOPS)

EX. 1.55m x 1.05m CONC. BOX
US INV: 180.47
DS INV: 180.33
CAPACITY = 2.6 m^3/s (OVERTOPS)

4.5 (4.5)

3.7 (6.0)

7.3 (9.1)

GEORGIAN TRAIL

EX. TWIN 1200mm CSP
US INV: 180.09
DS INV: 179.71
CAPACITY = 4.5 m^3/s

EX. TWIN 700mm CSP
US INV: 178.44
DS INV: 177.76
CAPACITY = 2.5 m^3/s (OVERTOPS)

EX. 1500mm CSP
US INV: 178.90
DS INV: 178.10
CAPACITY = 3.7 m^3/s (OVERTOPS)

EX. 0.9m x 1.5m CONC. PIPE ARCH
US INV: 178.41
DS INV: 177.70
CAPACITY = 1.5 m^3/s

3.3 (3.3)

EX.DITCH @ 0.5%
CAPACITY = 1.8 m^3/s

7.3 (9.1)

HIGHWAY 26

EX. 0.9m x 1.35m CSP ARCH
US INV: 178.40
DS INV: 17789
CAPACITY = 2.0 m^3/s

PR. 0.9m x 3.0m CONC. BOX
US INV: 177.83
DS INV: 177.63
CAPACITY = 7.5 m^3/s

EX. 2.45m x 0.90m CONC. BOX & PR. 900mm CSP
US INV: 178.08
DS INV: 178.28
COMBINED CAPACITY = 7.5 m^3/s

WATERCOURSE 7
EX. OUTLET CHANNEL
CAPACITY = 1.2 m^3/s

WATERCOURSE 8
PR. OUTLET CHANNEL
4.5m BOTTOM WIDTH @ 1%
CAPACITY = 9.5 m^3/s

WATERCOURSE 9
PR. OUTLET IMPROVEMENTS
WIDEN BY 1.0m EACH SIDE
CAPACITY = 9.1 m^3/s

1.2 (1.2)

7.0 (9.3)

7.3 (9.1)

LEGEND

1.2 (1.2) — 100-YEAR FLOW (REGIONAL FLOW)

PROPOSED WORKS HIGHLIGHTED IN GREEN

OPTION 3: IMPROVE WATERCOURSE 9 OUTLET (INCREASE TO REGIONAL FLOW CAPACITY)

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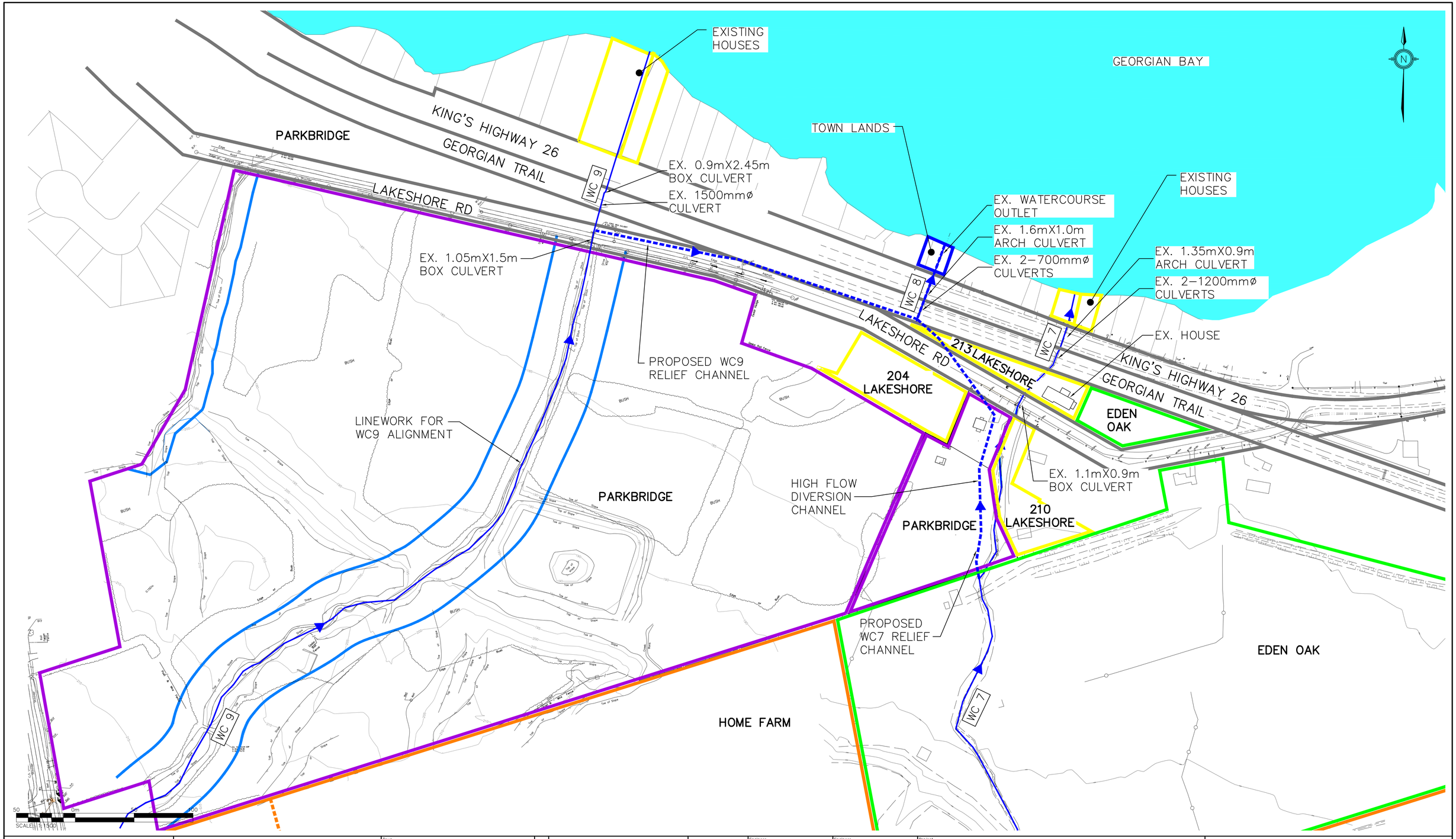
Drawn By	Design By	Project
B.D.	B.D.	1000-4031 & 721-3464
Check By	Check By	Scale / Drawing
J.P.	B.H.	FIGURE 7



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Engineer

PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

FOR DISCUSSION
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Engineer

Project: CRAIGLEITH REGIONAL SWM PLAN
TOWN OF THE BLUE MOUNTAINS

Drawing: CONCEPTUAL WC7 & WC9
FLOOD RELIEF CHANNEL PLAN

CROZIER & ASSOCIATES Consulting Engineers

Drawn By	Design By	Project
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Check By	Check By	Scale / Drawing
B.D.	B.R.C.H.	1:1500 / FIGURE8