FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

ABBOTTS RESIDENTIAL DEVELOPMENT

TAMMY ABBOTTS
TOWN OF THE BLUE MOUNTAINS

PREPARED BY:

C.F. CROZIER & ASSOCIATES INC.
40 HURON STREET, SUITE 301
COLLINGWOOD, ONTARIO
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OCTOBER 2019

CFCA FILE NO. 332-4581

CROZIER
CONSULTING ENGINEERS

Revision Number	Date	Comments
Rev.0	October 2019	Issued for Draft Plan Approval

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1.0 Introduction

C.F. Crozier and Associates (Crozier) has been retained by Tammy Abbotts to provide engineering support for the Draft Plan Application for the proposed Abbotts Residential Development. The Site is legally described as All of Lots 35, 36, 37, 38 and 39 Southwest Side of Bay Street West and covers an area of approximately 1.07 ha. Refer to the Site Location on Figure 1 for the location of the proposed development.

External documents and plans were reviewed in the course of completing this engineering report. As such, the servicing and design considerations contained herein are assisted by the following:

- Geotechnical Report by Peto MacCallum Ltd – October 2019;
- Municipal Lands Topographic Survey by Van Harten – August 2019;
- Site Survey by Van Harten – March 2018; and
- As Constructed Drawings provided by the Town of The Blue Mountains – 1976 (Bay Street West), 1976 (Lansdowne Street North), 1976 (Huron Street West), 1978 (Lansdowne Street North), and 1978 (Lakeshore Drive).

2.0 Site Description & Development Concept

The Site is bounded by residential dwellings to the north and south, Lansdowne Street North to the west and a condominium community to the east. An unopened 20 m Municipal road allowance is located along the north (Bay Street West) and east (Victoria Street) property lines. The majority of the Site has been cleared with the exception being the unopened Municipal road allowances.

The proposed development consists of a residential subdivision including 22 semi-detached units fronting onto a public roadway. Refer to Draft Plan prepared by Van Harten, dated September 5, 2019 in Appendix A.

The proposed internal roadway will be designed with a modified urban cross section, similar to the existing Bay Street West, located immediately to the west of the Site across Lansdowne Street North. Refer to Figure 2 for the proposed road cross-section. The roadway will be located within the existing Municipal right-of-way (ROW), and is to be super-elevated on the south and west sides to allow stormwater runoff to flow towards the proposed drainage buffer.

3.0 Water Servicing

Potable water for the proposed development will be supplied by the Town of The Blue Mountains water distribution system. According to the 2017 Year End Water and Wastewater Capacity Assessment the Town's water system has a total capacity of approximately 15,699 units (16,390 m^3/day), which included the water supply available from the Town of Collingwood (1,250m^3/day). At the end of 2017, 9,383 units were allocated and 3,064 units were reserved, which leaves 3,252 units available. Per the Technical Memo #6 prepared by C3 Water (August 2016) the existing Municipal water system will have supply and treatment capacity until 2029, which is beyond the expected construction date for the proposed development.

3.1 Existing Water Servicing

A review of the As-Constructed drawings of the surrounding water distribution infrastructure indicates that the following are municipal water lines located within public roadways:

- 400 mm diameter trunk watermain on Lansdowne Street North;
- 400 mm diameter trunk watermain on Huron Street West; and
- 150 mm diameter trunk watermain on Lakeshore Drive.

3.2 Design Water Demand

To estimate the proposed water demands for future development of the Site, the MOECC Design Guidelines for Drinking-Water Systems (2008) and Town of The Blue Mountains Engineering Standards (2009) were consulted to determine the average, maximum day and peak hour water demands.

Water demands for the residential development were determined using the following design figures:

- Average Residential Flow Rate 450 L/cap/day
- Max Day/Peak Hour Factors 2.0/4.5

It is estimated that the maximum water demands for the proposed development are as follows:

- Average Day 0.26 L/sec
- Max Day 0.53 L/sec
- Peak Hour 1.19 L/sec

Fire flows required to service the site were calculated to be 116.70 L/s per the Fire Underwriter's Survey and 45 L/s per the Ontario Building Code. The preliminary design flow (peak hour + fire flow) for the Subject Site is 117.89 L/s, subject to detailed design.

Refer to Appendix B for detailed water demand calculations.

3.3 Proposed Water Servicing

The proposed watermain for the development is to be municipally owned and operated. Watermain will be installed in the roadway per the Town of The Blue Mountain Standards. Connections for the Site are proposed at the existing 400 mm diameter trunk watermains along Lansdowne Street North and Huron Street West. Based on the expected water demand and similar developments in the area, the proposed internal watermain is expected to be a 150 mm diameter pipe with individual lot services of appropriate size. This information is being provided to the Town to incorporate into the Town-wide water model to confirm sizing and available pressures. Internal watermain sizing may be subject to change during detailed design. Refer to Figure 3 for the Preliminary Servicing Plan.

Fire flow protection will be provided by way of an existing fire hydrant along the frontage of Lansdowne Street North and proposed fire hydrants within the Site. It is noted that a hydrant flow test has not been completed to verify existing pressures and flow relationships; however, it is expected that adequate fire flows will be available.

4.0 Sanitary Servicing

Sanitary servicing for the development will be achieved via connection to the Town of The Blue Mountains sanitary sewer system.

The Environmental Services Department of the Town of The Blue Mountains produces annual reports on the operation of the Town water and waste water system. We reviewed the most recent report entitled "2017 Annual Performance Report" to assess the Treatment Plant's available capacity to support the Aquavil development.

The Thornbury Waste Water Treatment Plant (WWTP) provides an average daily capacity of 3,580 m^3/day and peak design capacity of 7,196 m^3/day. According to the 2017 Performance Report the WWTP is currently operating at 64% capacity (2,773 m^3/day).

4.1 Existing Sanitary Servicing

Further review of the As-Constructed drawings of the surrounding infrastructure indicates that the following are municipal sanitary sewers located within public roadways:

- 300 mm diameter sanitary trunk sewer on Lansdowne Street North;
- 150 mm diameter sanitary forcemain on Lansdowne Street North;
- 300 mm diameter sanitary sewer on Bay Street West;
- 450 mm diameter sanitary sewer on Huron Street West; and,
- 300 mm diameter sanitary sewer on Lakeshore Drive

Our office completed an investigation of the existing sanitary sewers on April 11, 2019 to determine the accuracy of the record drawings. The elevations of the existing manholes were surveyed, and the depth of the inverts were measured, which generally conformed with the as recorded drawings provided by the Town.

4.2 Design Sanitary Flow

To estimate the proposed sanitary flows produced by the future development of the Site, the MOECC Design Guidelines for Sewage Works (2008) and the Town of The Blue Mountains Engineering Design Standards (2009) were consulted to determine the Average Daily Domestic Flow.

Sewage demands for the Site were determined using the following design figures:

- Average Residential Flow Rate 450 L/cap/day
- Infiltration 0.23 L/s/ha
- Persons Per Residential Unit 2.3

The maximum expected sanitary sewer flows for Site has been calculated to be 1.53 L/s. Refer to Appendix C for the detailed sanitary flow calculations. Due to the relatively low flows from the development it is assumed that there will be sufficient capacity in the existing municipal sanitary system, which will need to be confirmed during detailed design.

4.3 Proposed Sanitary Servicing

The Site will be serviced via a gravity sewer connected to the existing manhole (SAN MH33) and 300 mm diameter sanitary sewer along Lansdowne Street North. The internal sanitary sewer will follow the internal roadway network with individual connections to each building. The proposed sanitary sewer will be sufficiently deep to drain units with basements. Due to the relatively low peak sanitary flows calculated in the previous section it is reasonable to assume that a 200 mm diameter internal sanitary sewer will provide adequate capacity to convey the wastewater to the Municipal system. The proposed sanitary system is to be owned and maintained by the Municipality. Refer to Figure 3 for the Preliminary Servicing Plan.

Sanitary maintenance hole(s) will be installed with spacing consistent with Town standards. The proposed 200 mm diameter internal sanitary sewer will be designed with sufficient slope to provide cleaning velocity within the sewer to reduce maintenance issues post construction.

5.0 Utilities

The proposed development will be serviced with natural gas, telephone, cable TV and hydro. All such utilities are currently available on the boundary roadways. Utilities have not been contacted at the time of this investigation. Circulation and coordination with the utilities will be undertaken to confirm capacity at the appropriate phase of design.

6.0 Stormwater Management

The management of stormwater and site drainage for the proposed development must comply with the policies and standards of the various agencies including the Town of The Blue Mountains, Grey Sauble Conservation Authority (GSCA), and Ministry of the Environment, Conservation and Parks (MECP).

The stormwater management (SWM) criteria that will be met with the proposed development are as follows:

- Water Quality Control
  - o "Enhanced Protection" given that Georgian Bay is the receiver.

- Water Quantity Control
  - o The proposed SWM design must control post development flows to the available capacity of the Lansdowne Street ditch.

- Erosion Control
  - o Due to the proximity of the development to Georgian Bay erosion control is not required.

- Development Standard
  - o Modified urban cross section;
  - o Lot grading at 2% optimum; and
  - o Drainage buffer to convey frequent and infrequent rainfall/runoff events.

6.1 Existing Drainage Conditions

6.1.1 Internal Drainage

On-site soils are classified as Kemble silty clay (Ksc) (Soil Survey of Grey County, 1979). These soils are considered as imperfectly drained. Van Harten completed a topographic survey for the Site, dated March 2018, which showed that the existing grading of the property is relatively flat but naturally slopes south to north towards Georgian Bay. Stormwater runoff from the Site is assumed to flow via sheet flow towards the existing ditches located within the unopened road allowances of Victoria Street and Bay Street West. An existing 450 mm diameter CSP culvert, which crosses the road at the Lansdowne and Bay Street West intersection, conveys the runoff from the Site to the Lansdowne Street ditch. The existing ditch is directly connected to Georgian Bay, which is the ultimate receiver. Refer to Figure 4 for the Pre-Development Drainage Areas.

Our office has reviewed the Draft Geotechnical Investigation Report, dated September 26, 2019 by Peto MacCallum Ltd. (PML), which will be submitted under separate cover. It is understood that the Site will require topsoil to be stripped to a depth of approximately 0.7 to 1.4 m. PML identified the presence of a perched ground water table 2.0 to 3.0 m below existing grade on September 10th, 2019. PML will be conducting year-long water table monitoring program to confirm the seasonal high ground water table. It recommended that the Site will need to provide a minimum of 0.5 m of clearance of between the basement slab and the seasonal high groundwater table.

6.1.2 External Drainage

Based on the available topographic survey data and As-Constructed drawings received from the Town there are external flows being conveyed through the Site. The external sources of runoff include the residential lots to the south, the condominium lands to the east (Bayside Villas) and a 400 mm diameter culvert crossing Huron Street West within the Victoria Street Municipal road allowance. Refer to Figure 4 for the pre-development drainage areas. The run-off from these external lands, as well as those from the Site, will be conveyed to a suitable outlet post development. The external catchments that drain through the Site are described below:

External Catchment #1: Huron Street West

Based on site visits completed by Crozier the existing lots fronting onto Huron Street West are graded with split drainage and a portion of runoff is directed into the Site by way of sheet flow. To be conservative we have assumed the catchment includes approximately half of each lot backing onto the Site, which has been estimated as 0.5 ha.

External Catchment #2: Bayside Villas

Similar to the lots on Huron Street West, the rear yards of Bayside Villas contribute runoff to the Site. The topographic survey completed by Van Harten (2018) shows a 450 mm diameter CSP culvert that discharges into the existing ditch within the Victoria Street North Municipal road allowance. The size of the catchment has been conservatively estimated to be approximately 0.19 ha.

External Catchment #3: Victoria Street North Road Allowance

The Victoria Street North Municipal road allowance is used as a servicing corridor and overland flow route. An existing 400mm diameter CSP culvert crosses Huron Street West and discharges into the existing ditch within the proposed development. Based on site visits completed by Crozier, there does not appear to be any upstream culverts or ditches that discharge into the road allowance further upstream. The intersection of King Street West and Victoria Street South appears to be a high point with ditches conveying flows east and west along the King Street West right-of-way (ROW). The approximate size of the catchment that contributes runoff to the Site was estimated as 0.59 ha.

Immediately downstream of the Site is the Lansdowne Street ditch, which conveys flows from the Site and other external catchments to Georgian Bay. There are currently four (4) CSP culverts that discharge into the ditch, which were surveyed by Van Harten in August 2019. The external catchments that contribute to the Lansdowne Street North ditch are described below:

External Catchment #4: Lansdowne Street North

An existing 400 mm diameter CSP culvert crosses Bay Street West and discharges into the Lansdowne Street North ditch. The size of the catchment that contributes runoff to this culvert has been estimated as approximately 0.6 ha, based on site visits completed by Crozier. Drainage from the lots fronting onto Lansdowne Street North and the south side of Bay Street West flow towards the existing ditches that convey stormwater to the culvert. It was observed that Lansdowne Street North has an existing crest in the roadway halfway between Huron Street West and Bay Street West, which directs stormwater north and south of the high point.

External Catchment #5: Bay Street West

Our office has reviewed the As-Built drawings for Bay Street West, which features a modified urban cross-section with curb and gutter on the north side of the road. The roadway has a crest approximately 100 m west of Lansdowne Street North, which directs runoff east and west to an existing catchbasin manhole and Little Beaver Creek respectively. The existing catchbasin outlets to the Lansdowne Street North ditch via a 300 mm diameter CSP culvert. The approximate size of the catchment that drains to the catchbasin is 0.28 ha.

External Catchment #6: Lakeshore Drive

An existing 400 mm diameter CSP culvert crosses Lansdowne Street North downstream of the Site and discharges into the existing ditch. However, it appears that the ditch on the east side of Lansdowne Street North does not drain towards the culvert so the flow being conveyed by the culvert is negligible.

The roadway cross-section for Lakeshore Drive does not include road side ditches or curb and gutter to convey stormwater, so it can be reasonably assumed that a majority of the drainage flows via sheet flow to Georgian Bay. However, upon review of the As-Built drawings for Lakeshore Drive provided by the Town, it appears that the slope of the road generally conveys stormwater towards Lansdowne Street North. Therefore, we have accounted for a catchment size of approximately 0.4 ha contributing to the Lansdowne Street North ditch.

To determine the flows for the external catchments the Rational Method was used for the 2-year to 100-year storm events. Table 1 below summarises the external flow rates upstream and downstream of the Site:

Table 1: External Catchment Peak Flow Calculations

	Pre-Development Uncontrolled Peak Flow Rate (m^3/s)			
	5 yr	**10 yr**	**25 yr**	**100 yr**
Ex. Catchment #1	0.067	0.078	0.092	0.113
Ex. Catchment #2	0.025	0.030	0.035	0.043
Ex. Catchment #3	0.106	0.123	0.145	0.177
Ex. Catchment #4	0.089	0.104	0.123	0.150
Ex. Catchment #5	0.042	0.049	0.057	0.070
Ex. Catchment #6	0.054	0.063	0.074	0.090

6.2 Proposed Drainage Conditions

6.2.1 Internal Drainage

The grading for each lot will consist of rear to front drainage. Side-yard swales will be graded at 2% (min.) to direct runoff from the lots towards the internal roadway. The internal roadway will be super elevated on the south and west side to allow water to flow towards the 6-metre drainage buffer. Refer to Figure 5 for the Post-Development Drainage Plan.

To determine the pre and post development flows for the Site the Modified Rational Method was used to quantify the capacity required to safely convey the 2-year to 100-year storm events. Refer to Appendix D for the Modified Ration Method calculations. Per the Town of The Blue Mountains Engineering Standards, the Intensity Duration Frequency Curves for the Owen Sound area were used in the calculations. The pre and post-development flows are shown below in Table 2.

Table 2: Modified Rational Method Peak Flow Calculations

	Pre Development				Post Development			
	5 yr	**10 yr**	**25 yr**	**100 yr**	**5 yr**	**10 yr**	**25 yr**	**100 yr**
Uncontrolled Peak Flow (m^3/s)	0.198	0.231	0.272	0.333	0.295	0.344	0.406	0.497

Since External Catchments 1, 2 and 3 drain through the site, the total flow rate that must be conveyed by the proposed drainage buffer is the sum of the following flows resulting in a total conveyed flow rate of 0.830 m^3/s.

- Ex. Catchment #1 (100-year) 0.113 m^3/s
- Ex. Catchment #2 (100-year) 0.043 m^3/s
- Ex. Catchment #3 (100-year) 0.177 m^3/s
- Proposed Development (100-year) 0.497 m^3/s

A Flowmaster model was prepared to demonstrate the capacity of the drainage buffer. A typical cross-section for the 6 m wide ditch and a more narrow 4.4 m wide section at the north-east corner of the proposed development were modelled using the estimated 100-year flow rate.

The drainage buffer will drain towards a proposed culvert crossing and into the existing Lansdowne Street North ditch. A 600 mm diameter culvert will be required, per Town standards for a ditch to ditch crossing. A 600 mm diameter CSP culvert, at the same 3.0% slope as the existing 400 mm diameter culvert, has sufficient capacity to convey, up to and including, the 25-year post development flow rate. Refer to Appendix E for the Flowmaster Design Sheets and supporting calculations. Flows that exceed the 25-year storm event will flow overland into the existing ditch on the west side of Lansdowne Street North.

6.2.2 External Drainage

The proposed outlet for the stormwater that is conveyed through the proposed development, which includes the external flows upstream of the Site, is the existing ditch on the west side of Lansdowne Street North. The ditch was surveyed by Van Harten (August 2019) in order to model the existing cross-sections in Flowmaster with the increased flow rate post development. Two of the most narrow cross-sections were modelled using the flow rates calculated for the internal and external catchments that drain towards the existing ditch. The total anticipated flow rate, for the post development 100-year storm event is 1.14 m^3/s. Refer to Appendix E for the Flowmaster Design Sheets and supporting calculations.

Based on the Flowmaster model, the existing ditch has sufficient capacity to convey the increased run-off produced by the proposed development.

6.3 Stormwater Quantity Control

Due to the proximity to Georgian Bay it is not proposed to provide quantity control for the proposed development. Based on the preliminary modelling, the existing ditch on the west side of Lansdowne Street North has capacity available for the increased flow rate produced as a result of the proposed development.

If quantity control is required during the detailed design stage, onsite storage could be achieved through a variety of methods. Table 3 below presents typical forms of quantity control and their suitability for the Site.

Table 3: SWM Facility Options for Abbotts Residential Development

SWM Facility Type	Comments	Recommended (Yes/No)
Wet Pond	Wet ponds are an effective way to control the flows and are not affected by groundwater and bedrock. However, this form of quantity control takes up a significant amount of space and, as per MOE design guidelines, are not recommended for areas less than 5 ha.	**No**
Dry Pond	Similar to wet pond, dry ponds take up large areas and are not recommended for areas less than 5 ha.	**No**
Infiltration Basin	Infiltration basins are acceptable for smaller areas like Abbotts Residential Development. However, infiltration basins take up more space than is available and may be limited by the seasonal high groundwater elevation.	**No**
Surface Storage	Since the developed area will include a drainage buffer there is an opportunity to grade the buffer to include a “sawtooth” design to allow for surface storage.	**Yes**
Super Pipes	Storage within the pipes, manholes and catchbasins located within the internal roadway can be utilised in conjunction with an orifice plate placed on an outlet structure downstream to provide storage internally. Since storm sewer is not anticipated for the proposed development superpipes are likely not a viable alternative.	**No**
Subsurface Storage Tanks	Similar to super pipes, the internal roadway could be used for subsurface storage tanks (Stormtech chambers or equivalent), which can control a larger volume. Although storm sewer is not likely to be included during detailed design of the proposed development, an inlet headwall at the downstream end of the drainage buffer could convey stormwater to a subsurface storage tank.	**Yes**
Low Impact Designs (LIDs)	The individual lots in Abbotts Residential Development provide opportunities to include LIDs such as infiltration trenches and soak-away pits into the grading plans to provide additional quantity control. Due to the relatively small amount of storage these methods provide, they would likely need to be accompanied with another method of quantity control. However, due to the silty clay soil characteristics these methods may not be appropriate for the proposed development.	**No**

6.4 Stormwater Quality Control

It will be necessary to implement stormwater management practices to address the water quality control requirements of the regulatory agencies. Georgian Bay is the ultimate receiver of drainage from the Site and therefore the development will incorporate measures to provide "enhanced protection" to treat runoff before entering the Bay.

Some of the significant factors involved in selecting the optimal SWM approach in Table 3 were the "infill" nature of the development and the relatively small size of the Site, which make it unsuitable for an end-of-pipe stormwater management facility (i.e. wet pond). Therefore, a treatment train approach will be developed, consisting of lot level control and end of pipe control. The proposed treatment train will provide the Enhanced Quality Control required by the review agencies and is listed below:

1. **Lot Level Control**
    - Reduced lot grading; and
    - Grassed swales underlain with permeable material.
2. **End of Pipe Control**
    - Oil/grit separators (Stormceptor or equivalent).

6.4.1 Stormwater Lot Level Controls

These controls will primarily consist of disconnected roof leaders, reduced lot grading and grass swales. Swales will be underlain with select permeable material and perforated tile ("French Drains") where deemed necessary. These swales are intended to promote water quality benefits of vegetation and filtering from a nutrient perspective. The majority of the lot level controls will be implemented within the individual lot grading plans.

In addition to the lot level controls, we will also evaluate the water quality benefits of incorporating an enhanced swale to the treatment train. The enhanced swale would be located within the drainage buffer and provide pre-treatment to the stormwater prior to being conveyed to the end-of-pipe controls.

6.4.2 Stormwater End-Of-Pipe Controls

Although the proposed development will utilize a drainage buffer to convey stormwater from the Site, instead of storm sewer, an end-of-pipe control system may be incorporated during the detailed design stage to provide quality control. An inlet headwall at the downstream end of the drainage buffer could be used to convey the stormwater to an oil/grit separator (OGS).

Oil/grit separators are typically recommended to treat runoff from the roadways, which are the main source of oils and sediment from the vehicles and expected maintenance activities on the site. These structures are typically pre-manufactured and provide effective removal of oils and total suspended solids. The oil/grit separators are sized to treat minimum 95% annual runoff and a minimum 80% of annual total suspended solids (TSS) removal.

7.0 Conclusions and Recommendations

Based on the information offered in this report, we offer the following conclusions:

1. A 20 m ROW is proposed for the public roadways and will consist of a modified urban cross section consisting of curb and gutter along the frontage of the lots;
2. A public watermain will be extended from the Lansdowne Street North and loop through development where it will connect to Huron Street West. Additional watermain modelling and hydrant testing may be required;
3. A public sanitary sewer will be extended from Lansdowne Street North along the internal roadway. The existing sanitary sewer downstream of the Site is sufficiently sized to convey the proposed sewage generated;
4. Internal preliminary grading has been completed to maintain existing elevations of the Site. It has been assumed that all lots will drain from rear to front towards the proposed roadway and ultimately to Georgian Bay. The overall master grading will be completed during detailed design;
5. The proposed internal drainage buffer has capacity to convey internal and external runoff through the Site and a 600 mm diameter culvert will be used to convey stormwater from the Site to the proposed outlet;
6. The existing external ditch on Lansdowne Street North has capacity to convey the increased runoff from the Site through to Georgian Bay;
7. Quantity controls for the Site will not be required due to the proximity to the ultimate receiver, Georgian Bay; and
8. Water quality controls for the Site will be provided by a treatment train consisting of lot level and end of pipe controls.

Based on the above conclusions, we recommend the approval of the Planning Applications for the Abbotts Residential Development, from the perspective of functional servicing and stormwater management.

Respectfully submitted,

C.F. CROZIER & ASSOCIATES INC.

Kevin Morris, P. Eng.
Founding Partner

C.F. CROZIER & ASSOCIATES INC.

George Cooper, E.I.T.
Engineering Intern

KM/gc

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APPENDIX A

Draft Plan (Van Harten, 2019)

SCHEDULE: RE: SECTION 51 - THE PLANNING ACT.

(a) BOUNDARIES AS SHOWN
(b) WIDTHS OF ROADS AS SHOWN
(c) KEY PLAN AS SHOWN
(d) RESIDENTIAL SEMI-DETACHED LOTS
(e) RESIDENTIAL
(f) DIMENSIONS OF UNITS AS SHOWN
(g) NATURAL FEATURES N/A
(h) MUNICIPAL WATER AVAILABLE
(i) GRAVEL AND LOAM
(j) CONTOURS AS SHOWN
(k) ALL MUNICIPAL SERVICES AVAILABLE
(l) NIL

LAND USE SCHEDULE

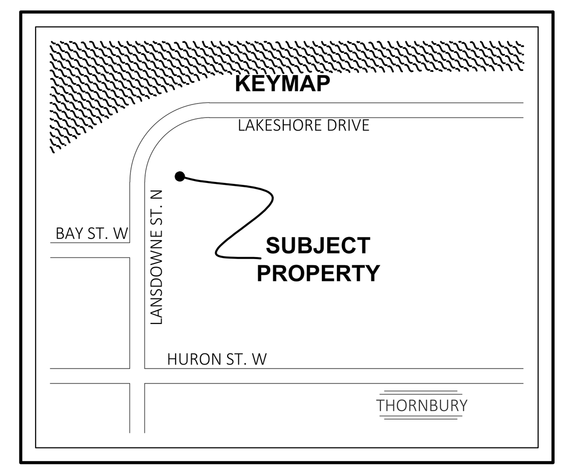
DESCRIPTION	UNITS	LOTS/BLOCKS	AREA (ha.)
Semi-Detached Residential	22	LOTS 1-11	1.01

LEGAL DESCRIPTION

ALL OF LOTS 35, 36, 37, 38, AND 39 SOUTHWEST SIDE OF BAY STREET TOWN OF THORNBURY COUNTY OF GREY

SCALE 1 : 300

VAN HARTEN SURVEYING INC.



OWNER'S CERTIFICATE

TAMMY ABBOTTS AUTHORIZES THE SUBMISSION OF THIS DRAFT PLAN OF SUBDIVISION TO THE TOWN OF THORNBURY PLANNING DEPARTMENT.

TAMMY ABBOTTS DATE

SURVEYOR'S CERTIFICATE

I CERTIFY THAT THE BOUNDARIES OF THE LAND TO BE SUBDIVIDED AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE CORRECTLY SHOWN.

JAMES M. LAWS, O.L.S.
Van Harten Surveying Inc.

SEPTEMBER 5, 2019
DATE

METRIC:

DISTANCES AND COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

Van Harten SURVEYING INC.
LAND SURVEYORS and ENGINEERS

Elmira	Guelph	Orangeville
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DRAWN BY: S. J. CHECKED BY: JML PROJECT No. 25655-18

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APPENDIX B

Water Demand Calculations

File:	332-4581
Date:	July 19, 2019
By:	BDP
Check By:	KM

Abbotts Residential Development - Domestic Water Design Criteria

Developed Site Area	1.73	ha
Number of Residential Units	22	units
Persons Per Unit (Town of The Blue Mountains Engineering Standards)	2.3	persons/unit
Residential Population	51	persons
Water Design Flows		
Residential	450	L/C-day
Total Domestic Water Design Flows		
Average Residential Daily Flow	0.26	L/sec
Max Day Peak Factor (Town of The Blue Mountains Engineering Standards)	2.00	
Max Day Demand Flow	**0.53**	L/sec
Peak Hour Factor (Town of The Blue Mountains Engineering Standards)	4.50	
Peak Hour Flow	**1.19**	L/sec

ABBOTTS RESIDENTIAL DEVELOPMENT

October 4, 2019

Fire Protection Volume Calculation - Semi Detached Units

CFCA File: 332-4581

Water Supply for Public Fire Protection - 1999
Fire Underwriters Survey

Part II - Guide for Determination of Required Fire Flow

1. An estimate of fire flow required for a given area may be determined by the formula:

F = 220 * C * sqrt A

where
F = the required fire flow in litres per minute
C = coefficient related to the type of construction
= 1.5 for wood frame construction (structure essentially all combustible)
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
= 0.8 for non-combustible construction (unprotected metal structural components)
= 0.6 for fire-resistive construction (fully protected frame, floors, roof)
A = The total floor area in square metres (including all storeys, but excluding basements at least 50 percent below grade) in the building considered.

Proposed Buildings	Fire resistive construction
270 sq.m. total floor area	1.0 C

*Total floor area was obtained assuming 30% lot coverage for semi-detached homes (2 units, 50m deep and 9m wide)

Therefore F= 4,000 L/min (rounded to nearest 1000 L/min)

Fire flow determined above shall not exceed:
30,000 L/min for wood frame construction
30,000 L/min for ordinary construction
25,000 L/min for non-combustible construction
25,000 L/min for fire-resistive construction

2. Values obtained in No. 1 may be reduced by as much as 25% for occupancies having low contents fire hazard or may be increased by up to 25% surcharge for occupancies having a high fire hazard.

Non-Combustible	-25%	Free Burning	15%
Limited Combustible	-15%	Rapid Buring	25%
Combustible	No Charge		

Low fire Hazard occupancy for dwellings	0% reduction

0 L/min reduction

Note: Flow determined shall not be less than 2,000 L/min

3. Sprinklers - The value obtained in No. 2 above maybe reduce by up to 50% for complete automatic sprinkler protection.

Buildings will have automatic sprinklers (typical 30% reduction)
0 L/min reduction

Water Supply for Public Fire Protection - 1999
Fire Underwriters Survey

Part II - Guide for Determination of Required Fire Flow

4. Exposure - To the value obtained in No. 2, a percentage should be added for structures exposed within 45 metres by the fire area under consideration. The percentage shall depend upon the height, area, and construction of the building(s) being exposed, the separation, openings in the exposed building(s), the length and height of exposure, the provision of automatic sprinklers and/or outside sprinklers in the building(s) esposed, the occupancy of the exposed building(s) and the effect of hillside locations on the possible spread of fire.

Separation	Charge	Separation	Charge
0 to 3 m	25%	20.1 to 30 m	10%
3.1 to 10 m	20%	30.1 to 45 m	5%
10.1 to 20 m	15%		

Exposed buildings

Name		**Distance**		
North	Adjacent Dwelling	35	5%	200
East	Adjacent Dwelling	2.4	25%	1000
South	Adjacent Dwelling	15	15%	600
West	Adjacent Dwelling	2.4	25%	1000
				2,800 L/min Surcharge

Determine Required Fire Flow

No.1	4,000		
No. 2	0 reduction		
No. 3	0 reduction		
No. 4	2,800 surcharge		
Required Flow:	**6,800 L/min**		
Rounded to nearest 1000l/min:	**7,000 L/min**	or	116.7 L/s
			1,849 USGPM

Determine Required Fire Storage Volume

Flow from above 7,000 L/min

Required duration 2.00 hours

Therefore: 840,000 Litres or
840 cu.m. is the required fire storage volume.

Required Duration of Fire Flow	
Flow Required L/min	Duration (hours)
2,000 or less	1.0
3,000	1.25
4,000	1.5
5,000	1.75
6,000	2.0
8,000	2.0
10,000	2.0
12,000	2.5
14,000	3.0
16,000	3.5
18,000	4.0
20,000	4.5
22,000	5.0
24,000	5.5
26,000	6.0
28,000	6.5
30,000	7.0
32,000	7.5
34,000	8.0
36,000	8.5
38,000	9.0
40,000 and over	9.5

Fire Protection Volume Calculation - Semi Detached Units
CFCA File: 332-4581

Fire Protection Water Supply Guideline
Part 3 of the Ontario Building Code (2006)

$$Q = KVS_{TOT}$$

Q = minimum supply of water in litres (L)
K = water supply coefficient
V = total building volume in cubic metres
S_{TOT} = total of spatial coefficient values from property line exposures on all sides

K =	23.0	Group C building with combustible construction (Table 1)
V =	1485	Total building volume in cubic metres
S_{TOT} =	2	S_{TOT} Need Not Exceed 2.0

Q = 68310 L

Based on ranges listed in Table 2, the required minimum water supply flow rate is **2700 L/min**

45 L/s

APPENDIX C

Sanitary Flow Calculations

File:	332-4581
Date:	July 19, 2019
By:	BDP
Check By:	KM

Abbotts Residential Development - Sanitary Design Criteria

Developed Site Area (Roads + Residences)	1.73	ha
Number of Residential Units	22	units
Person Per Residential Unit	2.30	persons/unit
Residential Population	51	persons

Unit Sewage flows

Residential	450	L/C-day
Infiltration (typical)	0.23	L/s/ha

Total Design Sewage Flows

Infiltration/Inflow Residential		0.40	L/sec
Average Daily Residential Flow		0.26	L/sec
Residential Peak Factor	(Harmon Formula)	4.3	
Total Peak Daily Flow		**1.53**	**L/sec**

APPENDIX D

Modified Rational Method

Owen Sound IDF Curve Parameters		
Storm Event	**A**	**B**
2	22.3	-0.714
5	29.1	-0.724
10	33.6	-0.729
25	39.3	-0.734
50	43.5	-0.736
100	47.7	-0.738

Total Site Area = 1.66 ha

Internal Catchment #1

Surface	Pre-development		Post-development	
	Area (ha)	Runoff Coefficient	Area (ha)	Runoff Coefficient
Landscapes	1.66	0.40	1.01	0.40
Asphalt	0.00	0.90	0.44	0.90
Building	0.00	0.90	0.21	0.90
Total *	1.66	0.40	1.66	0.60

CROZIER
CONSULTING ENGINEERS

PROJECT: Abbotts Residential Development
PROJECT No.: 332-4581
FILE: Modified Rational Method
DATE: 2019.09.25
DESIGN: GC
CHECK:

Modified Rational Method Storage Sizing (2-Year Storm)

Peak Flow

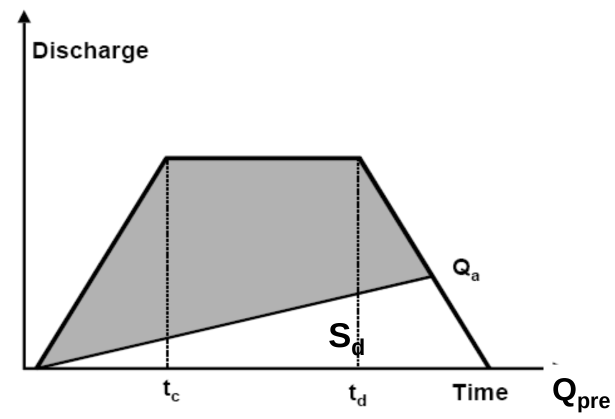
$Q_{post} = 0.0028 \cdot C_{post} \cdot i_{(T_d)} \cdot A$

Intensity

$i_{(T_d)} = A\,(T_d)^{B}$

Storage

$S_d = Q_{post} \cdot T_d - Q_{pre}\,(T_d + T_c) / 2$



Pre-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	80.15
Return Period	2 yr		
Time of Concentration (min)	10		
Coeff A	22.3		
Coeff B	-0.714		
Runoff Coeff (Unadjusted)	0.40	Flow (m^3/s)	0.149
Area (ha)	1.66		

Post-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	80.15
Return Period	2 yr		
Time of Concentration (min)	10		
Coeff A	22.3		
Coeff B	-0.714		
Runoff Coeff (unadjusted)	0.60	Uncont. Flow (m^3/s)	0.222
Area (ha)	1.66		

Target Flow (m^3/s)	0.149

REQUIRED STORAGE VOLUME:	**44.0**

Storage Volume Determination (Detailed)				
T_d	i	T_d	Q_{Uncont}	S_d
min	*mm/hr*	*sec*	*m^3/s*	*m^3*
10	80.15	600	0.222	44.0
15	60.00	900	0.166	38.1
20	48.86	1200	0.136	28.6
25	41.67	1500	0.116	17.0
30	36.58	1800	0.102	3.9
35	32.77	2100	0.091	-10.2
40	29.79	2400	0.083	-25.2
45	27.38	2700	0.076	-40.7
50	25.40	3000	0.070	-56.8
55	23.73	3300	0.066	-73.3
60	22.30	3600	0.062	-90.2
65	21.06	3900	0.058	-107.4
70	19.98	4200	0.055	-124.8
75	19.02	4500	0.053	-142.5
80	18.16	4800	0.050	-160.5
85	17.39	5100	0.048	-178.6

CROZIER CONSULTING ENGINEERS

PROJECT: Abbotts Residential Development
PROJECT No.: 332-4581
FILE: Modified Rational Method
DATE: 2019.09.25
DESIGN: GC
CHECK:

Modified Rational Method Storage Sizing (5-Year Storm)

Peak Flow

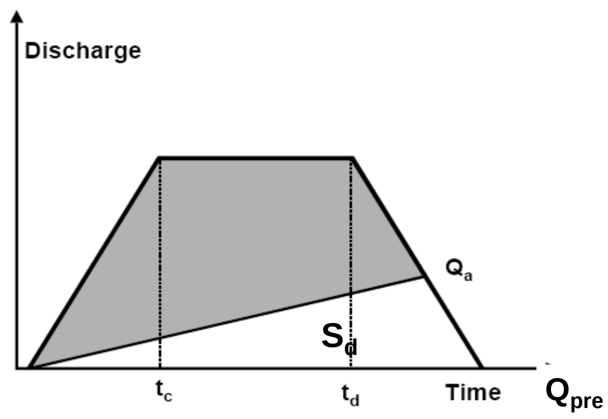
$$Q_{post} = 0.0028 \cdot C_{post} \cdot i_{(T_d)} \cdot A$$

Intensity

$$i_{(T_d)} = A\,(T_d)^{B}$$

Storage

$$S_d = Q_{post} \cdot T_d - Q_{pre}\,(T_d + T_c) / 2$$



Pre-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	106.48
Return Period	5 yr		
Time of Concentration (min)	10		
Coeff A	29.1		
Coeff B	-0.724		
Runoff Coeff (Unadjusted)	0.40	Flow (m^3/s)	0.198
Area (ha)	1.66		

Post-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	106.48
Return Period	5 yr		
Time of Concentration (min)	10		
Coeff A	29.1		
Coeff B	-0.724		
Runoff Coeff (unadjusted)	0.60	Uncont. Flow (m^3/s)	0.295
Area (ha)	1.66		

Target Flow (m^3/s)	0.198

REQUIRED STORAGE VOLUME:	**58.5**

Storage Volume Determination (Detailed)

T_d	i	T_d	Q_{Uncont}	S_d
min	*mm/hr*	*sec*	*m^3/s*	*m^3*
10	106.48	600	0.295	58.5
15	79.39	900	0.220	49.8
20	64.47	1200	0.179	36.5
25	54.85	1500	0.152	20.4
30	48.07	1800	0.133	2.5
35	42.99	2100	0.119	-16.8
40	39.03	2400	0.108	-37.0
45	35.84	2700	0.099	-58.2
50	33.21	3000	0.092	-79.9
55	30.99	3300	0.086	-102.3
60	29.10	3600	0.081	-125.1
65	27.46	3900	0.076	-148.3
70	26.03	4200	0.072	-171.8
75	24.76	4500	0.069	-195.7
80	23.63	4800	0.066	-219.8
85	22.61	5100	0.063	-244.2

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PROJECT: Abbotts Residential Development
PROJECT No.: 332-4581
FILE: Modified Rational Method
DATE: 2019.09.25
DESIGN: GC
CHECK:

Modified Rational Method Storage Sizing (10-Year Storm)

Peak Flow

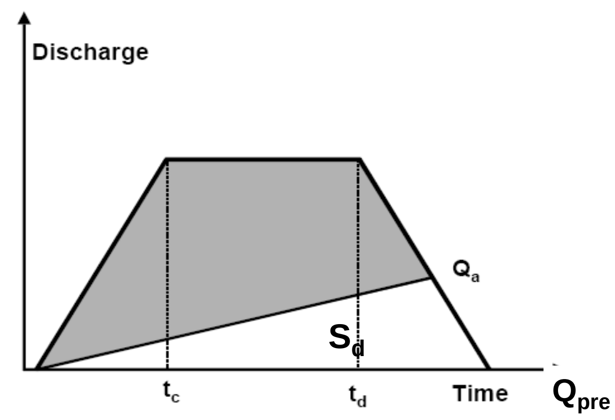
$Q_{post} = 0.0028 \cdot C_{post} \cdot i_{(T_d)} \cdot A$

Intensity

$i_{(T_d)} = A\,(T_d)^{B}$

Storage

$S_d = Q_{post} \cdot T_d - Q_{pre}\,(T_d + T_c) / 2$



Pre-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	124.05
Return Period	10 yr		
Time of Concentration (min)	10		
Coeff A	33.6		
Coeff B	-0.729		
Runoff Coeff (Unadjusted)	0.40	Flow (m^3/s)	0.231
Area (ha)	1.66		

Post-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	124.05
Return Period	10 yr		
Time of Concentration (min)	10		
Coeff A	33.6		
Coeff B	-0.729		
Runoff Coeff (unadjusted)	0.60	Uncont. Flow (m^3/s)	0.344
Area (ha)	1.66		

Target Flow (m^3/s)	0.231

REQUIRED STORAGE VOLUME:	**68.2**

Storage Volume Determination (Detailed)

T_d	i	T_d	Q_{Uncont}	S_d
min	*mm/hr*	*sec*	*m^3/s*	*m^3*
10	124.05	600	0.344	68.2
15	92.31	900	0.256	57.5
20	74.84	1200	0.208	41.6
25	63.61	1500	0.177	22.6
30	55.69	1800	0.155	1.4
35	49.77	2100	0.138	-21.3
40	45.16	2400	0.125	-45.2
45	41.44	2700	0.115	-70.1
50	38.38	3000	0.106	-95.7
55	35.80	3300	0.099	-121.9
60	33.60	3600	0.093	-148.7
65	31.70	3900	0.088	-175.9
70	30.03	4200	0.083	-203.6
75	28.56	4500	0.079	-231.6
80	27.24	4800	0.076	-259.9
85	26.07	5100	0.072	-288.5

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CONSULTING ENGINEERS

PROJECT: Abbotts Residential Development
PROJECT No.: 332-4581
FILE: Modified Rational Method
DATE: 2019.09.25
DESIGN: GC
CHECK:

Modified Rational Method Storage Sizing (25-Year Storm)

Peak Flow

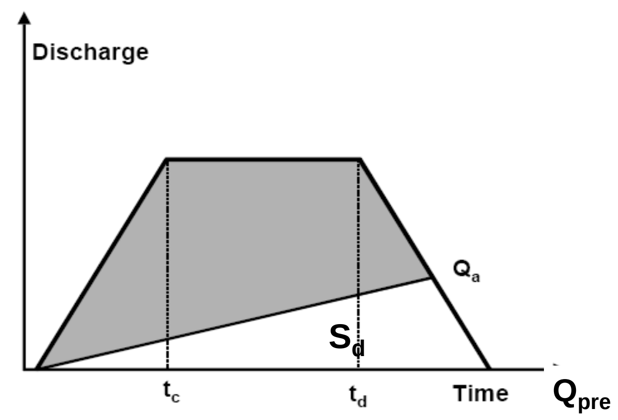
$Q_{post} = 0.0028 \bullet C_{post} \bullet i_{(T_d)} \bullet A$

Intensity

$i_{(T_d)} = A\ (T_d)^{B}$

Storage

$S_d = Q_{post} \bullet T_d - Q_{pre}\ (T_d + T_c)\ /\ 2$



Pre-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	146.40
Return Period	25 yr		
Time of Concentration (min)	10		
Coeff A	39.3		
Coeff B	-0.734		
Runoff Coeff (Unadjusted)	0.40	Flow (m^3/s)	0.272
Area (ha)	1.66		

Post-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	146.40
Return Period	25 yr		
Time of Concentration (min)	10		
Coeff A	39.3		
Coeff B	-0.734		
Runoff Coeff (unadjusted)	0.60	Uncont. Flow (m^3/s)	0.406
Area (ha)	1.66		

Target Flow (m^3/s)	0.272

REQUIRED STORAGE VOLUME:	**80.4**

Storage Volume Determination (Detailed)				
T_d	i	T_d	Q_{Uncont}	S_d
min	*mm/hr*	*sec*	*m^3/s*	*m^3*
10	146.40	600	0.406	80.4
15	108.72	900	0.302	67.4
20	88.02	1200	0.244	48.1
25	74.73	1500	0.207	25.2
30	65.37	1800	0.181	-0.2
35	58.37	2100	0.162	-27.3
40	52.92	2400	0.147	-55.9
45	48.54	2700	0.135	-85.5
50	44.93	3000	0.125	-116.0
55	41.89	3300	0.116	-147.2
60	39.30	3600	0.109	-179.0
65	37.06	3900	0.103	-211.4
70	35.10	4200	0.097	-244.3
75	33.36	4500	0.093	-277.5
80	31.82	4800	0.088	-311.1
85	30.43	5100	0.084	-345.1

PROJECT:	Abbotts Residential Development
PROJECT No.:	332-4581
FILE:	Modified Rational Method
DATE:	2019.09.25
DESIGN:	GC
CHECK:	

CROZIER CONSULTING ENGINEERS

Modified Rational Method Storage Sizing (50-Year Storm)

Peak Flow

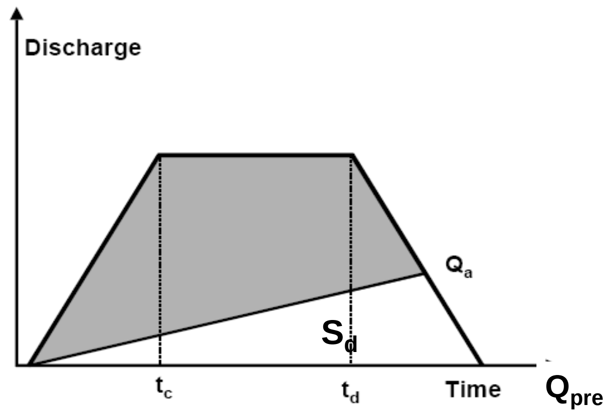
$Q_{post} = 0.0028 \cdot C_{post} \cdot i_{(T_d)} \cdot A$

Intensity

$i_{(T_d)} = A\,(T_d)^{B}$

Storage

$S_d = Q_{post} \cdot T_d - Q_{pre}\,(T_d + T_c)\,/\,2$



Pre-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	162.63
Return Period	50 yr		
Time of Concentration (min)	10		
Coeff A	43.5		
Coeff B	-0.736		
Runoff Coeff (Unadjusted)	0.40	Flow (m^3/s)	0.302
Area (ha)	1.66		

Post-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	162.63
Return Period	50 yr		
Time of Concentration (min)	10		
Coeff A	43.5		
Coeff B	-0.736		
Runoff Coeff (unadjusted)	0.60	Uncont. Flow (m^3/s)	0.451
Area (ha)	1.66		

Target Flow (m^3/s)	0.302

REQUIRED STORAGE VOLUME:	**89.3**

Storage Volume Determination (Detailed)

T_d	i	T_d	Q_{Uncont}	S_d
min	*mm/hr*	*sec*	*m^3/s*	*m^3*
10	162.63	600	0.451	89.3
15	120.67	900	0.335	74.6
20	97.65	1200	0.271	53.0
25	82.86	1500	0.230	27.4
30	72.45	1800	0.201	-1.0
35	64.68	2100	0.179	-31.3
40	58.63	2400	0.163	-63.1
45	53.76	2700	0.149	-96.2
50	49.75	3000	0.138	-130.1
55	46.38	3300	0.129	-165.0
60	43.50	3600	0.121	-200.4
65	41.01	3900	0.114	-236.5
70	38.83	4200	0.108	-273.1
75	36.91	4500	0.102	-310.1
80	35.20	4800	0.098	-347.6
85	33.66	5100	0.093	-385.4

CROZIER CONSULTING ENGINEERS

PROJECT: Abbotts Residential Development
PROJECT No.: 332-4581
FILE: Modified Rational Method
DATE: 2019.09.25
DESIGN: GC
CHECK:

Modified Rational Method Storage Sizing (100-Year Storm)

Peak Flow

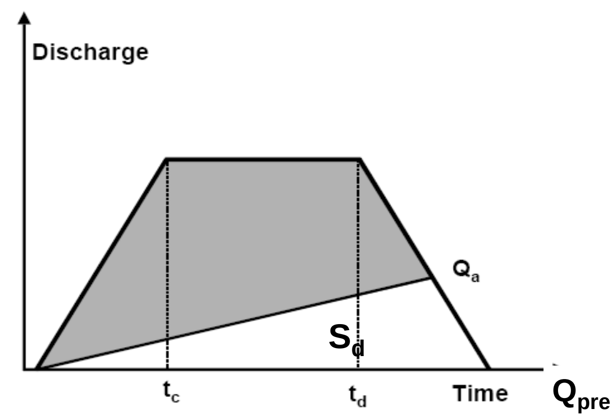
$$Q_{post} = 0.0028 \bullet C_{post} \bullet i_{(T_d)} \bullet A$$

Intensity

$$i_{(T_d)} = A\,(T_d)^{B}$$

Storage

$$S_d = Q_{post} \bullet T_d - Q_{pre}\,(T_d + T_c) / 2$$



Pre-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	178.98
Return Period	100 yr		
Time of Concentration (min)	10		
Coeff A	47.7		
Coeff B	-0.738		
Runoff Coeff (Unadjusted)	0.40	Flow (m^3/s)	0.333
Area (ha)	1.66		

Post-Development Scenario Data			
Inputs		**Outputs**	
IDF Location	Owen Sound	Intensity (mm/hr):	178.98
Return Period	100 yr		
Time of Concentration (min)	10		
Coeff A	47.7		
Coeff B	-0.738		
Runoff Coeff (unadjusted)	0.60	Uncont. Flow (m^3/s)	0.497
Area (ha)	1.66		

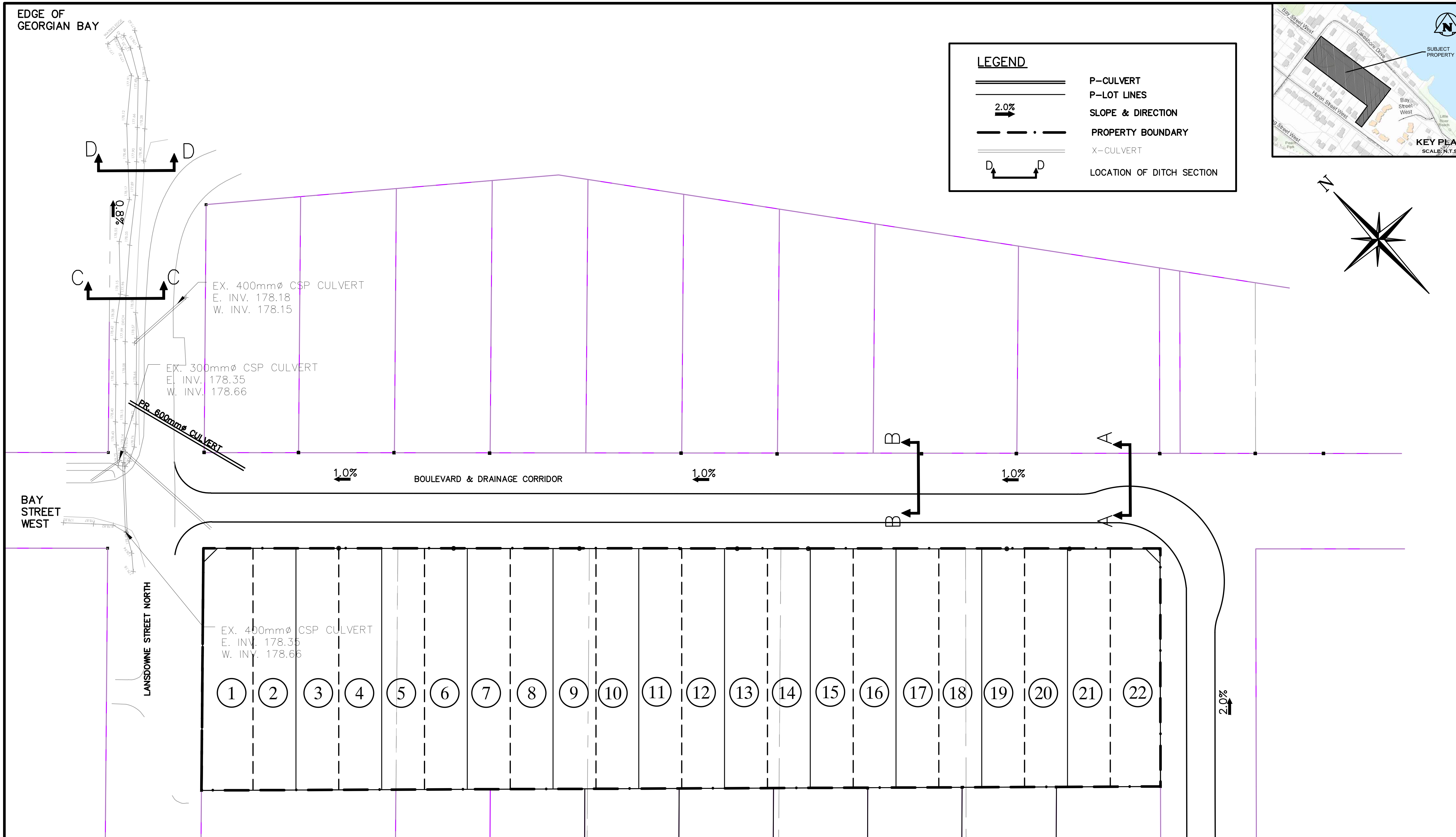
Target Flow (m^3/s)	0.333

REQUIRED STORAGE VOLUME:	**98.3**

Storage Volume Determination (Detailed)				
T_d	i	T_d	Q_{Uncont}	S_d
min	*mm/hr*	*sec*	*m^3/s*	*m^3*
10	178.98	600	0.497	98.3
15	132.69	900	0.368	81.8
20	107.31	1200	0.298	57.8
25	91.02	1500	0.253	29.4
30	79.56	1800	0.221	-1.9
35	71.00	2100	0.197	-35.5
40	64.34	2400	0.179	-70.7
45	58.98	2700	0.164	-107.1
50	54.57	3000	0.151	-144.7
55	50.86	3300	0.141	-183.1
60	47.70	3600	0.132	-222.3
65	44.96	3900	0.125	-262.1
70	42.57	4200	0.118	-302.5
75	40.46	4500	0.112	-343.3
80	38.58	4800	0.107	-384.6
85	36.89	5100	0.102	-426.3

APPENDIX E

FlowMaster Design Sheets



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TEMPORARY BENCHMARKS

TBM#1–
TBM#2–
TBM#3–

No.	ISSUE	DATE: MM/DD/YYYY
1	ISSUED FOR DRAFT PLAN APPROVAL	09/27/2019

DRAFT
FOR DISCUSSION PURPOSES ONLY

ABBOTTS RESIDENTIAL DEVELOPMENT
TOWN OF THE BLUE MOUNTAINS

Drawing
INTERNAL AND EXTERNAL DRAINAGE CROSS SECTIONS

CROZIER
CONSULTING ENGINEERS
THE HARBOUREDGE BUILDING,
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COLLINGWOOD, ON L9Y 4R3
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Drawn By N.L.	Design By N.L.	Project 332–4581	
Check By K.M.	Check By G.C.	Scale 1:750	Drawing 001

J:\300\332 - Hahne Property\4581-Abbotts Condo Dev\CAD\Civil_Sheets\4581_700-FIG.dwg, 001, 2019-10-03 11:15:29 AM, nlefebvre

Worksheet for Drainage Buffer Section A-A

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	1.00000	%
Discharge	0.83	m^3/s

Section Definitions

Station (m)	Elevation (m)
0+00	0.99
0+03	0.00
0+06	0.99

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.99)	(0+03, 0.00)	0.030
(0+03, 0.00)	(0+06, 0.99)	0.030

Options

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

Results

Normal Depth	0.47	m
Elevation Range	0.00 to 0.99 m	
Flow Area	0.67	m^2
Wetted Perimeter	3.01	m
Hydraulic Radius	0.22	m
Top Width	2.86	m
Normal Depth	0.47	m
Critical Depth	0.43	m
Critical Slope	0.01574	m/m

Worksheet for Drainage Buffer Section A-A

Results

Velocity		1.23	m/s
Velocity Head		0.08	m
Specific Energy		0.55	m
Froude Number		0.81	
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	0.47	m
Critical Depth	0.43	m
Channel Slope	1.00000	%
Critical Slope	0.01574	m/m

Cross Section for Drainage Buffer Section A-A

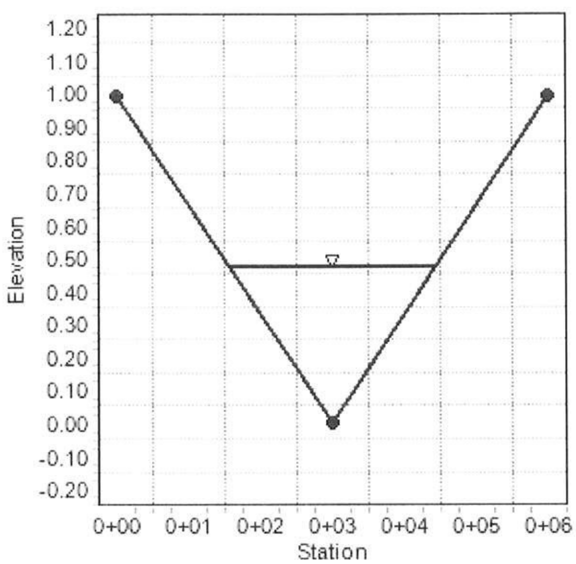
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	1.00000	%
Normal Depth	0.47	m
Discharge	0.83	m³/s

Cross Section Image



Worksheet for Drainage Buffer Section B-B

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	1.00000	%
Discharge	0.83	m^3/s
Section Definitions		

Station (m)	Elevation (m)
0+00	0.71
0+02	0.00
0+04	0.71

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.71)	(0+02, 0.00)	0.030
(0+02, 0.00)	(0+04, 0.71)	0.030

Options

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

Results

Normal Depth	0.47	m
Elevation Range	0.00 to 0.71 m	
Flow Area	0.68	m^2
Wetted Perimeter	3.02	m
Hydraulic Radius	0.22	m
Top Width	2.87	m
Normal Depth	0.47	m
Critical Depth	0.43	m
Critical Slope	0.01574	m/m

Worksheet for Drainage Buffer Section B-B

Results

Velocity		1.23	m/s
Velocity Head		0.08	m
Specific Energy		0.55	m
Froude Number		0.81	
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	0.47	m
Critical Depth	0.43	m
Channel Slope	1.00000	%
Critical Slope	0.01574	m/m

Cross Section for Drainage Buffer Section B-B

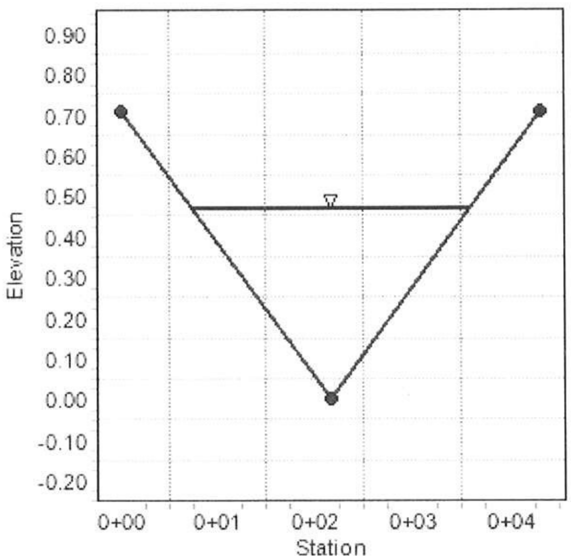
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	1.00000	%
Normal Depth	0.47	m
Discharge	0.83	m^3/s

Cross Section Image



Worksheet for 600mm Diameter Culvert Crossing

Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.019	
Channel Slope	0.03000	m/m
Normal Depth	0.46	m
Diameter	0.60	m

Results

Discharge	0.68	m³/s
Flow Area	0.23	m²
Wetted Perimeter	1.28	m
Hydraulic Radius	0.18	m
Top Width	0.51	m
Critical Depth	0.53	m
Percent Full	76.7	%
Critical Slope	0.02357	m/m
Velocity	2.92	m/s
Velocity Head	0.44	m
Specific Energy	0.90	m
Froude Number	1.38	
Maximum Discharge	0.78	m³/s
Discharge Full	0.73	m³/s
Slope Full	0.02622	m/m
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
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	76.67	%
Downstream Velocity	Infinity	m/s

Worksheet for 600mm Diameter Culvert Crossing

GVF Output Data

Upstream Velocity	Infinity	m/s
Normal Depth	0.46	m
Critical Depth	0.53	m
Channel Slope	0.03000	m/m
Critical Slope	0.02357	m/m

Cross Section for 600mm Diameter Culvert Crossing

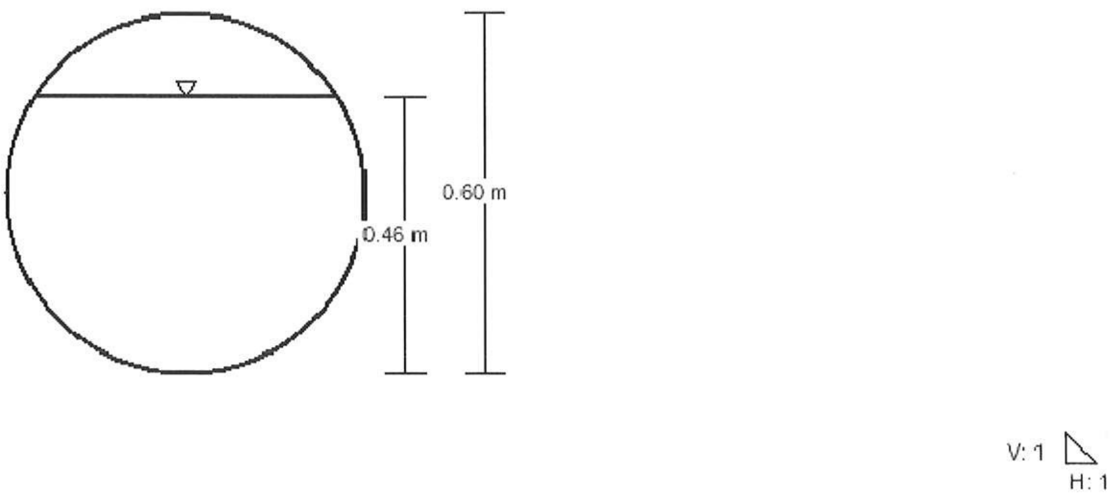
Project Description

Friction Method	Manning Formula
Solve For	Discharge

Input Data

Roughness Coefficient	0.019	
Channel Slope	0.03000	m/m
Normal Depth	0.46	m
Diameter	0.60	m
Discharge	0.68	m^3/s

Cross Section Image



Worksheet for Lansdowne Post-Con Ditch C-C

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00800	m/m
Discharge	1.14	m³/s
Section Definitions		

Station (mm)	Elevation (m)
-0+500	178.86
0+000	178.35
2+200	178.15
3+400	177.96
5+700	178.58
7+800	178.65

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(-0+500, 178.86)	(2+200, 178.15)	0.030
(2+200, 178.15)	(3+400, 177.96)	0.030
(3+400, 177.96)	(5+700, 178.58)	0.030
(5+700, 178.58)	(7+800, 178.65)	0.030

Options

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

Results

Normal Depth	0.44	m
Elevation Range	177.96 to 178.86 m	
Flow Area	1.08	m²
Wetted Perimeter	5.16	m

Worksheet for Lansdowne Post-Con Ditch C-C

Results

Hydraulic Radius		0.21	m
Top Width		5.06	m
Normal Depth		0.44	m
Critical Depth		0.39	m
Critical Slope		0.01603	m/m
Velocity		1.05	m/s
Velocity Head		0.06	m
Specific Energy		0.49	m
Froude Number		0.73	
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	0.44	m
Critical Depth	0.39	m
Channel Slope	0.00800	m/m
Critical Slope	0.01603	m/m

Cross Section for Lansdowne Post-Con Ditch C-C

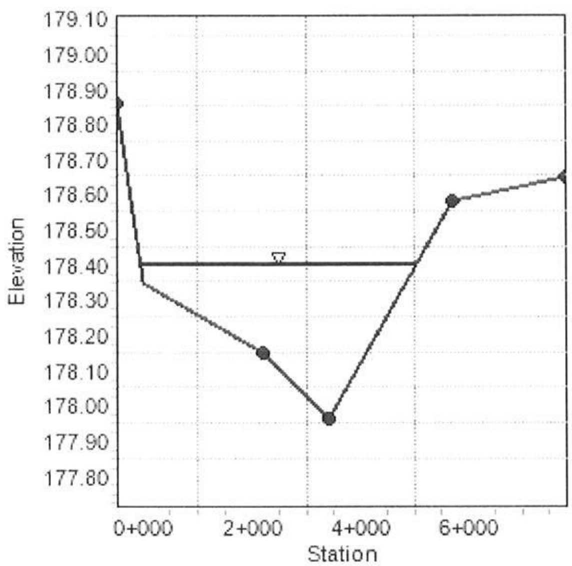
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00800	m/m
Normal Depth	0.44	m
Discharge	1.14	m^3/s

Cross Section Image



Worksheet for Lansdowne Post-Con DItch D-D

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.00800	m/m
Discharge	1.14	m³/s
Section Definitions		

Station (mm)	Elevation (m)
0+000	178.77
4+050	178.17
5+450	177.89
6+950	178.36
11+700	178.50

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+000, 178.77)	(4+050, 178.17)	0.030
(4+050, 178.17)	(5+450, 177.89)	0.030
(5+450, 177.89)	(6+950, 178.36)	0.030
(6+950, 178.36)	(11+700, 178.50)	0.030

Options

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

Results

Normal Depth	0.51	m
Elevation Range	177.89 to 178.77 m	
Flow Area	1.15	m²
Wetted Perimeter	6.05	m
Hydraulic Radius	0.19	m

Worksheet for Lansdowne Post-Con DItch D-D

Results

Top Width		5.93	m
Normal Depth		0.51	m
Critical Depth		0.44	m
Critical Slope		0.01545	m/m
Velocity		0.99	m/s
Velocity Head		0.05	m
Specific Energy		0.56	m
Froude Number		0.72	
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	0.51	m
Critical Depth	0.44	m
Channel Slope	0.00800	m/m
Critical Slope	0.01545	m/m

Cross Section for Lansdowne Post-Con DItch D-D

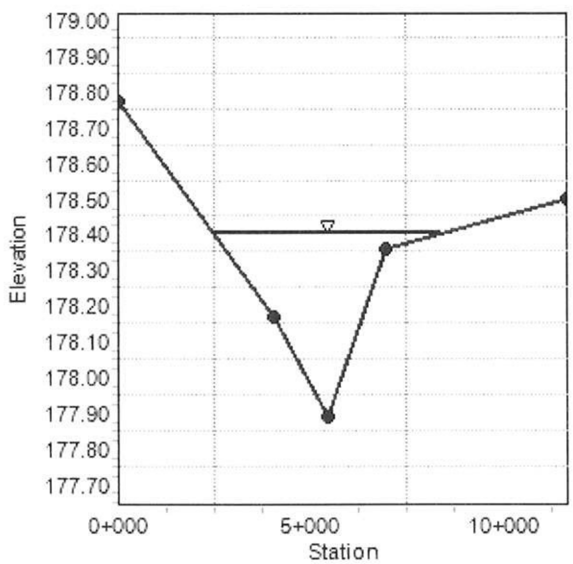
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

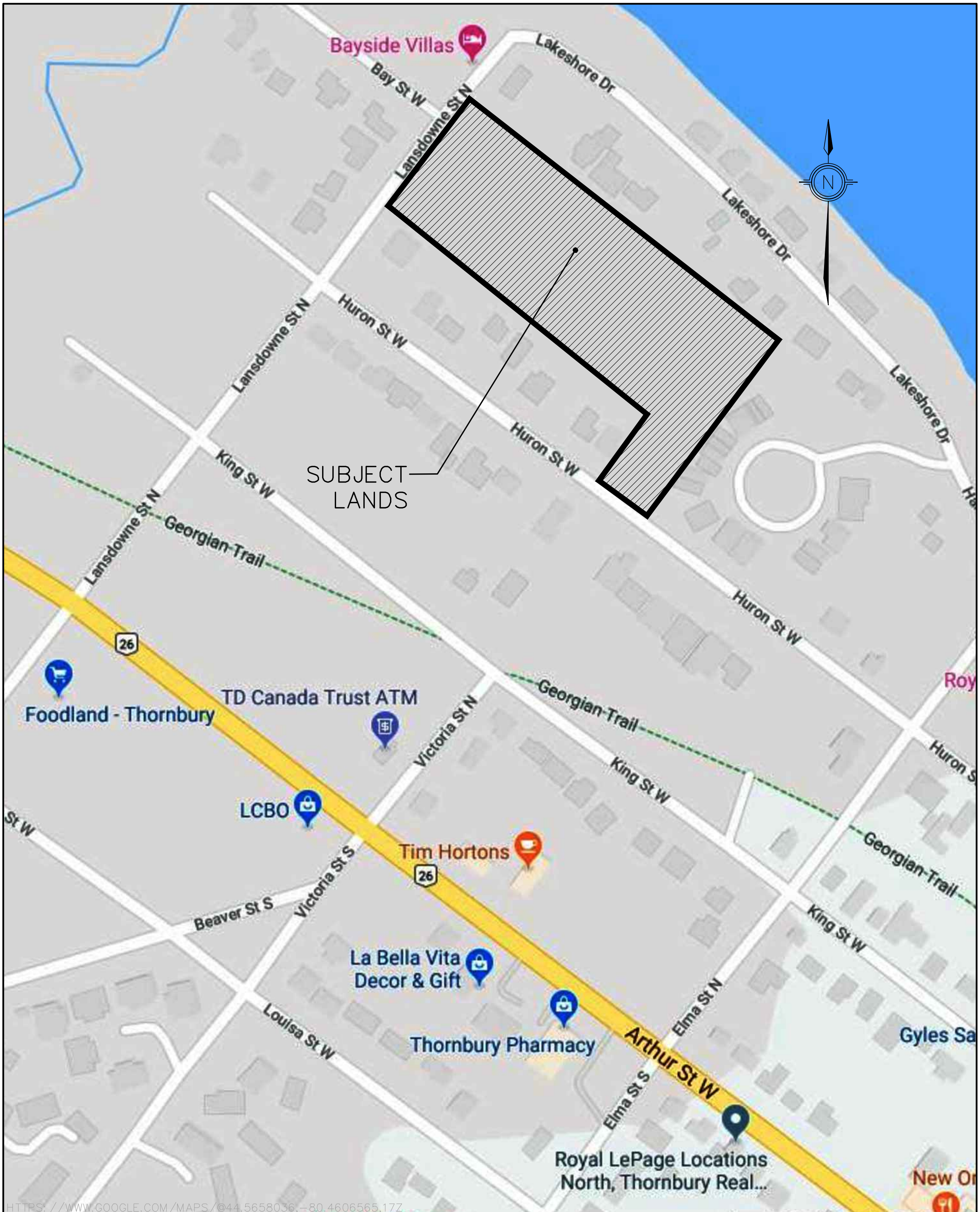
Input Data

Channel Slope	0.00800	m/m
Normal Depth	0.51	m
Discharge	1.14	m³/s

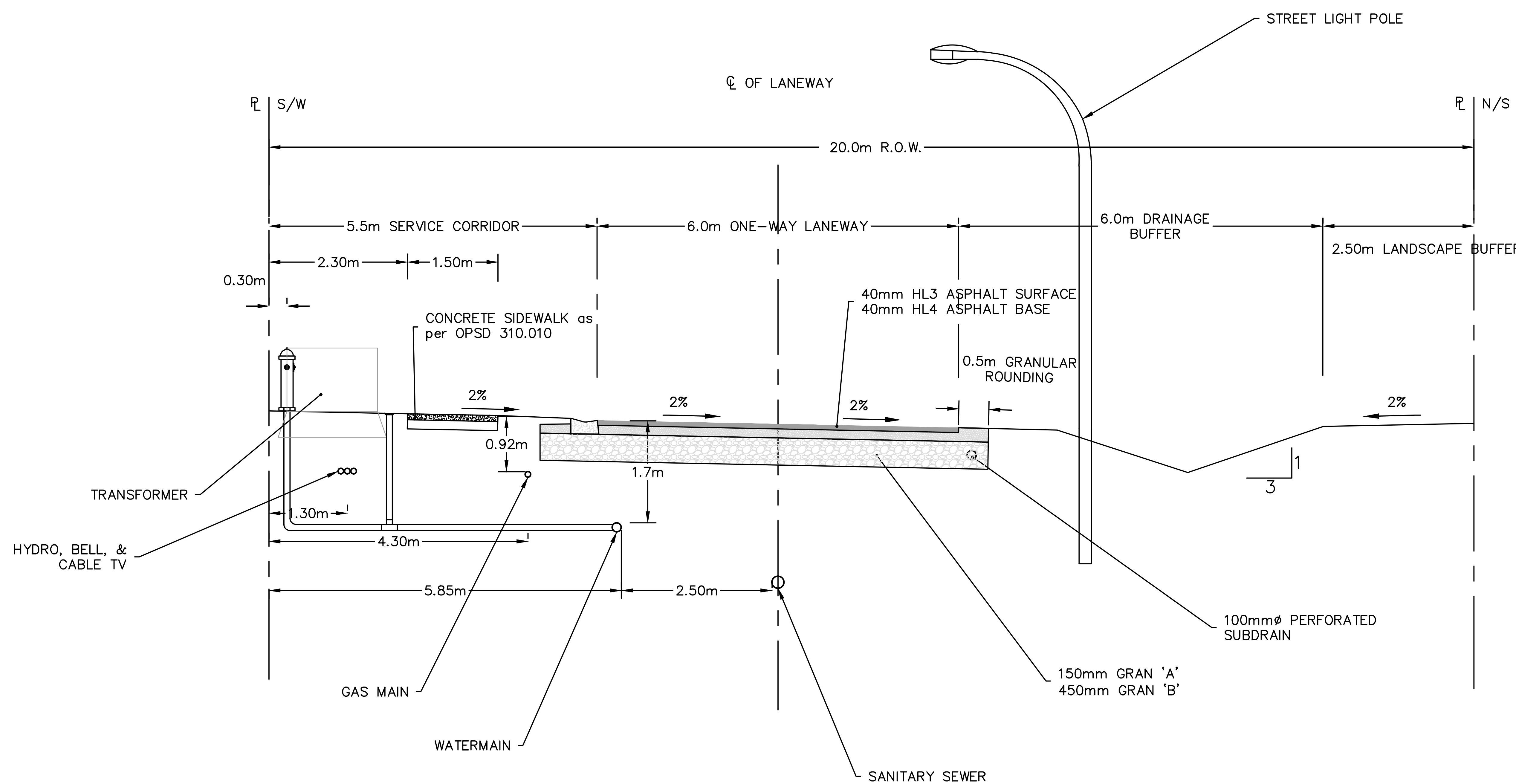
Cross Section Image



FIGURES



Legend	Project	
= SUBJECT LANDS	ABBOTT'S RESIDENTIAL CONDOMINIUMS DEVELOPMENT	CROZIER 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
	Drawing SITE LOCATION PLAN	Drawn By N.L. / Design By G.C. / Project 332–4581 Scale N.T.S. / Date 05/03/2019 / Check By K.A.M. / Drawing FIG. 1



NOTES:

REFER TO TOWN OF THE BLUE MOUNTAINS ENGINEERING STANDARDS FOR "URBAN" ROAD SECTIONS FOR 20M RIGHT OF WAY

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TEMPORARY BENCHMARKS

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TBM#2–
TBM#3–

Town

No.	ISSUE	DATE: MM/DD/YYYY
1	ISSUED FOR DRAFT PLAN APPROVAL	10/04/2019

Engineer

Engineer

DRAFT
FOR DISCUSSION PURPOSES ONLY

Project: ABBOTTS RESIDENTIAL DEVELOPMENT
TOWN OF THE BLUE MOUNTAINS

Drawing: PROPOSED ROAD SECTION

CROZIER CONSULTING ENGINEERS

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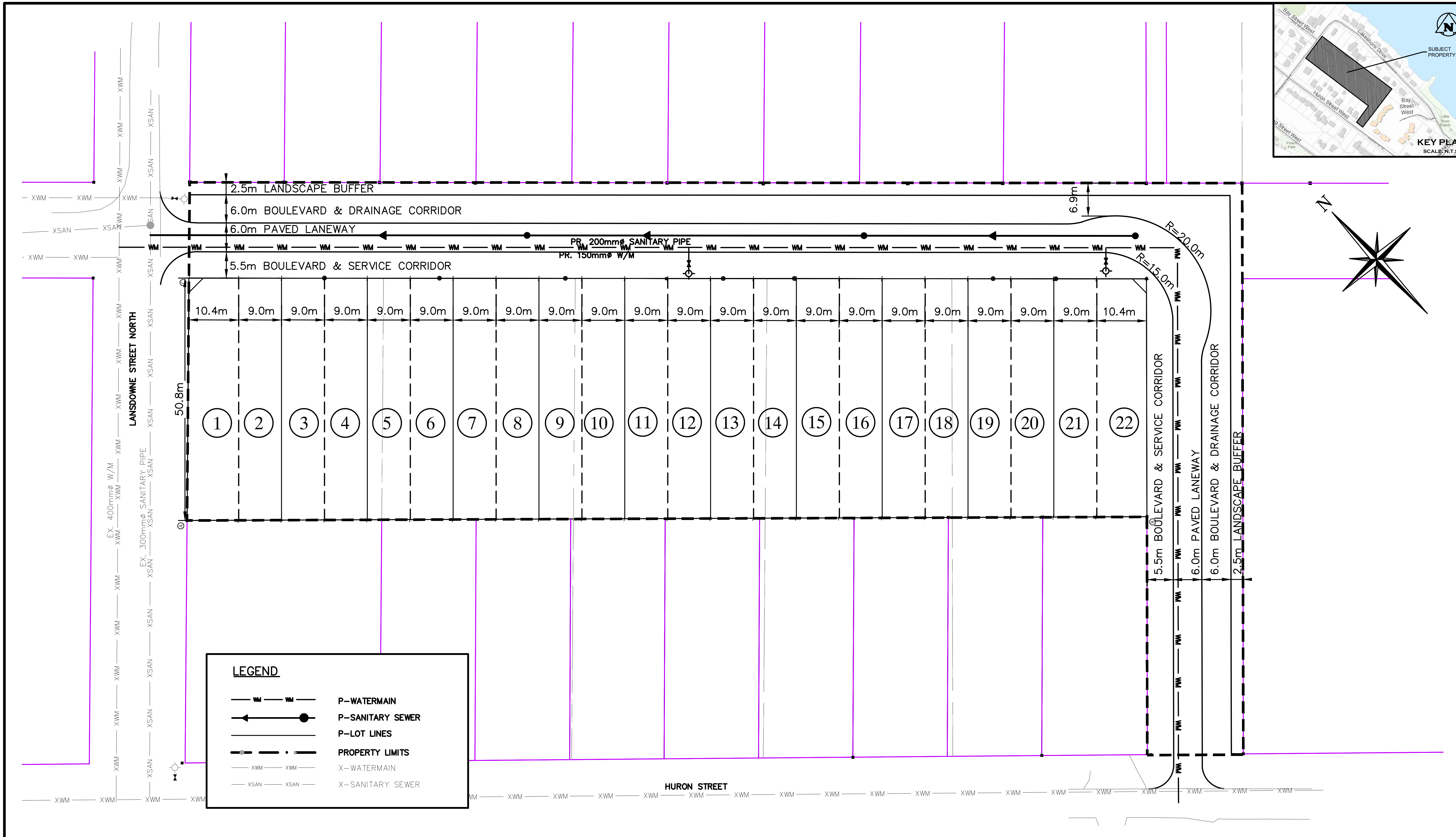
Drawn By N.L.	Design By J.P.	Project 332-4581	
Check By K.M.	Check By G.C.	Scale 1:750	Drawing FIG. 2



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TEMPORARY BENCHMARKS

TBM#1–
TBM#2–
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No.	ISSUE	DATE: MM/DD/YYYY
1	ISSUED FOR DRAFT PLAN APPROVAL	09/27/2019

DRAFT
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ABBOTTS RESIDENTIAL DEVELOPMENT
TOWN OF THE BLUE MOUNTAINS

Drawing
PRELIMINARY SERVICING PLAN

CROZIER
CONSULTING ENGINEERS
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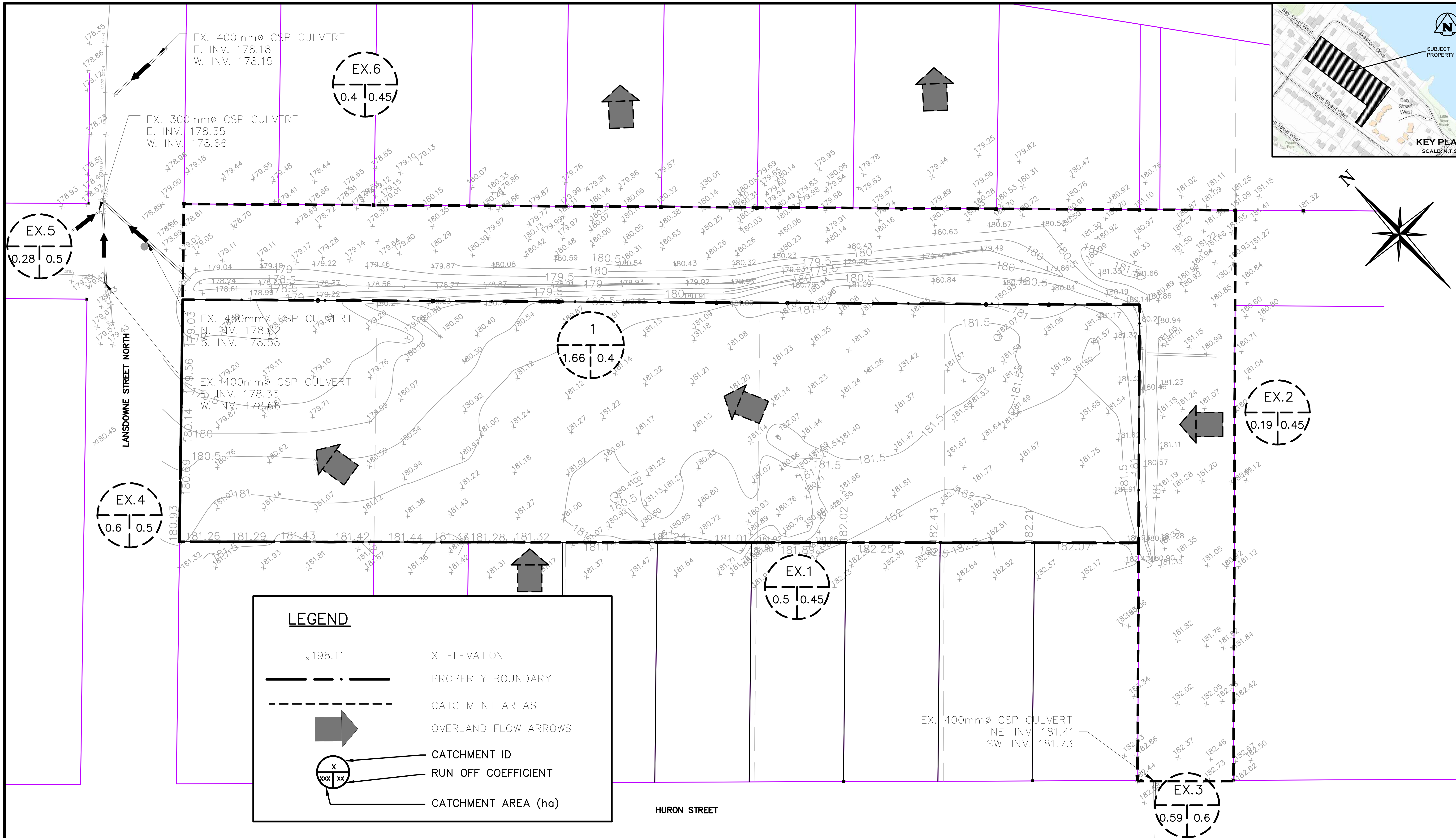
Drawn By	N.L.	Design By	N.L.	Project	332-4581
Check By	K.M.	Check By	G.C.	Scale 1:750	Drawing FIG. 3

J:\300\332 - Hahne Property\4581-Abbotts Condo Dev\CAD\Civil_Sheets\4581_700-FIG.dwg, FIG. 3, 2019-10-03 11:08:39 AM, nlefebvre



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Town

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1	ISSUED FOR DRAFT PLAN APPROVAL	09/27/2019

Engineer

Engineer

DRAFT
FOR DISCUSSION PURPOSES ONLY

Project

ABBOTTS RESIDENTIAL DEVELOPMENT
TOWN OF THE BLUE MOUNTAINS

Drawing

PRE-DEVELOPMENT DRAINAGE AREAS

CROZIER
CONSULTING ENGINEERS

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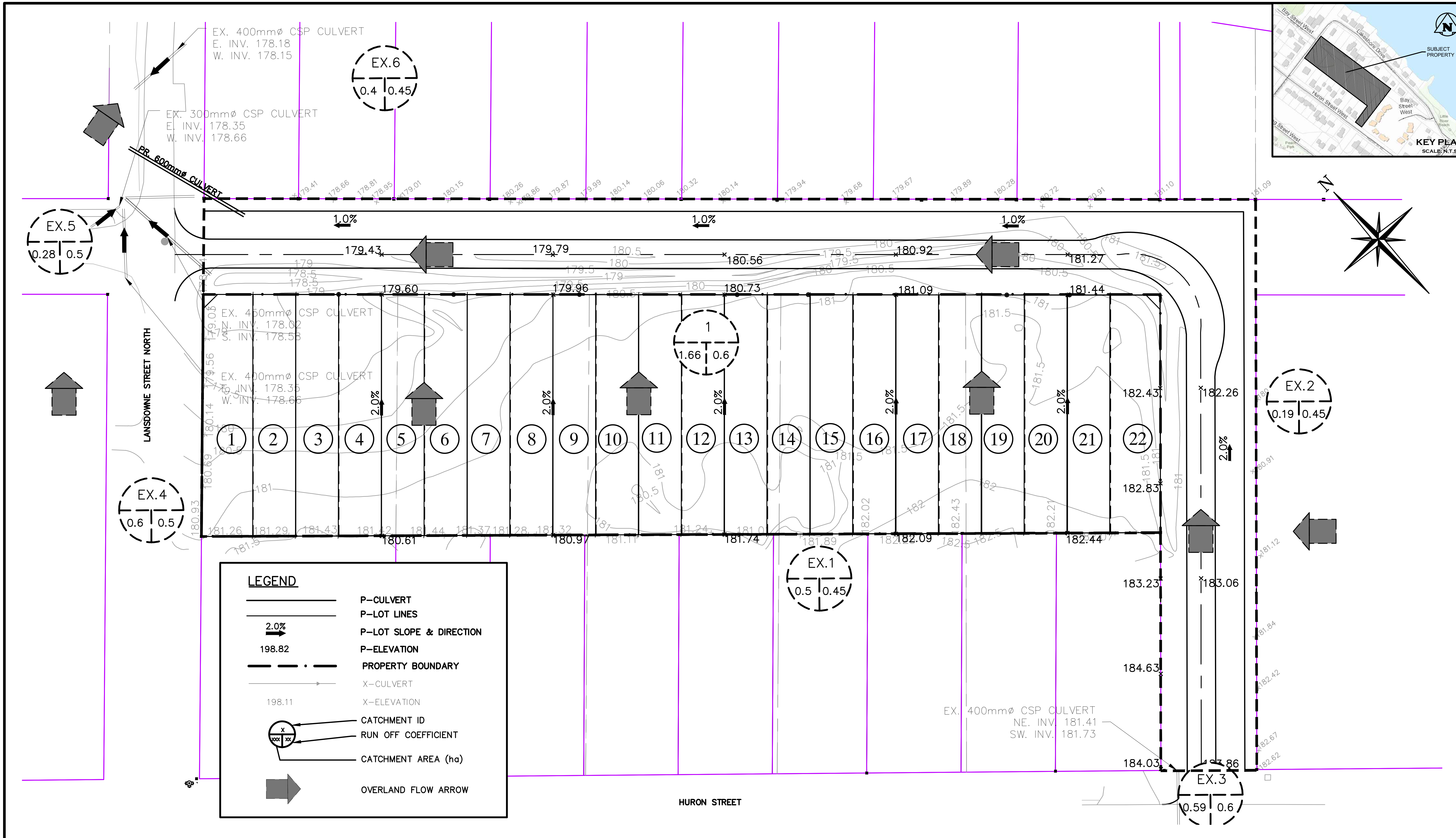
Drawn By N.L.	Design By N.L.	Project	332-4581
Check By K.M.	Check By G.C.	Scale 1:750	Drawing FIG. 4

J:\300\332 - Hahne Property\4581-Abbotts Condo Dev\CAD\Civil_Sheets\4581_700-FIG.dwg, FIG. 4, 2019-10-03 11:10:23 AM, nlefebvre



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ABBOTTS RESIDENTIAL DEVELOPMENT
TOWN OF THE BLUE MOUNTAINS

Drawing
POST DEVELOPMENT DRAINAGE PLAN

CROZIER
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Drawn By N.L.	Design By N.L.	Project	332–4581
Check By K.M.	Check By G.C.	Scale 1:750	Drawing FIG. 5