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FUNCTIONAL SERVICING REPORT

Millbrook South East Subdivision

East Side of County Road 10, South of Fallis Line
Community of Millbrook
Township of Cavan Monaghan
County of Peterborough

April 2021
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Prepared For: **Vargas Properties Inc.**

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VALDOR WATER RESOURCES

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1.0 INTRODUCTION

Valdor Engineering Inc. has been retained by Vargas Properties Inc. to provide consulting engineering services for the proposed Millbrook South East Subdivision located on 29.5 hectare parcel on the east side of County Road 10, south of Fallis Line, in the Community of Millbrook, Township of Cavan Monaghan, County of Peterborough as illustrated in **Figure 1**.

1.1 Existing Conditions

The subject site is bounded to the west by County Road 10 and external lands fronting County Road 10, to the north by agricultural lands and the future Fallis Line extension, to the east by forested lands, and to the south by residential development along Nina Court. The southern portion of the site is traversed by a tributary of Baxter Creek which flows in an easterly direction. The geotechnical and topographical conditions of the site are summarized as follows:

1.1.1 Geotechnical

A Geotechnical Investigation Report prepared by GHD Ltd. (March 8, 2021) for the subject site included thirteen boreholes and six test pits, ranging in depth from 3.0 m to 8.2 m. Based on the investigation it was determined that the site is covered by a topsoil layer having a depth of approximately 150-300mm underlain by silty sand and then glacial till and/or silty clay. The boreholes are included in **Appendix "H"**.

1.1.2 Topography

The surface condition of the subject site can be generally described having a rolling topography. Based on a recent topographic survey of the site, the property generally slopes down from the intersection of County Road 10 and Fallis Line in a southeasterly direction towards the tributary of Baxter Creek. Based on an existing elevation of approximately 255.6 m at the County Road 10 and Fallis Line intersection, and an existing elevation of 209.6 at the south-east limit of the development, the differential of 46.0 m equates to an overall average slope of approximately 6.4% which is considered to be relatively steep.

1.2 Proposed Development

The proposed development consists of a mix of lots for detached dwellings and street townhomes as well a block for medium density housing and a commercial block. The lot frontages for the detached dwellings will range from 10.7m to 15.9m while the townhomes will consist of 7.6m frontages. Access for the subdivision will consist of a road network with a road connection off Fallis Line and a pedestrian access to Nina Court at the south limit of the subdivision. A block of land has been established for a stormwater management facility to control and treat stormwater runoff. The remainder of the site, associated with the tributary of Baxter Creek and the forested lands in the eastern portion of the site, will be retained in environmental protection blocks.

A copy of the Draft Plan of Subdivision as well as the Conceptual Master Plan is contained in **Appendix "A"** together with the calculation of the equivalent population which is summarized in **Table 1**.

Table 1. Development Statistics – Proposed Draft Plan

Land Use	**Area** (Ha)	**Residential Units** (No.)	**Equivalent Population** (persons)
Detached Dwellings	6.24	128	448
Street Townhomes	1.28	48	168
Commercial	1.30		5
Stormwater Management Pond	1.62		
Natural Heritage Systems	15.46		
Parkland & Trails	0.36		
Roads & Road Widenings	3.22		
TOTAL	**29.48**	**176**	**621**

1.3 Purpose of Report

This report has been prepared in support of the application for draft plan approval for the subject property. The primary intent of the report is to demonstrate the viability of water and wastewater servicing, storm drainage and stormwater management, grading as well as vehicular and pedestrian access for the proposed development with respect to applicable guidelines, policies and design criteria.

This report has been prepared based on a review of the topographic survey and background studies, discussions with municipal staff and a visit to the site. This document provides guidance for detailed engineering design of the subdivision. A **Preliminary Servicing & Grading Plan** is included in a pouch at the rear of the report.

1.4 Approving Authorities

This report will be circulated for review, comment and approval to:

1. The Township of Cavan Monaghan;
2. The County of Peterborough; and
3. The Otonabee Region Conservation Authority (ORCA).

2.0 WATER SERVICING

The existing Millbrook water servicing system consists of a water treatment facility, with water taken from three local wells, a water storage tank and a network of watermains that service most of the existing urban area of the community.

The existing Millbrook Water Treatment Plant (WTP) consists of 3 wells, each with 25 L/s capacity, chlorine disinfection and a chlorine contact tank. The existing water storage tank was built in 1976 and is located on the east end of Millbrook on a local high point of land. The existing 10.4m diameter tank has a useable storage capacity of 1,410m^3 with a top water level at an elevation of 278.0m.

The Township of Cavan Monaghan completed a Class Environmental Assessment (Class EA) in June 2014 to investigate on the alternatives to address concerns associated with the water storage and water servicing needs. In this regard, the expansion of the existing urban boundary of Millbrook required additional water storage and expansion of the existing water servicing network to the new development area.

As a result, a new, larger water storage tank was constructed on the Township Office site, a new watermain was constructed to connect to the tank to the existing water supply main and the original water storage tank in Millbrook as decommissioned. A booster station was also constructed within the Township Office site to ensure proper minimum fire pressures are maintained during maximum day demand throughout the higher elevations within the development.

The Township has recently initiated a Water and Wastewater Master Servicing Study as part of a Municipal Class Environmental Assessment to examine water and wastewater servicing alternatives within the current urban boundary and beyond. This study should consider the proposed draft plan as well as the ultimate development for the subject lands.

The following is a summary of the water servicing requirements for the subject site.

2.1 Domestic Demand

The domestic water demand is to be calculated using the Township and Ministry of the Environment design standards which includes the following parameters:

Residential Average Day Demand: 450 L/person/day
Maximum Day Factor: 2.00
Peak Hour Factor 3.00

A detailed tabulation of the domestic water demand calculation is detailed in **Table B1** of **Appendix "B"**. The domestic demands for the proposed draft plan and the ultimate development are summarized in **Table 2.**

Table 2. Domestic Water & Fire Flow Demand

Land Use	Equivalent Population (Persons)	Domestic Demand (L/min)	Maximum Day Demand (L/min)	Peak Hour Demand (L/min)	Fire Flow (L/min)	Maximum Day Plus Fire Flow (L/min)
Detached Dwellings	448	140	280	420	8,000	
Street Townhomes	168	52.5	105	157.5	7,000	
Commercial Block	5	1.6	3.1	4.7		
TOTAL	**621**	**194.1**	**388.1**	**582.2**	**8,000**	**8,388.1**

2.2 External Watermains

The location of the existing Water Treatment Plant, existing water storage tank and the existing water booster station are indicated in **Figure 2**. The subject site will be serviced by an 250mm diameter watermain on Fallis Line which has been stubbed immediately east of County Road 10. To complete a loop, the proposed watermain system will also connect to the existing 150mm diameter watertmain on Nina Court at the south limit of the site,

2.3 Local Watermains & Service Connections

The local water distribution system within the subdivision will consist of watermains ranging in diameter from 150mm to 250mm. This water system will connect to the trunk watermain.

In accordance with Township standards the individual detached dwellings are each to have separate water connections. Based on Ontario Building Code (OBC 2012) regulations (7.6.3.4.(1) and (5) and Table 7.6.3.4), the dwellings will be serviced with 25mm diameter water connections given that it is anticipated that the dwellings will each have more than 16 fixture units.

Water meters are to be purchased from the Township and will be installed in the basement of each dwelling with a remote readout device located on the exterior ground floor wall of the house. Generally, residential water meters are selected to be one size smaller than the water service and therefore 20mm x 25mm water meters will be installed.

The configuration of the site watermain is illustrated on the **Preliminary Site Servicing & Grading Plan**. A copy of the Township standard water service connection and water meter details is included in **Appendix "B"**.

2.4 Fire Protection

The fire flow required for the proposed dwelling units was calculated using the criteria indicated in the *Water Supply for Public Fire Protection Manual*, 1999, by the Fire Underwriters Survey (FUS). The calculation incorporates various parameters such as coefficient for fire-resistant construction, an area reduction accounting for a fire-resistant (one hour rating) protection, a reduction for low-hazard occupancies, and a factor for neighbouring building proximity.

The calculation was completed to reflect the governing conditions which are the largest detached dwelling and the largest interior townhouse unit. Based on the calculations, the minimum fire suppression flow required for the detached dwellings and the townhouse units is 8,000 L/min and 7,000 L/min respectively. The detailed fire flow calculation is shown in **Table B2-1** to **Table B2-2** of **Appendix "B"**. In accordance with the Township standards, this flow must be available at the nearest hydrant with a minimum pressure of 140 KPa.

Fire hydrants will be provided along the municipal roads such that a fire hydrant will be available within 90m of the principle entrance of each unit as set out in the Ontario Building Code (OBC 2012). A copy of the standard fire hydrant detail is included in **Appendix "B"**.

The City is currently conducting a master servicing study as part of the Growth Management Plan and based on a Council Meeting held on October 18th 2021 there is unused water treatment capacity available in the system to accommodate this development with further plans to upgrade the current water system to accommodate future growth. Please refer to Appendix 'K' which provides an update to the Growth Management Study.

3.0 WASTEWATER SERVICING

The community of Millbrook is currently serviced by the existing Millbrook Wastewater Treatment Plant (WWTP) located at the east limit of Centennial Lane. This WWTP was built in 1975 and the plant was upgraded in 2004 to improve the treatment quality.

In May 2013 the Township of Cavan Monaghan completed a Class Environmental Assessment (Class EA) which investigated the alternatives to address concerns associated with the existing WWTP, in particular, the fact that it did not have sufficient capacity to sustain the projected growth. In addition, the existing plant is at the end of its useful life and requires substantial upgrade and rehabilitation. Based on the recommendations of the EA, the expansion and upgrade of the existing Millbrook WWTP was completed in 2015 to include a high-level tertiary treatment facility that is able to provide improved effluent quality to meet the current effluent discharge criteria, as well as the increased capacity to accommodate future flows.

The Township has recently initiated a Water and Wastewater Master Servicing Study as part of a Municipal Class Environmental Assessment to examine water and wastewater servicing alternatives within the current urban boundary and beyond. This study should consider the proposed draft plan as well as the ultimate development for the subject lands.

The location of the existing sanitary sewers and the WWTP is indicated in **Figure 3**. The following is a summary of the wastewater servicing analysis for the subject site.

3.1 Wastewater Loading

The wastewater loading is to be calculated using the Township engineering design standards which include the following parameters:

Residential Average Daily Flow: 450 L/person/day

Residential Peaking Factor: $K_H = 1 + \frac{14}{4 + \sqrt{P}}$

Where: K_H = Harmon Peaking Factor (Max. 4.0, Min. 2.75)
p = Population in thousands

Extraneous Flow, *I*: 0.28 L/s/Ha (Infiltration)

Design Flow, Q = $Q \times K_H + I$

Based on the above criteria the sewage flow calculations are provided in **Table C1** contained in **Appendix "C"** and the wastewater flow for the proposed draft plan and the ultimate development are summarized in **Table 3.**

Table 3. Wastewater Loading Summary

Land Use	**Area** (Ha)	**Equivalent Population** (Persons)	**Average Daily Flow** (L/s)	**Peaking Factor**	**Peak Daily Flow** (L/s)	**Infiltration Rate** (L/s)	**Total Flow** (L/s)
Detached Dwellings	6.24	448	2.33	4.00	9.32	1.76	11.08
Street Townhomes	1.28	168	0.88	4.17	3.65	0.36	4.01
Commercial	1.30	5	1.50	2.50	3.74	0.36	4.10
Roads	3.22					0.91	0.91
TOTAL	**12.04**	**621**	**4.71**		**16.71**	**3.40**	**20.10**

The City is currently conducting a master servicing study as part of the Growth Management Plan and based on a Council Meeting held on October 18th 2021 where an update to the Growth Management Plan was presented there is unused wastewater treatment capacity available in the existing plant to accommodate this development with further plans to upgrade the existing wastewater treatment plant to accommodate future growth.

3.2 External Sanitary Sewers

A 525mm diameter trunk sanitary sewer has been constructed to the south limit of the subject lands. This trunk sewer will be extended northerly along Street "A" to the proposed Fallis Line extension and will service the subject lands as well as future development lands north of the proposed Fallis Line extension, east of County Road 10. The alignment of the trunk sanitary sewer is indicated in **Figure 3**. Refer to Appendix 'K' which provides an update to the Growth Management Study indicating that there is sufficient reserve capacity in the wastewater treatment plant to accommodate additional development.

An analysis of the downstream sanitary sewer was conducted to confirm that there is sufficient capacity. In this regard, a sanitary sewer design sheet has been prepared which indicates that there is sufficient capacity in the downstream sanitary sewer from the subject site to the WWTP to accommodate the subject lands. The design sheet is included in **Appendix "C"** together with the sanitary sewer drainage plan as delineated in **Figure C-1**.

3.3 Local Sanitary Sewers & Service Connections

The subject site will be serviced by a local sanitary system consisting of 200mm diameter sewers. The local sewer will be designed such that the upstream end of each length will have a minimum 1% slope to assist with self-cleansing. In accordance with standard practice, manholes will be provided for maintenance access at a maximum spacing of 120m.

Each dwelling unit will be provided with a 100mm diameter single connection in accordance with Township standards.

4.0 STORM CONVEYANCE SYSTEM

The subject site is located in the Baxter Creek watershed which is one of the twelve watersheds under the jurisdiction of the Otonabee Region Conservation Authority (ORCA). Baxter Creek originates from the Oak Ridges Moraine and flows in an easterly direction and outlets into the Otonabee River. Baxter Creek meets the Otonabee River approximately 20 km upstream of Rice Lake. A map illustrating the Baxter Creek watershed is contained in **Appendix "D"**.

In accordance with Township standards, a major / minor system storm conveyance concept has been incorporated into the functional servicing design for the subject development. The following sections provide a brief summary of the storm drainage components:

4.1 Minor System Design

As per the Township engineering design criteria, the proposed development is to be serviced with a minor storm sewer system that is designed to convey runoff from the 5-year storm event. The rainfall intensity values, *I*, are calculated in accordance with the 2014 rainfall intensity duration frequency (IDF) data for the Peterborough Airport weather station, obtained from Environment Canada. Based on this data the rainfall intensity for the 5- and 100-year rainfall events is calculated as follows:

$$I_5 = \frac{844}{(t+7.5)^{0.78}} \qquad I_{100} = \frac{1697}{(t+10.5)^{0.81}}$$

The peak flows are calculated using the following formula:

$Q = R \times A \times I \times 2.778$ where: Q = peak flow (L/s)
A = area in hectares (ha)
I = rainfall intensity (mm/hr)
R = composite runoff coefficient
t = time of concentration (min)

The proposed storm sewer will discharge to the proposed stormwater management facility (SWM pond) located in the north-west corner of the site.

The IDF curve data is included in **Appendix "D"**. A schematic design of the minor system is illustrated in on the **Preliminary Site Servicing & Grading Plan**.

4.2 Major System Design

The major system will generally be comprised of an overland flow route along the municipal road network directing drainage to a safe outlet. This major system will convey flows which are in excess of the capacity of the minor storm sewer system. The major system flow route is illustrated in on the **Preliminary Servicing & Grading Plan**. Flows from catchment 202 and the residential/commercial block in catchment 201 will be directed to Street 'A' through future road allowances. Major flows will be captured at the low point on Street 'A' and conveyed to the SWM pond via an overland flow route. Major storm flows are conveyed along Street "A" with a flow depth of 0.11 m. This ponding depth is well within the maximum allowable depth of 0.3 m as per typical municipal requirements. The detailed *FlowMaster* calculations are included in **Appendix "D".**

4.3 Foundation Drainage

It is anticipated that the dwellings will have basements and therefore a foundation weeping tile system will be required. In accordance with Township standards, storm service connections are to be provided to each dwelling unit. A hydraulic grade line analysis of the storm sewer system will be completed at the detailed design stage to ensure that basements are protected during the 100-year storm event.

4.4 Roof Drainage

It is anticipated that the proposed dwellings will have conventional peaked roof with eaves troughs and downspouts. As per standard practice the downspouts are to discharge to grade over splash pads, preferably towards sodded areas. Roof downspouts are not to be connected to the storm sewer.

4.5 Floodplain Analysis

The south part of the subject site is traversed by a tributary of Baxter Creek which flows in an easterly direction under Street "A". The total upstream pre-development drainage area of this tributary is approximately 68.29 ha (*Catchments 401-403*), as shown on **Figure 4A**. The total post-development drainage of this tributary is approximately 75.49 ha (Catchments 401-402, 201-206, 403A-403B), as shown on **Figure 4B**, reflecting the post development site conditions.

In order to determine the extent of the Regulatory floodplain at this location, a HEC-RAS model was prepared and the Regulatory floodplain has been delineated for both the pre-development (**Figure 5**) and post-development (**Figure 6**) conditions. As indicated in **Figure 6**, the Regulatory flood plain will be entirely contained within open space blocks and therefore the proposed lots are protected from flooding. Supporting documentation, VO5 and HEC-RAS modelling output, and hydraulic calculations are provided in **Appendix "E"**.

A 2.4 m wide by 1.5 m high concrete box culvert, embedded by 0.30 m, is proposed at Street "A" to accommodate the watercourse. This culvert has been sized to convey the regional flow.

5.0 STORMWATER MANAGEMENT

5.1 Storm Drainage Areas

Based on the topographic survey and the proposed draft plan of subdivision, the following is a summary of the pre and post-development drainage areas.

5.1.1 Pre-Development

Under pre-development conditions, the subject site north of the Baxter Creek tributary (*Catchment 101*, 11.77 ha) drains in a south-easterly direction to the tributary, along with an external area fronting County Road 10 (*Catchment 103*, 4.27 ha). The portion of the site located to the south of the tributary (*Catchment* 102, 2.94 ha) generally drains in a north-easterly direction to the tributary.

For the purpose of the pre-development modelling, drainage boundaries were determined based on the post-development limits, except along the northern boundary where a drainage divide was identified.

The existing land uses comprise of forests and meadows (all catchments), as well as row crops (*Catchment 101*) and some open space and impervious areas (*Catchment 103*). **Figure 7** illustrates the drainage patterns for the pre-development condition.

5.1.2 Post-Development

The proposed development consists of a mix of residential (single detached, townhomes and medium density) and commercial land uses, in addition to the SWM block and the open space blocks to remain undeveloped.

The majority of the subject site (*Catchment 201*, 12.86 ha) will drain to the SWM pond. An external future development area (residential) along County Road 10 (*Catchment 202*, 4.27 ha) has also been identified and assumed to be conveyed to the SWM pond. Drainage will be conveyed to the SWM pond (*Catchment 203*, 1.39 ha) via the storm sewer system, or overland via the road network to the low point on Street “A” adjacent to the SWM pond maintenance access road. Catchment 205 (0.25 ha) is categorized as natural heritage areas and will drain uncontrolled.

Discharge from the SWM pond will be released to the Baxter Creek tributary downstream of the Street “A” crossing via an outlet pipe under Street “A”. **Figure 8** illustrates the drainage patterns for the post-development condition.

Due to grading constraints, the rear of lots along the eastern and southern portion of the site (*Catchment 204*, 2.28 ha) will drain uncontrolled. Adequate overcontrol will be provided by the SWM pond to account for this uncontrolled drainage.

5.2 Stormwater Management Design Criteria

The proposed SWM facility shall be designed to provide the following levels of control as per the requirements of the Ministry of the Environment (MOE), Otonabee Region Conservation Authority (ORCA) and Township of Cavan Monaghan:

- **Quality control**: The permanent pool shall be sized to provide Enhanced (Level 1) treatment of stormwater runoff for the proposed development.
- **Erosion control**: Stormwater runoff from the 25 mm storm event shall be stored and released over a minimum 24-hour period (48 hours preferred).
- **Flood control**: Flood storage and control shall be provided to maintain peak outflows from the pond at or below pre-development levels for the critical of the 6, 12 & 24-hour SCS, the 6, 12 & 24-hour AES, and the 4-hour Chicago storm distributions, for the 2-yr through 100-yr design storm events.

5.3 Stormwater Management Pond Design

A wet detention pond SWM facility is proposed to serve the subject site. The total service area for the SWM pond is approximately 18.52 ha (including the future development area). The proposed SWM pond is located at the south-west corner of the proposed development, as illustrated in **Figure 9**.

Per the Township standards, MOE SWM pond criteria and recommendations in the geotechnical report, the SWM pond design includes 5H:1V side slopes for 3.0 to either side of the normal water level with 4H:1V slopes above, and 3H:1V slopes below, the permanent pool. A 4.0 m wide maintenance access road is provided along the top of the pond with a maximum 10% slope.

5.3.1 Quality Control

Various source controls, conveyance and end-of-pipe SWM facilities were considered to provide the appropriate level of stormwater quality control. Reduced lot grades, rear and side yard swales, and discharge of roof leaders to pervious surfaces will augment the control provided by the SWM facility and promote infiltration where possible. Based on a preliminary review of available controls, it appears that the primary and most effective option to provide water quality control for runoff from the contributing drainage areas is a SWM facility. The options reviewed are as follows:

- Roof Leader to Ponding Areas or Soakaway Pits (Lot Level): The Township design criteria do not address the use of ponding areas or soakaway pits in the rear yards. Roof leaders will discharge directly to pervious surfaces to encourage infiltration and filtration on the lots. Soakaway pits can be an effective means of improving infiltration of stormwater, but require a large area in comparison to typical residential rear yard dimensions. As a result, soakaway pits and ponding areas are not recommended.
- Grassed Swales (Conveyance): Rear and side yard swales will be incorporated into the grading plan. The swales will convey runoff to rear lot

catch basins. The number of rear lot catch basins will be minimized in order to encourage infiltration via swales.

- Stormwater Management Facilities (End-of-Pipe): Based on discussions with the ORCA, SWM facilities are required to provide water quality, extended detention and flood control of stormwater runoff. Stormwater management facilities will be constructed within the subject property.
- Oil/Grit Separation Technologies (End-of-Pipe): These SWMF's can be effective for smaller, high impervious sites where spill protection is desired and when area for a stormwater pond is unavailable. The construction of the stormwater pond will eliminate the need for any oil/grit separation units.
- Infiltration Trenches/Basins (End-of-Pipe): These SWMF's are most effective in areas with highly pervious soils and large areas.

In accordance with the ORCA requirements for development within the Baxter Creek watershed, Enhanced (Level 1) water quality protection shall be provided by the proposed SWM facility. Based on a total average imperviousness of 75.0%, the required permanent pool volume is provided below.

SWM Pond Permanent Pool Volume Calculation

Volume required for catchment with 75.0% imperviousness:	233.3	m^3/ha
Less 40 m^3/ha of extended detention storage zone:	- 40.0	m^3/ha
Permanent Pool Volume Required:	193.3	m^3/ha

The permanent pool storage volume required for the Pond is 193.3 m^3/ha × 18.52 ha = 3,581 m^3.

In order to maintain a permanent pool of water in the pond and to prevent the mixing of surface water with ground water, the base of the SWM pond will be protected with an appropriate liner. A review of the Geotechnical Investigation Report for the site indicates that the native, undisturbed silty clay, or till with finer-grained gradation (silts and clays) would have a sufficiently low permeability and could substitute for a liner.

The normal water level of the permanent pool for the pond is set at an elevation of 212.50 m. The bottom of the pond is set at an elevation of 210.50 m in the forebay and 211.00 in the main cell, providing a permanent pool depth of 2.00 m and 1.50 m, respectively. The actual permanent pool storage volume provided is approximately 5,097 m^3 which is greater than the minimum required volume (3,581 m^3). The required and provided quality control volume together with the elevation of the normal water level are summarized in **Table 5**.

The forebay has been sized based on MOE design criteria and supporting calculations are provided below. These calculations have been completed based on the more conservative development scenario, which includes the potential future development to the west.

Forebay Sizing Calculations

The proposed forebay is approximately 58 m in length and 25 m in width, on average. The resultant length-to-width ratio is therefore 2.3:1. Using the methodology provided in the *Stormwater Management Planning and Design Manual*, the recommended forebay length based on particulate settling is calculated using the following expression:

$$Dist = \sqrt{\frac{r \cdot Q_p}{V_s}} \qquad [1]$$

where: *Dist* is the forebay length (m)
r is the length-to-width ratio of the forebay (2.3:1 or r = 2.3)
Q_p is the pond's peak discharge (0.026 m^3/s, VO modelling of 25 mm storm)
V_s is the settling velocity (0.0003 m/s for 150 μm particles)

Solving [1] gives:

$$Dist = \sqrt{\frac{2.3 \times 0.026}{0.0003}} = 14.1\ m$$

The recommended forebay length based on flow dispersion calculations is calculated using the following expression:

$$Dist = \frac{8 \cdot Q}{d \cdot V_f} \qquad [2]$$

where: *Dist* is the forebay length (m)
Q is the peak inlet flow (1.535 m^3/s, VO modeling of 5-year storm)
d is the depth of the permanent pool in the forebay (2.00 m)
V_f is the desired velocity in the forebay (0.50 m/s)

Solving [2] gives:

$$Dist = \frac{8 \times 1.535}{2.00 \times 0.50} = 12.3\ m$$

The distance from the headwall to the forebay berm is 58 m. The proposed design therefore satisfies the minimum forebay length recommendations.

The minimum recommended forebay bottom width is calculated as follows, based on the maximum distance from the calculations above:

$$Width = \frac{Dist}{8} = \frac{12.3}{8} = 1.5\ m$$

The design proposes an average forebay bottom width of approximately 12 m, which satisfies this criterion.

5.3.2 Erosion Control

In accordance with the ORCA guidelines, erosion control shall be provided using an extended detention active storage zone sized to capture the runoff resulting from a 25 mm rainfall event and to release the runoff over a period of at least 24 hours. Based on the VO5 modelling of this storm condition (i.e. the 25 mm 4-hour Chicago storm distribution), the estimated runoff volume is 13.88 mm distributed over the 18.52 ha catchment area draining to the SWM pond, for a required erosion control volume of 2,571 m^3.

Based on the design for the SWM pond, the extended detention volume provided is 3,573 m^3 at an elevation of 213.20 m. This exceeds the required erosion control volume of 2,571 m^3. The proposed extended detention depth is 0.70 m, which is less than the maximum recommended extended detention depth of 1.00 m.

The extended detention function of the pond will be controlled with a 140 mm diameter orifice plate (*Orifice #1*) located in the box manhole control structure to achieve the minimum required drawdown time of 24 hours (48 hours is considered preferable).

The drawdown time can be calculated using the following expressions, from the *Stormwater Management Planning and Design Manual*:

$$t_d = \frac{0.66 \cdot C_2 \cdot h_1^{1.5} + 2 \cdot C_3 \cdot h_1^{0.5}}{2.75 \cdot A_o} \qquad [3]$$

where: t_d is the drawdown time (s)
h is the maximum water elevation above the orifice (0.63 m)
A_o is the cross-sectional area of the orifice (0.0155394 m^2)
C_2 is the slope coefficient from area-depth linear regression (1984.3)
C_3 is the intercept from area-depth linear regression (4403.0)

The variable *h* is the maximum water elevation above the centroid of the orifice and is calculated as follows (invert of orifice set at normal water level):

$$h_1 = HWL_{25mm} - \left[NWL + \frac{D}{2}\right] = 213.20 - \left[212.50 + \frac{0.140}{2}\right] = 0.63\ m$$

where: HWL_{25mm} is the high water level for the 25 mm rainfall (213.20 m)
NWL is the normal water level (212.50 m)
D is the diameter of the orifice (0.140 m)

Solving [3] yields:

$$t_d = \frac{0.66 \times (1984.3) \times (0.63)^{1.5} + 2 \times (4403.0) \times (0.63)^{0.5}}{2.75 \times (0.015394)} = 180{,}576s = 50.2\ hrs$$

The orifice size, erosion control release rate, draw down time, extended detention volume and water level are summarized in **Table 5**.

5.3.3 Quantity Control

As per the ORCA and the Township's standards, the SWM facility shall be designed to control the post-development peak flow to pre-development levels for the 2-year through 100-year design storms and to safely convey the greater of the uncontrolled 100-year or Regional flow.

A critical storm analysis was completed to determined which storm distribution (based on the latest Peterborough Airport IDF data for 1971-2006 obtained from Environment Canada) requires the largest storage volume to achieve pre-development target flow rates. Based on the results provided in **Table F.9** (provided in **Appendix "F"**), the 6-hour AES storm was identified as the critical storm requiring the largest storage volume to achieve the 100-year flow control.

The preliminary rating curve is provided in **Table F.5** (provided in **Appendix "F"**), and consists of a box manhole control structure with a 1.20 m wide weir (*Weir #1*) cut into the wall of the box manhole.

Table 4 shows the VO5 modelling results based on the 6-hour AES storm distribution, and **Table 5** shows the SWM pond performance characteristics for each return period event.

The SWM pond has been designed with a total active storage volume of 8,763 m^3 at an elevation of 214.00 m. The expected maximum storage required during 100-year storm conditions is approximately 7,101 m^3. The provided active storage is therefore sufficient.

As shown in **Table 4**, the peak discharge rates are equal to or less than the target release rates. Adequate overcontrol has been provided to account for the uncontrolled drainage areas. Supporting documentation (**Tables F.1-9**) and output from the VO5 modelling is provided in **Appendix "F"**.

Table 4. Summary of Storm Drainage Peak Flows

Return Period	**Existing Peak Flows** (m^3/s)	**Proposed Peak Flow** (m^3/s)
25mm Chicago	-	0.093
2-year	0.240	0.109
5-year	0.472	0.238
10-year	0.661	0.382
25-year	0.932	0.617
50-year	1.154	0.802
100-year	1.391	0.992
Regional	-	1.531

5.3.4 Thermal Mitigation Measures

Mitigation measures shall be incorporated into the SWM pond design to minimize thermal impacts to the receiving watercourse. These measures include a bottom draw pipe and a planting strategy to promote shading along the pond perimeter.

Bottom Draw Pipe

Instead of the common perforated riser configuration, a bottom draw pipe will be implemented for the extended detention component to discharge water from the deepest section of the pond where the water temperature is lowest. This outlet consists of a submerged intake headwall and a bottom draw pipe which discharges via an orifice plate in the quality control structure. Given that this pipe is sized for frequent rainfall events (25 mm storm), it will provide the greatest benefit to the thermal regime of the receiving watercourse.

Planting Strategy

In accordance with the Township and ORCA requirements the SWM facility will be planted to provide a natural appearance and to provide environmental benefits. The landscape plan will specify shade producing species to minimize solar heating of the permanent pool during summer months. The forebay design provides additional pond perimeter where shade producing vegetation can be planted.

5.3.5 SWM Pond Inspection & Maintenance

The stormwater management facility should be inspected periodically to determine the frequency of maintenance activities. As such, maintenance activities will be performed on an as-required basis. During the first two years of operation, it is recommended that the stormwater management facility be inspected following significant storm events to determine if and when maintenance activities are required. Subsequently, inspections should be carried out twice per year. The following items should be considered when inspecting the pond:

- Sediment accumulation to determine cleanout requirements;
- Erosion of side slopes and outfall channel;
- Safety hazards;
- Hydraulic operation of the pond;
- Drawdown time following a rainfall event (extended drawdown time greater than 50 hours may indicate a blocked orifice or intake);
- Condition of terrestrial and aquatic vegetation;
- Trash accumulation near hydraulic structures; and
- Surface sheen indicating possible oil contamination.

Table 5: SWM Facility Performance Summary

Quality Control		
	Protection Level	Level 1 (Enhanced)
	Permanent Pool Required (m^3)	3,581
	Permanent Pool Provided (m^3)	5,097
	Normal Water Level, NWL (m)	**212.50**

Erosion Control		
25mm Chicago	Orifice Size (mm)	140
	Draw Down Time (hrs)	50.2
	Flow In (m^3/s)	1.130
	Flow Out (m^3/s)	0.026
	Storage Used (m^3)	2,307
	Pond W.S. Elevation (m)	**212.97**

Quantity Control (6-hour SCS)		
2-year	Flow in (m^3/s)	1.082
	Flow Out (m^3/s)	0.054
	Storage Used (m^3)	3,763
	Pond W.S. Elevation (m)	**213.23**
5-year	Flow in (m^3/s)	1.535
	Flow Out (m^3/s)	0.211
	Storage Used (m^3)	4,739
	Pond W.S. Elevation (m)	**213.39**
10-year	Flow in (m^3/s)	1.845
	Flow Out (m^3/s)	0.345
	Storage Used (m^3)	5,298
	Pond W.S. Elevation (m)	**213.48**
25-year	Flow in (m^3/s)	2.282
	Flow Out (m^3/s)	0.544
	Storage Used (m^3)	6,023
	Pond W.S. Elevation (m)	**213.60**
50-year	Flow in (m^3/s)	2.603
	Flow Out (m^3/s)	0.711
	Storage Used (m^3)	6,573
	Pond W.S. Elevation (m)	**213.68**
100-year	Flow in (m^3/s)	2.931
	Flow Out (m^3/s)	0.882
	Storage Used (m^3)	7,101
	Pond W.S. Elevation (m)	**213.76**
Regional Storm (Timmins)	Flow in (m^3/s)	1.807
	Flow Out (m^3/s)	1.352
	Storage Used (m^3)	8,416
	Pond W.S. Elevation (m)	**213.95**

5.4 Site Water Balance

In accordance with the requirements of the ORCA, an annual site water balance assessment was completed by GHD for the subject development area to determine the overall infiltration deficit under proposed conditions and to design LID measures as part of an overall mitigation strategy to maintain pre-development infiltration volumes (*Updated Geotechnical Investigation Report; Proposed Residential and Commercial Development – Part Lot 13, Concession 5; Millbrook, Ontario; GHD*, March 2022). Excerpts from the *Geotechnical Investigation Report* regarding the water balance analysis (*Section 6.1.1 – Hydrogeology – Updated Water Balance Evaluation*) are included in **Appendix H**. The findings of the *Hydrogeological Assessment* water balance analysis are summarized below.

Pre-Development Infiltration Volume

For the pre-development condition, a total estimated infiltration volume of 43,579 m^3/year was determined. Refer to **Appendix H** for the detailed water balance calculations.

Post-Development Infiltration Volume (Unmitigated)

For the post-development condition, without infiltration enhancements, a total estimated infiltration of 31,665 m^3/year was determined. This corresponds to a total infiltration deficit of 11,914 m^3/year (43,579 m^3/year – 31,665 m^3/year = 11,914 m^3/year), a 27.3% decrease in annual infiltration compared to the pre-development condition. Refer to **Appendix H** for the detailed water balance calculations.

The decrease in annual infiltration indicates the need for Low Impact Development (LID) strategies be implemented in order to maintain pre-development infiltration rates. The LID measures that are proposed in order to meet the infiltration deficit include roof downspout disconnections and infiltration trenches. The design of these LID measures are presented below.

Proposed Infiltration BMP – Roof Downspout Disconnection

As per *Section 4.3 – Roof Downspout Disconnection* of the *Low Impact Development Stormwater Management Planning and Design Guide* (Credit Valley Conservation, Toronto and Region Conservation, 2010), roof downspout disconnection is a common practice to achieve water balance benefits. In order to achieve the required infiltration, roof downspout disconnections must meet the following design criteria:

- *Available Space: Simple downspout disconnection requires a minimum flow path length across the pervious area (at least 5 metres) and suitable soil conditions. If the flow path length is less than 5 metres and soils are hydrologic soil group (HSG) C or D, roof downspouts should be directed to another LID practice such as a rainwater harvesting system, soakaway, swale, bioretention area or perforated pipe system.*

  In order to meet this design criteria, houses will be required to direct roof downspouts to a discharge location a minimum distance of 5 m from the road right-of-way or rear-lot catchbasin. For runoff draining the back yard of the lot (split-drainage lots), a minimum flow length of 5 m will be achieved. For runoff draining to the front, roof downspouts will be installed as required in order to ensure a minimum flow length of 5 m. Where required, roof downspouts shall discharge to the side of the house into a side swale in order to meet the flow length requirement.

According to the hydrologic modelling completed for this site, the soils generally fall into hydrologic soil group B, which is considered to be acceptable for roof downspout disconnections.

- *Site Topography: Disconnected downspouts should discharge to a gradual slope that conveys runoff away from the building. The slope should be between 1% and 5%. Grading should discourage flow from reconnecting with adjacent impervious surfaces.*

In order to meet this design criteria, lots will generally be graded at 2%. Grading will be designed to discourage flow from reconnecting with adjacent impervious surfaces for a minimum of 5 m for the downspout discharge location.

- *Soils: If the infiltration rate of soils in the pervious area is less than 15 mm/hr (i.e., hydraulic conductivity less than 1x10-6 cm/s), as determined from measurements (see Appendix C for acceptable methods), they should be tilled to a depth of 300 mm and amended with compost to achieve an organic content in the range of 8 to 15% by weight or 30 to 40% by volume.*

As per *Section 4.2.5 – Hydraulic Conductivity* (*Updated Geotechnical Investigation Report; Proposed Residential and Commercial Development – Part Lot 13, Concession 5; Millbrook, Ontario; GHD*, March 2022) the infiltration rate varies between 30 and 75 mm/hr.

Assuming the lowest measured infiltration rate of 30 mm/hr to be conservative, the infiltration rate of the soils are considered acceptable. However, it is noted that significant compaction of soils can occur during home building and that topsoil depths are typically not monitored. In order to address this issue, a minimum topsoil depth of 150 mm is to be specified for pervious lot areas, and topsoil is to only be placed after the completion of construction works to minimize the extent of potential compaction within the soil profile. Topsoil shall be inspected for compaction prior to sodding and scarification will be provided as required.

- *Drainage Area:* For simple downspout disconnection the roof drainage area should not be greater than 100 square metres.

> Although most roof areas will exceed 100 m^2 (excluding townhouse lots), the roof drainage area to any single roof downspout will be less than 100 m^2 (split-draining, peaked roofs). However, when performing the infiltration calculations, it is assumed that no more than 100 m^2 of roof area per lot contributes to infiltration, to be conservative.

It is assumed that roof area for single detached residential lots is 45% of the total lot, and that all lots have an average minimum depth of 30 m. The average maximum roof area for each lot is therefore 144 m^2, 184 m^2, and 214 m^2, for the proposed 10.70 m, 13.7 m, and 15.90 m frontage single detached residential lots, respectively. Based on the design criteria, it is assumed that only 100 m^2 of roof area per lot can be applied towards infiltration mitigation. As per the draft plan, there are a total of 128 single detached residential lots, each with an applicable roof area of 100 m^2, for a total applicable roof area of 12,800 m^2 (1.28 ha).

It is assumed that the roof area for townhouse lots is 30% of the total lot. All townhouse lots will have 7.62 m of frontage and a depth of 30 m. The maximum roof area is therefore 68 m^2. It is assumed that the townhouse roofs will be split draining, and that roof drainage to the front will not meet the minimum 5 m flow path criteria. Because of this, only half of the roof area, or 34 m^2, is applicable towards infiltration mitigation. As per the draft plan, there are 48 townhouse lots, each with an applicable roof area of 34 m^2, for a total applicable roof area of 1,632 m^2 (0.16 ha).

Section 4.3 – Roof Downspout Disconnection of the *Low Impact Development Stormwater Management Planning and Design Guide* indicates that "a conservative runoff reduction rate estimate for roof downspout disconnection is 25% for hydrologic soil group (HSG) C and D soils and 50% for HSG A and B soils. These values apply to disconnections that meet the physical suitability and constraints criteria outlined in this section." As demonstrated above, the roof downspout disconnection design and applicable roof area meet the outlined design requirements and conditions for hydrologic soil group A and B soils. A runoff reduction of 50%, and the corresponding infiltration volume of 50%, is therefore considered acceptable.

Based on the *Geotechnical Investigation Report*, the total annual site precipitation is 252,823 m^3/yr for a total site area of 29.57 ha. This corresponds to a per hectare precipitation rate of 8,550 m^3/yr/ha. The total roof area applicable to roof downspout disconnection infiltration is 1.44 ha (1.28 ha + 0.16 ha = 1.44 ha). The precipitation to applicable roof areas is therefore 12,312 m^3/yr (1.44 ha x 8,550 m^3/yr/ha = 12,312 m^3/yr). Based on an evaporation rate of 20% of incident precipitation for impervious areas, as per the *Geotechnical Investigation Report*, the surplus for the applicable roof area is therefore 9,849 m^3/yr (12,312 m^3/yr x (1 – 0.2) = 9,849 m^3/yr).

Assuming an infiltration rate of 50%, the infiltration volume due to roof downspout disconnections is therefore 4,924 m^3/yr (9,849 m^3/yr x 0.50 = 4,924 m^3/yr).

Proposed Infiltration BMP – Infiltration Facilities

Roof downspout disconnection will account for an annual infiltration volume of 4,924 m^3/yr. The Phase 1 post-development infiltration deficit without infiltration measures is 11,914

m^3/yr. The remaining deficit of 6,990 m^3/yr (11,914 m^3/yr – 4,924 m^3/yr = 6,990 m^3/yr) will be infiltrated through the installation of infiltration trenches.

The areas directed to the infiltration trenches will include the rear yard areas indicated on **Figure 8**. The infiltration trenches have been designed based on a soil infiltration rate of 30 mm/hr, a drawdown time of 48 hours, a void ratio of 0.4, and a depth of 1.44 m. The width of each infiltration trench will vary, detailed calculations will be provided at detailed design.

Tables F.10 provide details on the location and sizing of the infiltration facilities. With the implementation of the proposed infiltration facilities, an additional infiltration capacity of 7,309 m^3/yr is achieved.

Post-Development Infiltration Volume (Mitigated)

With the inclusion of the roof downspout disconnection and infiltration facility LID measures, the total post-development infiltration volume will be 43,898 m^3/yr (31,665 m^3/yr + 4,924 m^3/yr + 7,309 m^3/yr = 43,898 m^3/yr), which is 100.7% of the annual pre-development infiltration volume (43,579 m^3/yr). The pre-development infiltration volumes will therefore be maintained under post-development conditions.

6.0 VEHICULAR & PEDESTRIAN ACCESS

The layout of the proposed subdivision has been developed with consideration for efficient and safe access and circulation of both vehicular and pedestrian traffic.

6.1 Municipal Roads

The subject site has a frontage on both Fallis Line and on County Road 10. Fallis Line, east of County Road 10 is a 20.0m wide Township road allowance which is currently unopened and untravelled. County Road 10 is an arterial road which is under the jurisdiction of the County of Peterborough. This road consists of a rural cross section having two lanes with partially paved shoulders and road side ditches.

The vehicular access to the subdivision will be facilitated by two connections, being the intersections of Street "A" with the proposed Fallis Line extension. Pedestrian access will also be provided to Nina Court at the south limit of the subdivision through a walkway and servicing block.

The streets in the subdivision will be in the form of 20.0m road allowances with Street "A" having 10.0m wide pavement and the rest of the streets will have 8.5m pavement width. The proposed roads will have urban sections having pavement crowned with a 2.0% cross fall and edged with concrete curb and gutter. The longitudinal slope of the road will generally be 0.50% with some length of road ranging up to 6.5% slope. The standard road cross sections are included in **Appendix "G"**.

Based on the recommendations contained in the Geotechnical Investigation Report for the site, the recommended minimum pavement structure for the proposed roads is as follows:

Municipal Roads

Material	**Compacted Depth**
HL3 Surface Course Asphalt	40mm
HL8 Base Course Asphalt	50mm
Granular “A”	150mm
Granular “B”	450mm

6.2 Driveways

Each dwelling will have an attached garage and driveway. The recommended pavement structure for the residential driveways is as follows:

Driveways

Material	**Compacted Depth**
HL3 Surface Course Asphalt	40mm
Granular “A”	150mm

The residential driveways will be either single or double car width. The slope of driveways is to be within the range of 1.0% to 7.0% in accordance with Township criteria.

6.3 Sidewalks, Walkways & Trails

Internal pedestrian access will be provided by standard 1.5m wide concrete sidewalks to safely guide residents through the subdivision for access to the proposed commercial block and the existing community centre. Sidewalks will be generally constructed on one side of each road. The sidewalks will have tactile warning plates at all curb ramps in accordance with Provincial accessibility standards.

Details of the standard sidewalk, curb ramp and tactile warning plate are included in **Appendix “G”**.

7.0 GRADING

As is typical will all subdivision, earthmoving is required, to varying degrees, in order to achieve the municipal design criteria and accommodate the development form.

7.1 Grading Criteria

The subject site is to be graded in accordance with the Township grading criterion which dictates that road grades are to range from 0.5% to 8.0% and that sodded yard areas are to range from 2.0% to 5.0%. For large grade differentials, a maximum slope 3H : 1V can be used for sodded embankments. In areas where space is limited, retaining walls can be utilized to accommodate grade differentials, however, their use should be minimized.

Given the relatively steep site, a road grade of 6.5% will be utilized for a length of Street "A".

7.2 Preliminary Design

Based on the topographic survey, the proposed subdivision configuration and the Township's criteria, a preliminary grading design has been prepared. The preliminary grading design, considered the following factors:

- Achieve the Township's lot grading criteria.
- Meet the Township's vertical road design parameters.
- Minimize the requirement for retaining walls.
- Match existing grades along the adjacent properties and road allowances.
- Grading along existing road allowances is to have consideration for their future urbanization and grades are to be established to accommodate future boulevard slopes in the range of 2 to 4%.
- Provide an overland flow route to direct drainage to a safe outlet.
- Provide sufficient cover over the sanitary sewer.

An analysis of the earthworks will be conducted using digital terrain modelling software at the detailed design stage to optimize the cut and fill volumes. Based on the **Preliminary Servicing & Grading Plan**, given that the site is relatively steep, many basement walk-out type lots will be utilized and some areas will require 3:1 slopes and retaining walls to accommodate the grade differential. With these measures we anticipate that it will be feasible to achieve the municipal grading design standards.

7.3 Permitting

A review of the Regulation Mapping indicates that the subject site is located within an area that is regulated by the ORCA. A grading permit is therefore required from their office under Ontario Regulation 166/06 prior to commencing topsoil stripping and earthworks. The permit application should be submitted in conjunction with the detailed design at the subdivision engineering stage.

8.0 EROSION & SEDIMENT CONTROL DURING CONSTRUCTION

Construction activity, especially operations involving the handling of earthen material, dramatically increases the availability of particulate matter for erosion and transport by surface drainage. In order to mitigate the adverse environmental impacts caused by the release of silt-laden stormwater runoff into receiving watercourses, measures for erosion and sediment control are required for construction sites. This is an extremely important component of land development that plays a large role in the protection of downstream watercourses and aquatic habitat.

The impact of construction on the environment is recognized by the Greater Golden Horseshoe Area Conservation Authorities. In December 2006 they released their document titled Erosion & Sediment Control Guidelines for Urban Construction (ESC Guideline). This document provides guidance for the preparation of effective erosion and sediment control plans.

Control measures must be selected that are appropriate for the erosion potential of the site and it is important that they be implemented and modified on a staged basis to reflect the site activities. Furthermore, their effectiveness decreases with sediment loading and therefore inspection and maintenance is required. The selection, implementation, inspection and maintenance of the control features are summarized as follows:

8.1 Control Measures

On relatively large sites, measures for erosion and sediment control typically include the use of sediment control basins, silt fencing, a mud mat and sediment traps. The following is a description of the sediment controls to be implemented on the subject site:

- **Temporary Sediment Control Basins** are commonly used to clarify silt-laden stormwater runoff by promoting sedimentation of the suspended particles in the runoff through long detention times. The proposed SWM pond will be utilized as temporary sediment control basins during construction. The basin is to be sized in accordance with the ESC Guideline based on a required storage volume of 250 m^3 per hectare of disturbed area (125 m^3/ha of permanent pool and 125 m^3/ha of active storage). The basin's outlet is to have a Hickenbottom riser and a minimum 75mm diameter orifice plate sized to provide a drawdown time in the order of 48 hours.
- **Silt Fences** are to be installed adjacent to all property limits subject to drainage from the development area prior to topsoil stripping and in other locations, such as at the bases of topsoil stockpiles. It is recommended that earthworks not extend immediately adjacent to the silt fence and instead 1m to 2m vegetated buffer be maintained for additional protection. The silt fences are to be constructed with 150 x 150mm wire farm fence fabric to properly support the geotextile. Heavy duty silt fence is recommended to be installed adjacent the South Wetland consisting of two rows of fence with a row of staked straw bales between.
- **Mud Mat** is to be installed at the construction entrance prior to commencing earthworks to minimize the tracking of mud onto municipal roads.
- **Sediment Traps** are to be installed at all catchbasin locations once the storm sewer system has been constructed to prevent silt laden runoff from entering.
- **Rock Check Dams** are to be constructed in swales and ditches to reduce velocities and trap sediment.

A set of Erosion and Sediment Control Plans are to be prepared at the detailed engineering design stage to reflect the various construction stages. Details of typical erosion and sediment control measures are included in **Appendix "I"**.

8.2 Construction Sequencing

The following is a summary of the scheduling of construction activities and the related implementation of sediment controls:

Stage 1 – Subdivision Earthworks

1. Construct mud mat for temporary construction access.
2. Install primary silt fencing around the limits of grading and secondary silt fencing along the south limit of the work area adjacent the existing wetland.
3. Install temporary swales and rock check dams.
4. Excavate and construct the temporary sediment basins including installation of hickenbottom drain and spillway and connect to temporary swales.
5. Strip any remaining topsoil, stockpile where indicated and install silt fence around the perimeter.
6. Rough grade the site by placing cut material in fill areas and spreading and compacting of imported fill. Maintain the mud mat to minimize the tracking of silt onto the municipal road and provide street sweeping as necessary.

Stage 2 – Subdivision Servicing & Road Construction

1. Install underground servicing, covering the end of the pipe at the end of each work day to ensure that silt does not enter the storm sewer.
2. Construct roads, install sediment controls on catchbasins and install temporary hickenbottom drains at low point of lot blocks.

Stage 3 – House Construction

1. Construct houses and maintain all sediment controls including regular street sweeping and catchbasin cleaning.
2. Stabilize all lot surfaces as soon as possible after completion of the houses.
3. Remove silt fencing on a phased basis as areas are stabilized.

8.3 ESC Inspection & Maintenance

In order to ensure that the erosion and sediment control measures operate effectively, they are to be regularly monitored and they will require periodic cleaning (e.g., removal of accumulated silt), maintenance and/or re-construction.

Inspections of all of the erosion and sediment controls on the construction site should be undertaken with the following frequency:

- On a weekly basis
- After every rainfall event
- After significant snow melt events
- Prior to forecasted rainfall events

If damaged control measures are found they should be repaired and/or replaced within 48 hours. Site inspection staff and construction managers should refer to the Erosion and Sediment Control Inspection Guide (2008) prepared by the Greater Golden Horseshoe Area Conservation Authorities. This Inspection Guide provides information related to the inspection reporting, problem response and proper installation techniques.

9.0 UTILITIES

While some external upgrades may be necessary by the utility providers, it is anticipated that utilities such as electrical (Hydro One Networks Inc.), natural gas (Enbridge Gas Distribution Inc.) and telecommunications (Nexicom Inc.) will be available to service the subject development. As per standard practice in subdivisions, utilities will be installed underground. Co-ordination with the Hydro One and the various utility companies will be undertaken at the detailed engineering design stage to determine appropriate locations for pedestals, transformers and street lights.

It is recommended that the utility installation be in the form of a joint trench. The process of joint trenching allows all of the utility companies to co-ordinate the placement of their lines in a common trench excavated by a single utility contractor. Joint trenching maximizes the efficiency of the available area in the utility corridor and provides for a safe installation.

In accordance with the Township requirements, street lights will be LED. The street lights will have black octagonal poles with black post top luminaires similar to those installed in the Phase 1 subdivision.

A copy of a typical joint trench detail is included in **Appendix "J"** together with a detail of the street light.

10.0 SUMMARY

Based on the analysis contained herein, the proposed residential subdivision can be adequately serviced with full municipal services (watermain, wastewater and storm) in accordance with the standards of the Township of Cavan Monaghan, the County of Peterborough and the Otonabee Region Conservation Authority design criteria and consists of the following:

Water

- The community of Millbrook is currently serviced by a well based water system with a treatment plant and water storage tank. A 300mm diameter trunk watermain was constructed northerly along County Road 10 with a water tower and booster station located on the site of the municipal offices. A 250mm diameter watermain was constructed on Fallis Line to the east side of County Road 10 which will service the subject development. A connection will also be made to the 150mm diameter watermain on Nina Court, to the south of the subject development, to complete a loop.
- A local water distribution system will be constructed along the roads to provide domestic supply and fire protection for the proposed dwellings. This local system will have pipe diameters ranging in size from 150mm to 250mm. Based on the Ontario Building Code (OBC 2012) requirements, the water service connections for the individual townhouse units are to be 25mm diameter.

Waste Water

- A 525mm diameter trunk sanitary sewer has been constructed on Nina Court to the south limit of the subject site. This trunk sewer will be extended northerly through the subject lands to service future development north of Fallis Line and on the east side of County Road 10. An analysis of the downstream sanitary sewer was conducted which confirmed that there is sufficient capacity in the sewer from the subject site to the WWTP.
- A local sanitary sewer system will be constructed along the proposed roads to provide service to the dwellings. In accordance with Township standards, the dwellings will be serviced with individual sanitary connections.

Storm Drainage

- The subject site is located in the Baxter Creek subwatershed. The Baxter Creek drains to the Otonabee River which discharges to Rice Lake.
- In accordance with Township criteria, the subject site will be serviced by minor system comprised of a municipal storm sewer sized for the 5-year storm event. This storm sewer will outlet to the proposed SWM facility.
- The major system will be comprised of an overland flow route which will convey runoff from rainfall events in excess of the capacity of the municipal storm sewer to the SWM facility.
- The Regulatory floodplain of the Baxter Creek tributary is contained entirely within the valley lands and therefore the proposed residential lots and the stormwater management pond are outside the Regulatory floodplain.

Stormwater Management

- A stormwater management facility will be constructed to service the subject property, as well as the potential future development to the west. This facility has been designed as a wet pond to provide Enhanced (Level 1) water quality treatment, extended detention for erosion control and flood control using the calculated pre-development flow targets up to and including the 100-year storm event. The wet pond consists of a sediment forebay and a main cell separated by a forebay berm.
- Thermal mitigation measures are to be incorporated in the design of the pond including bottom draw pipe and a planting strategy to provide shading around the pond perimeter.
- A site water balance assessment has been undertaken to ensure that pre-development infiltration volumes are maintained. Based on the analysis it was determined that mitigation measures are required in the form of infiltration trenches and roof top disconnections.

Vehicular & Pedestrian Access

- Vehicular access to the subject site will be provided by a road connections to the proposed Fallis Line extension.
- The proposed local roads will be constructed to urban standards having 20m wide road allowances. Street “A” will have a 10.0m pavement width and the rest of the streets will have 8.5m pavement width.
- Pedestrian access will be provided by 1.5m wide concrete sidewalks which are to be generally located on one side of each road.

Grading

- As is typical with large subdivision projects, earthmoving will be required to achieve the proposed subdivision grading necessary to meet the criteria of the Township. A detailed analysis of the earthworks will be conducted at the detailed design stage to optimize the cut and fill volumes.
- Given that the site is relatively steep, many basement walk-out type lots will be utilized and some areas will require 3:1 slopes and retaining walls to accommodate the grade differential. With these measures we anticipate that it will be feasible to achieve the municipal grading design standards.
- Since the subject site is located in an area which regulated by the ORCA, a permit will be required from their office prior to commencing earthworks.

Erosion & Sediment Control During Construction

- Erosion and sediment control (ESC) measures are to be implemented during construction to prevent silt laden runoff downstream in accordance with the Erosion & Sediment Control Guidelines for Urban Construction (December 2006). The ESC plans are to be prepared at the detailed engineering design stage and are to reflect the various construction stages.

Utilities

- Similar to the Phase 1 subdivision, utility servicing will include an underground joint utility trench for electrical, natural gas and telecommunications. Street lighting will be LED and will be comprised of black octagonal poles with black post top luminaires.

Subdivision Engineering Design

- Detailed design for the proposed development is to be prepared at the subdivision engineering stage. This detailed design is to include servicing and grading plans as well as a stormwater management report based on the criteria established in this Functional Servicing Report.

11.0 REFERENCES & BIBLIOGRAPHY

- Township of Cavan Monaghan, **Municipal Servicing Standards**, April 2017.
- Ontario Ministry of Environment, **Stormwater Management Planning & Design Manual,** Mar 2003.
- Ontario Ministry of Transportation, **Drainage Management Manual**, 1997.
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- GHD Inc., **Updated Geotechnical Investigation Report**, March 11, 2022.

Respectfully Submitted,

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obeaudin@valdor-engineering.com

Phoebe Yung, B.Eng.
Water Resource Analyst

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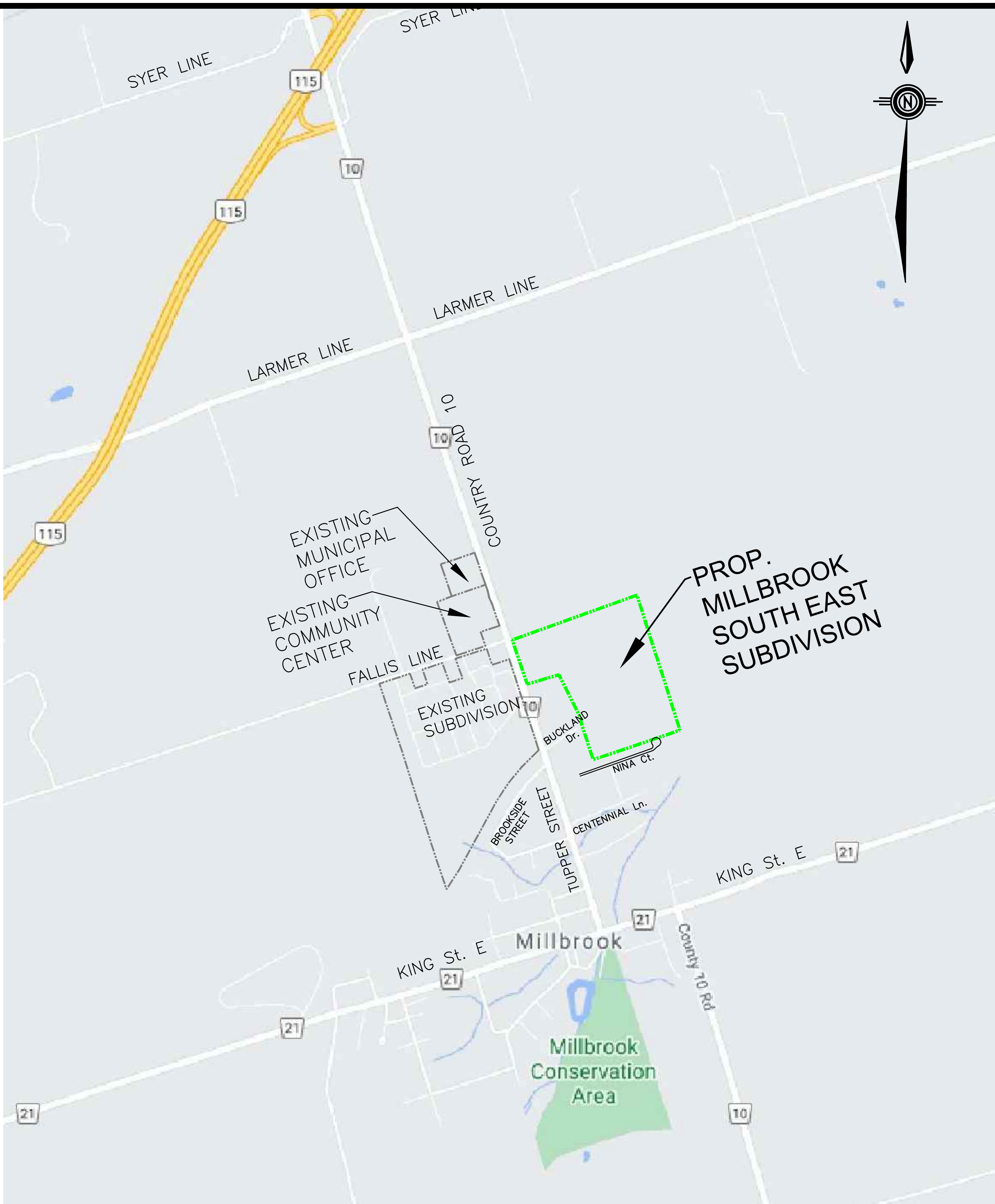
pyung@valdor-engineering.com



MILLBROOK SOUTH EAST SUBDIVISION

LOCATION MAP

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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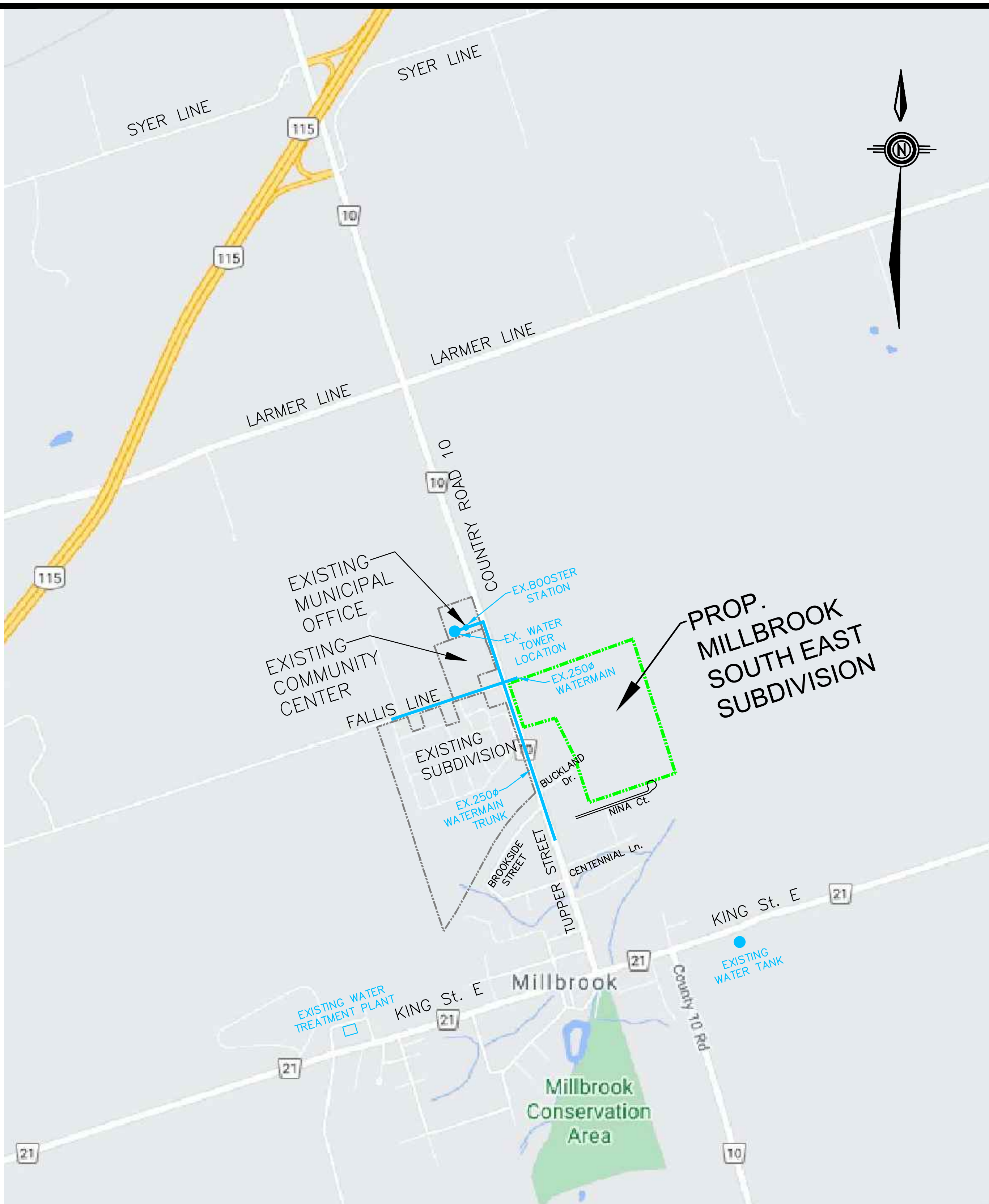
E-MAIL: info@valdor-engineering.com
www.valdor-engineering.com

SCALE	N.TS.	PROJECT	19121	FIGURE 1
DATE	March, 2022	DRAWN BY	V.L.	



MILLBROOK SOUTH EAST SUBDIVISION

WATER SERVICING EXTERNAL

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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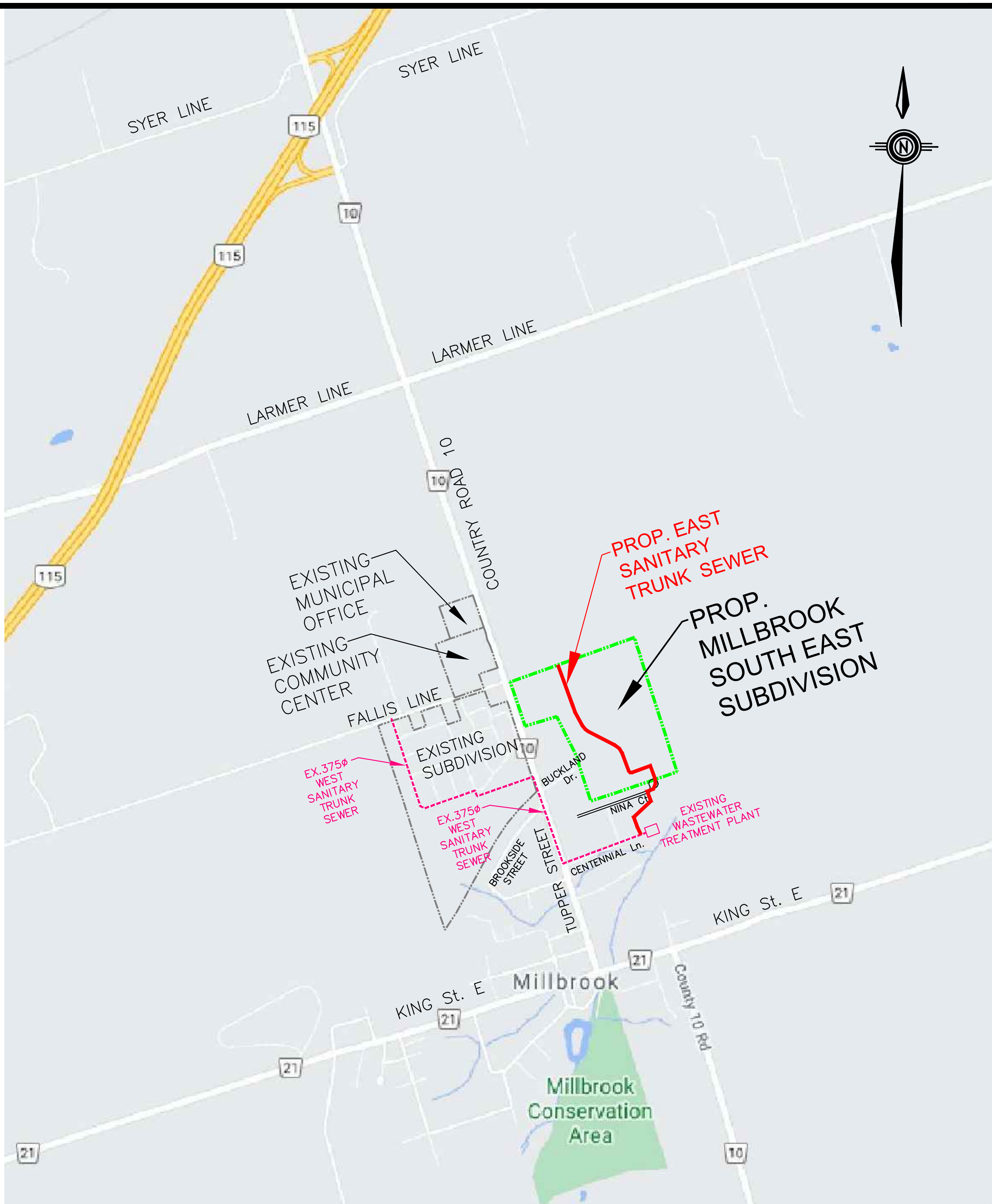
SCALE	N.TS.	PROJECT	19121	FIGURE 2
DATE	March, 2022	DRAWN BY	V.L.	



MILLBROOK SOUTH EAST SUBDIVISION

WASTEWATER SERVICING EXTERNAL

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
741 ROWNTREE DAIRY ROAD, SUITE 2, WOODBRIDGE, ONTARIO, L4L 5T9
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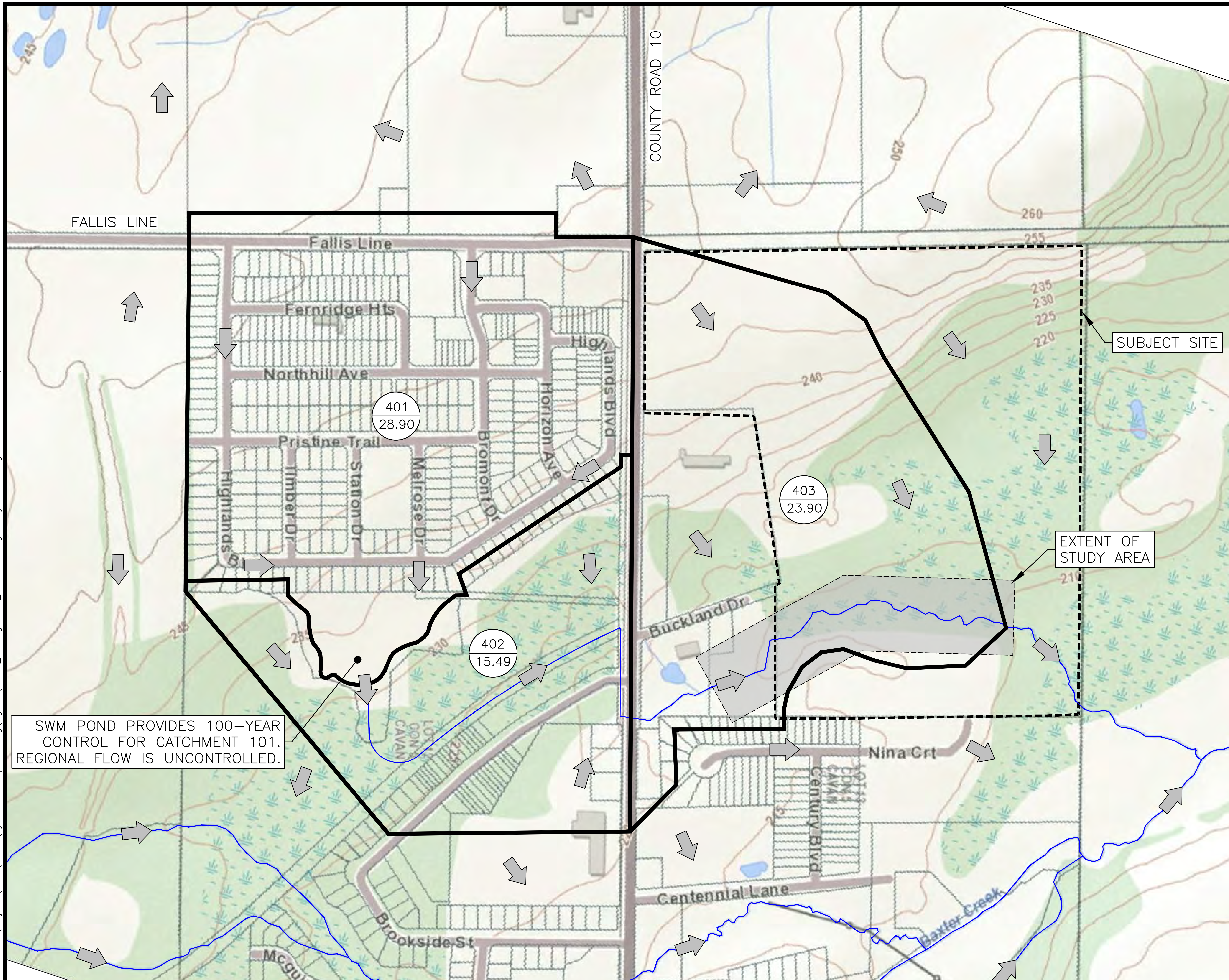
E-MAIL: info@valdor-engineering.com
www.valdor-engineering.com

SCALE	N.TS.	PROJECT	19121	FIGURE 3
DATE	March, 2022	DRAWN BY	V.L.	



LEGEND

401 / 28.90 — CATCHMENT ID / AREA (HA)

DRAINAGE BOUNDARY

WATERCOURSE

OVERLAND FLOW

PROJECT
MILLBROOK SOUTH EAST SUBDIVISION

TITLE
FLOODPLAIN MAPPING PRE-DEVELOPMENT DRAINAGE PLAN

VALDOR ENGINEERING INC.
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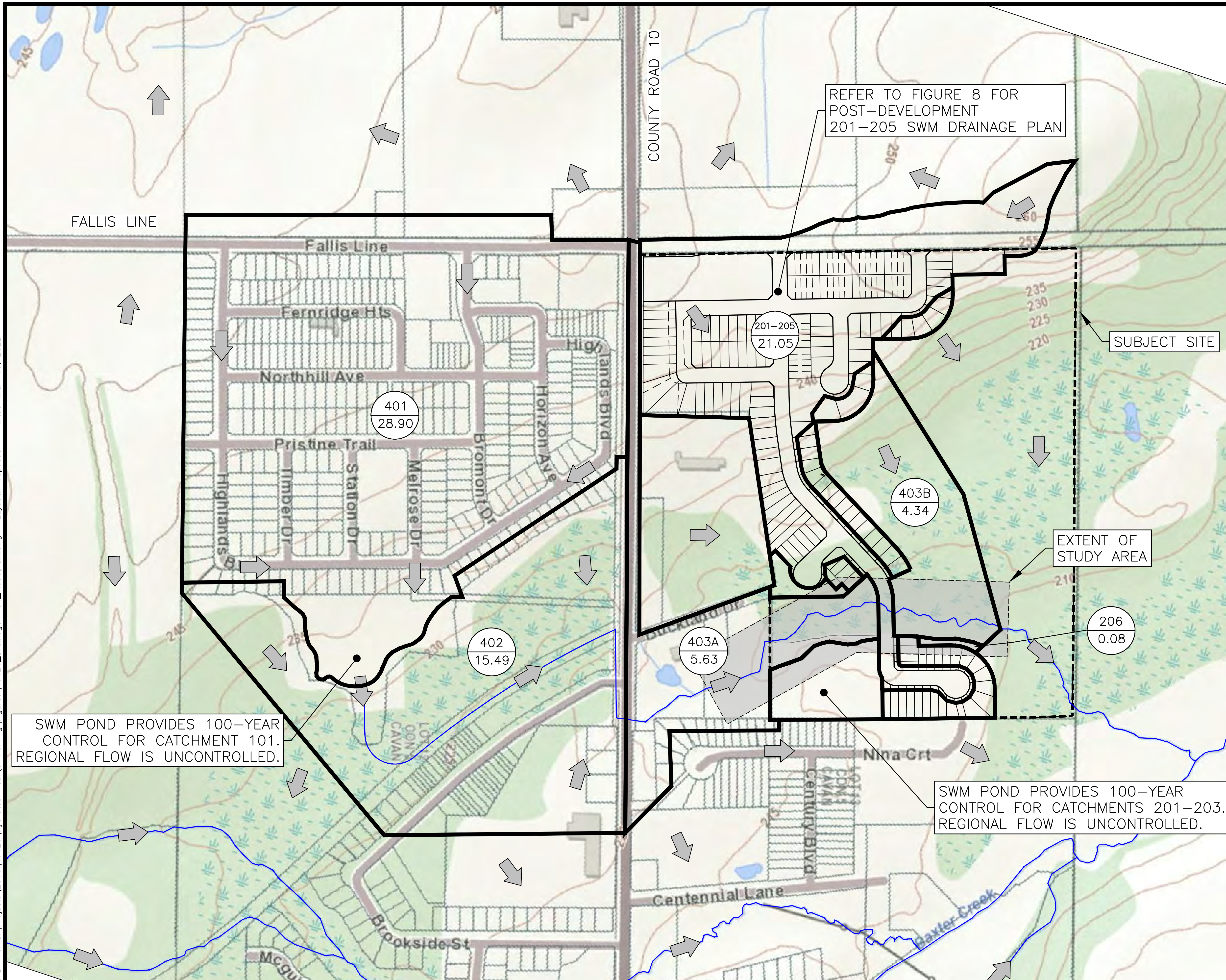
E-MAIL: info@valdor-engineering.com
www.valdor-engineering.com

PREPARED BY O.B.	CKD. BY O.B.
SCALE 1:5000	DATE MAR. 2022
PROJECT 19121	FIGURE 4A

ACAD File: S:\Projects\2019\19121\Hydrotechnical\0-Working\Figures\19121_Drainage Plan_Floodplain.dwg Layout: Existing Printed: Mar. 07, 2022



LEGEND

401	CATCHMENT ID
28.90	AREA (HA)
	DRAINAGE BOUNDARY
	WATERCOURSE
	OVERLAND FLOW

PROJECT	MILLBROOK SOUTH EAST SUBDIVISION
TITLE	FLOODPLAIN MAPPING POST-DEVELOPMENT DRAINAGE PLAN

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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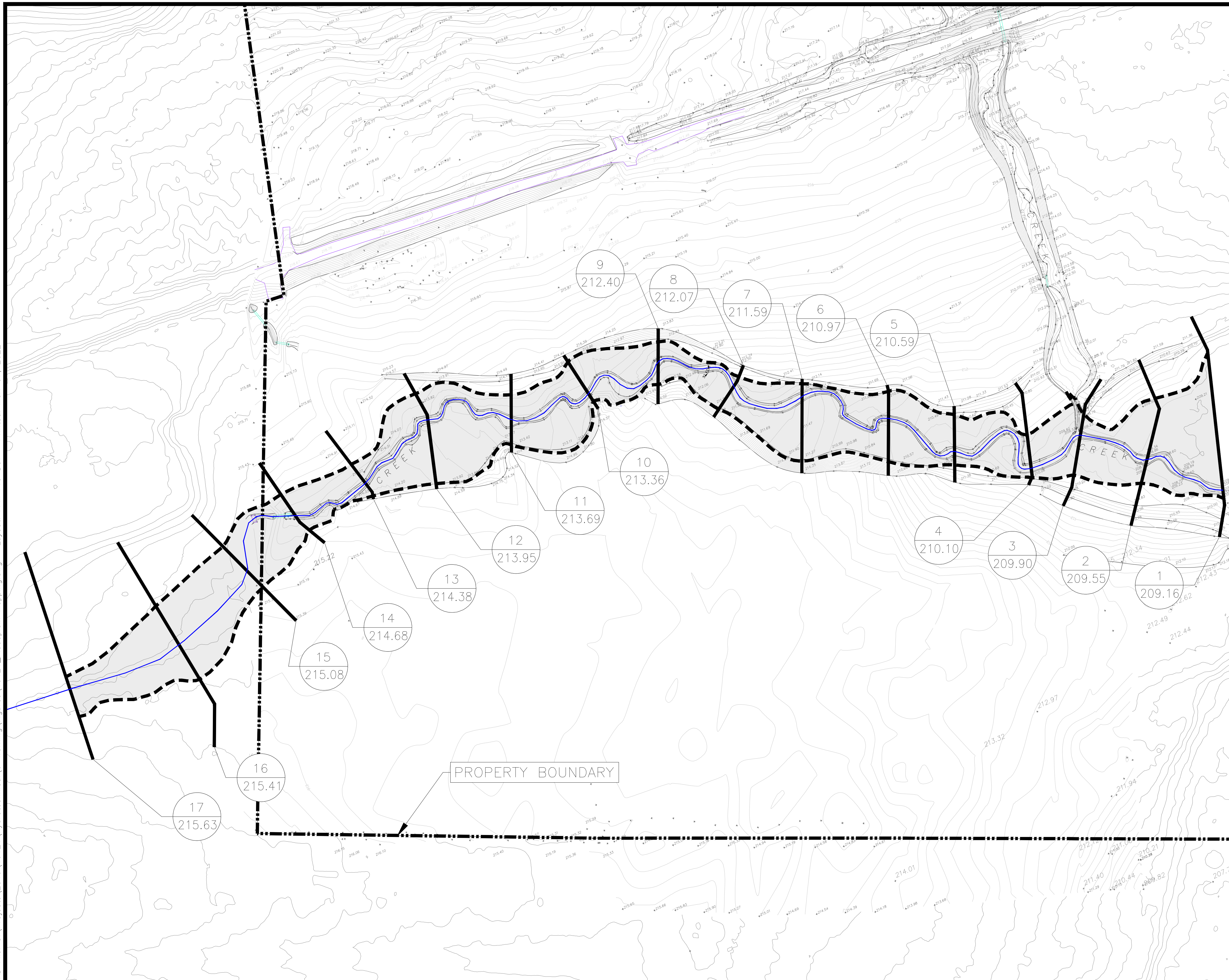
PREPARED BY	P.Y.	CKD. BY	B.C.
SCALE	1:5000	DATE	MAR. 2022
PROJECT	19121		FIGURE 4B

ACAD File: S:\Projects\2019\19121\Hydrotechnical\0-Working\Figures\19121_Drainage Plan_Floodplain.dwg Layout: Proposed Printed: Mar. 10, 2022



LEGEND

- EXISTING REGIONAL FLOODLINE
- PROPERTY BOUNDARY
- HEC-RAS CROSS-SECTION
- 1 / 209.17 — HEC-RAS CROSS-SECTION ID; REGIONAL WATER SURFACE ELEVATION (m)

PROJECT	**MILLBROOK SOUTH EAST SUBDIVISION**
TITLE	**FLOODPLAIN MAPPING PRE-DEVELOPMENT**

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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PREPARED BY	O.B.	CKD. BY	O.B.
SCALE	1:1250	DATE	MAR. 2022
PROJECT	19121		FIGURE 5

LEGEND

- PROPOSED REGIONAL FLOODLINE
- PROPERTY BOUNDARY
- HEC-RAS CROSS-SECTION
- HEC-RAS CROSS-SECTION ID (1)
- REGIONAL WATER SURFACE ELEVATION (m) (209.18)

PROJECT	MILLBROOK SOUTH EAST SUBDIVISION
TITLE	FLOODPLAIN MAPPING POST-DEVELOPMENT

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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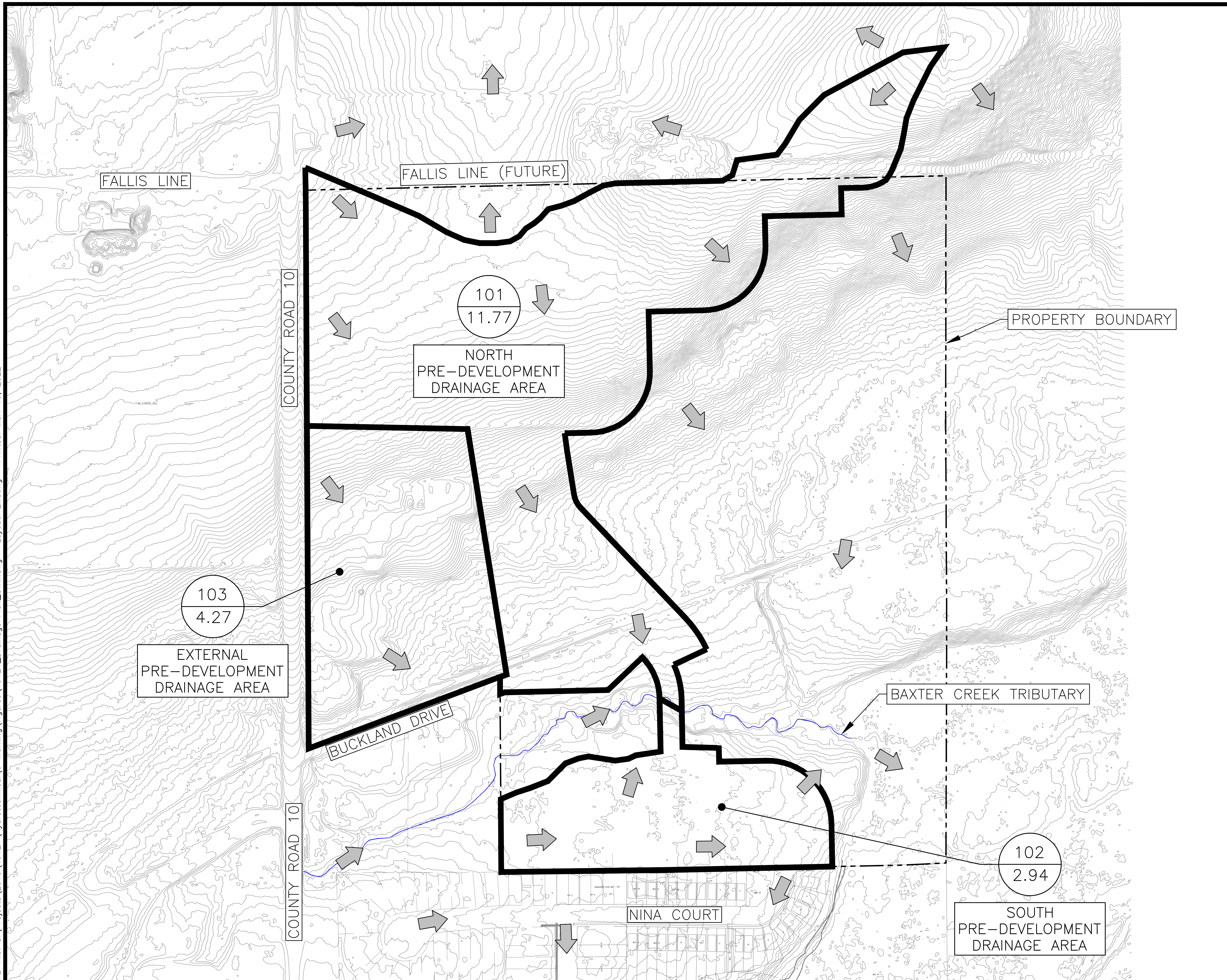
PREPARED BY	P.Y.	CKD. BY	B.C.
SCALE	1:1250	DATE	APR. 2022
PROJECT	19121		FIGURE 6

ACAD File: S:\Projects\2019\19121\Hydrotechnical\0-Working\Figures\19121_Floodplain Mapping.dwg Layout: FPM Post-Dev Printed: Apr. 18, 2022



ACAD File: S:\Projects\2019\19121\Hydrotechnical\0-Working\Figures\19121_Drainage Plan_SWM.dwg Layout: Existing Printed: Mar. 10, 2022

PROJECT	**MILLBROOK SOUTH EAST SUBDIVISION**
TITLE	**SWM DRAINAGE PLAN PRE-DEVELOPMENT**

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
571 Chrislea Road, Unit 4, 2nd Floor Vaughan, Ontario, L4L 8A2
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E-MAIL: info@valdor-engineering.com
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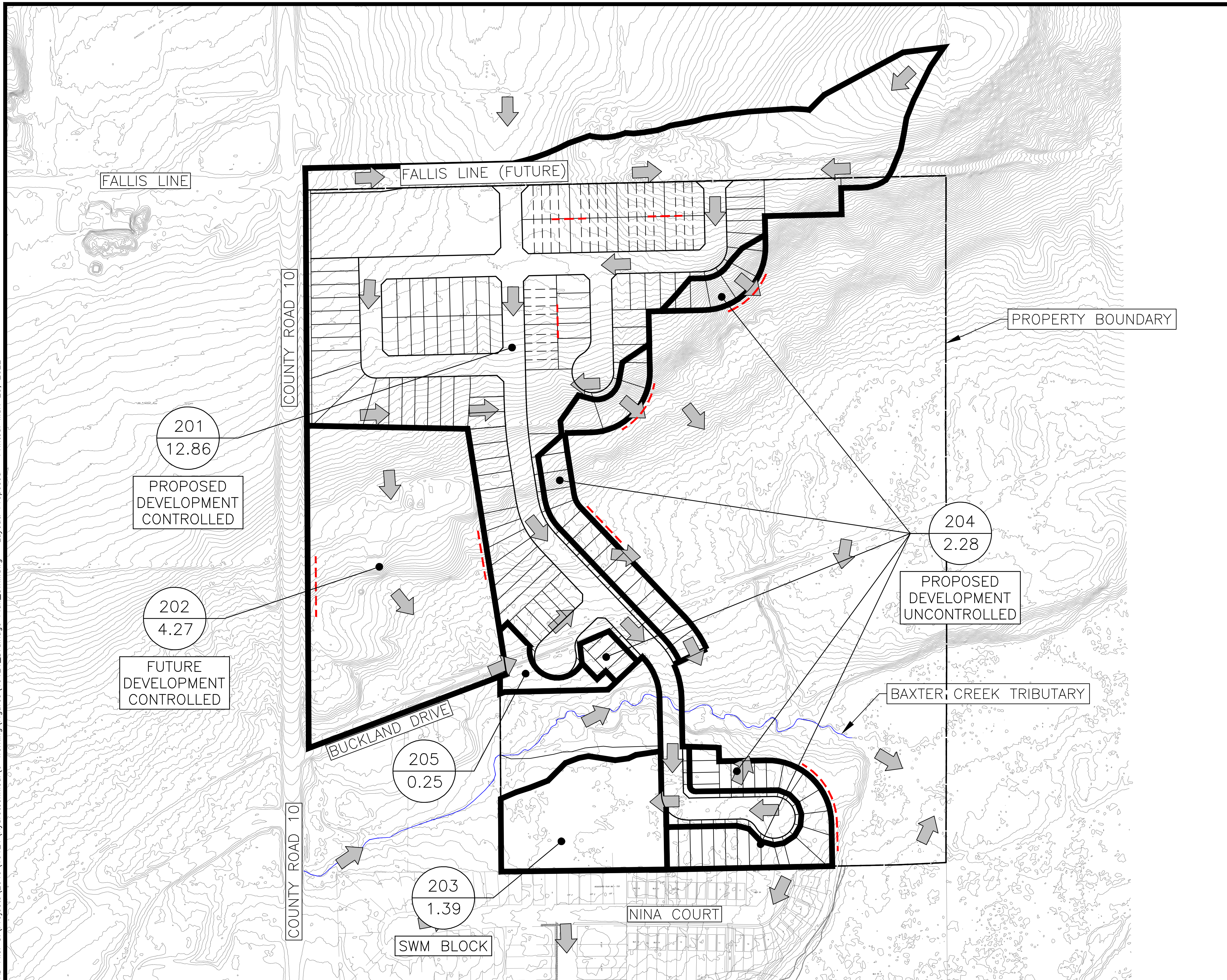
PREPARED BY	O.B.	CKD. BY	O.B.
SCALE	NTS	DATE	MAR. 2022
PROJECT	19121	FIGURE 7	

ACAD File: S:\Projects\2019\19121\Hydrotechnical\0-Working\Figures\19121_Drainage Plan_SWM.dwg Layout: Proposed Printed: Mar. 23, 2022



N

LEGEND

201 — CATCHMENT ID
9.37 — AREA (HA)

DRAINAGE BOUNDARY

OVERLAND FLOW

PROP. INFILTRATION TRENCH

PROJECT
MILLBROOK SOUTH EAST SUBDIVISION

TITLE
SWM DRAINAGE PLAN POST-DEVELOPMENT

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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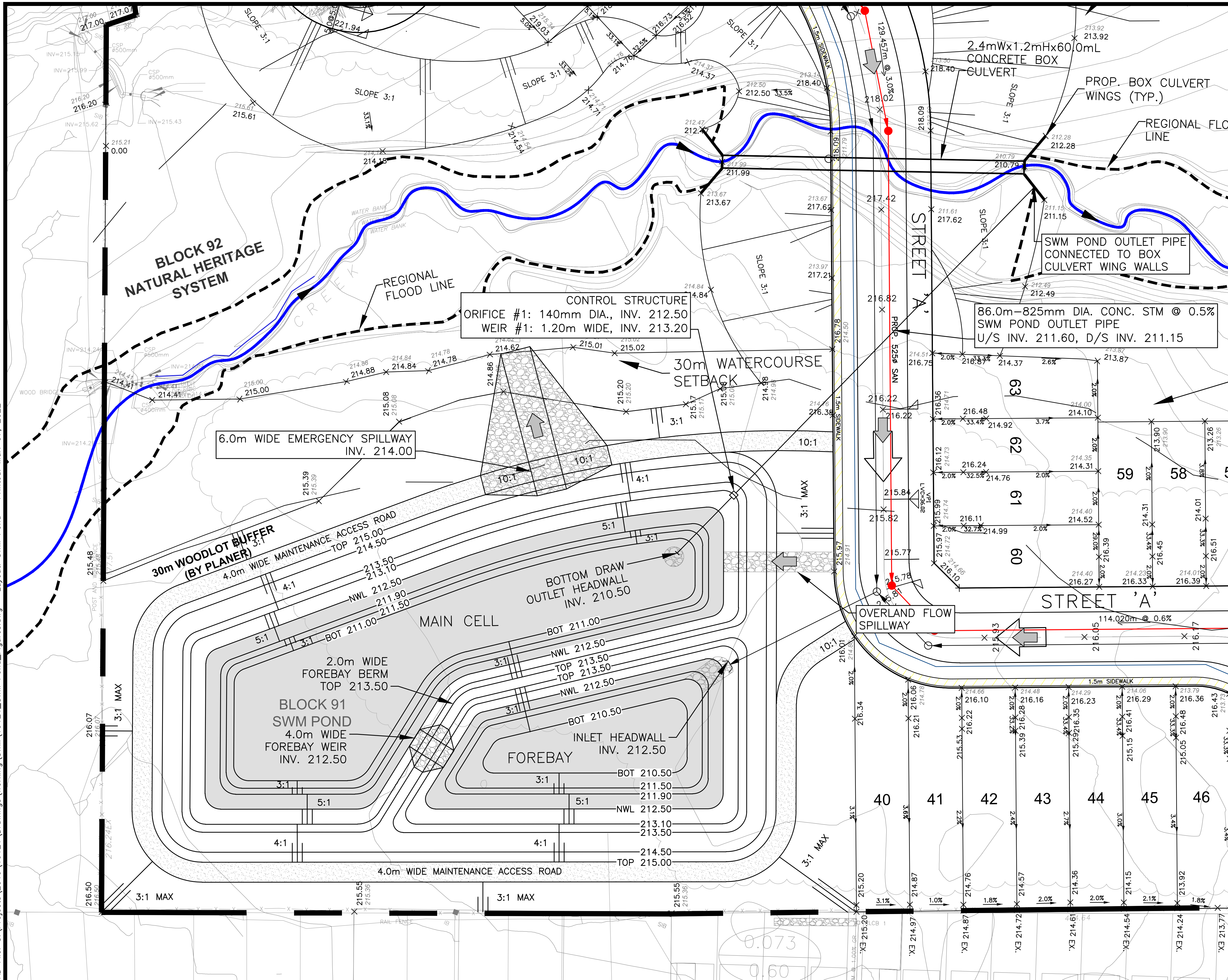
www.valdor-engineering.com

PREPARED BY	P.Y.	CKD. BY	B.C.
SCALE	NTS	DATE	MAR. 2022
PROJECT	19121		FIGURE 8



LEGEND

- OVERLAND FLOW
- PERMANENT POOL
- RIP-RAP EROSION PROTECTION
- MAINTENANCE ACCESS ROAD

RETURN PERIOD	POND WSEL (m)
25 mm	213.07
2-YEAR	213.32
5-YEAR	213.48
10-YEAR	213.59
25-YEAR	213.72
50-YEAR	213.80
100-YEAR	213.89
REGIONAL	214.03

PROJECT
MILLBROOK SOUTH EAST SUBDIVISION

TITLE
PRELIMINARY SWM POND

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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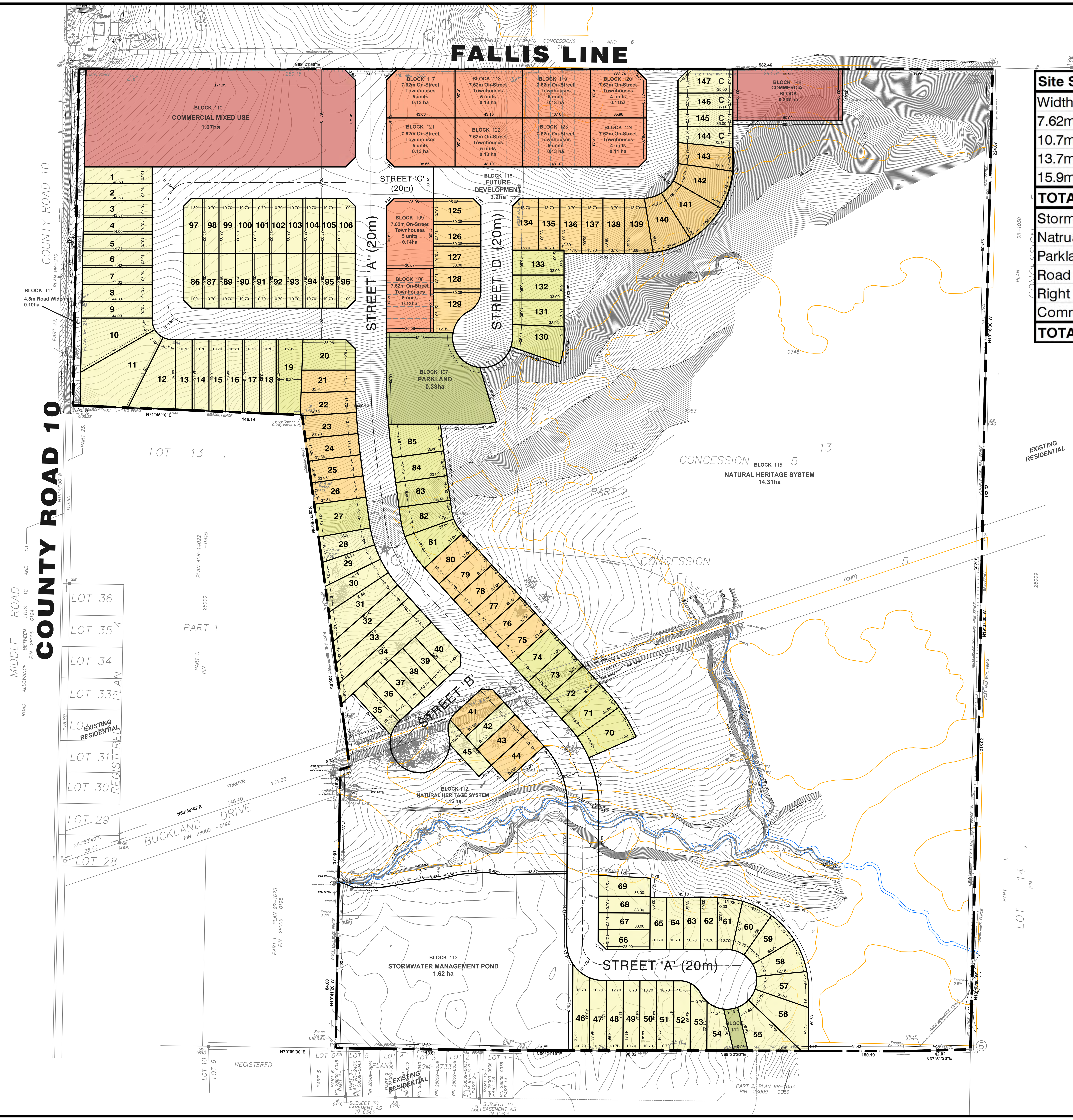
www.valdor-engineering.com

PREPARED BY O.B.	CKD. BY O.B.
SCALE 1:750	DATE MAR. 2022
PROJECT 19121	FIGURE 9

APPENDIX "A"

Draft Plan, Conceptual Master Plan & Equivalent Population Calculation



Site Statistics

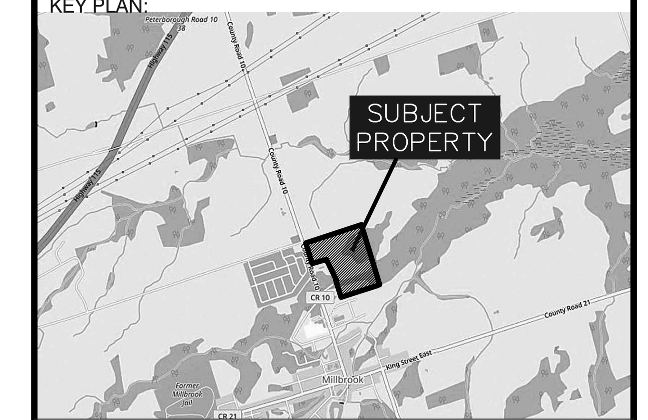
Width Minimums		Units	Area (ha)
7.62m (24') Street Townhouse	D	48	1.28
10.7m (35') Single Detached	C	81	3.78
13.7m (45') Single Detached	B	30	1.48
15.9m (52') Single Detached	A	17	0.98
TOTAL		**176**	**7.52**
Stormwater Management Pond			1.62
Natrual Heritage Systems			15.46
Parkland and Trails			0.36
Road Widening			0.10
Right of Way			3.12
Commerical Block			1.30
TOTAL SUBJECT SITE			**29.48**

TITLE:

CONCEPTUAL MASTER PLAN

LEGAL DESCRIPTION:
DRAFT PLAN OF SUBDIVISION
PART OF LOT 13
CONCESSION 5
TOWNSHIP OF CAVAN
COUNTY OF PETERBOROUGH

KEY PLAN:



REQUIRED INFORMATION:

AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT R.S.O. 1990.

(a) SEE PLAN
(b) SEE PLAN
(c) SEE KEY MAP
(d) SEE SCHEDULE OF LAND USE
(e) SEE PLAN
(f) SEE PLAN
(g) SEE PLAN
(h) PIPED WATER TO BE PROVIDED
(i) CLAY LOAM SOIL
(j) SEE PLAN
(k) SANITARY & STORM SEWERS TO BE PROVIDED
(l) SEE PLAN

NOTE: CONTOURS RELATE TO CANADIAN GEODETIC DATUM

SURVEYOR'S CERTIFICATE:

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AS SHOWN ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATE AND CORRECTLY SHOWN IN ACCORDANCE WITH A PLAN OF SURVEY PREPARED BY

APRIL 27 2021
DATE

DAVID COMERY
ONTARIO LAND SURVEYOR

IBW SURVEYORS
71 MEARNS COURT, BOWMANVILLE, ONTARIO, L1C 4N4
IBWSURVEYORS.COM | 1.800.667.0696

OWNER'S CERTIFICATE:

I HEREBY AUTHORIZE THE BIGLIERI GROUP LTD. TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE

DATE

FALLIS LINE PETERBOUROUGH COUNTY

REVISIONS

No.	Description	Date	Int.
3			
2			
1			

PROJECT No.: 20699

DATE: March 15th, 2022

SCALE: 1:2500

DRAFTED BY: EC

CHECKED BY:

DRAWING No.: **CMP-01**

BIGLIERI GROUP
2472 Kingston Rd, Scarborough
126 Catharine Street North, Hamilton
(416) 693-9155
thebiglierigroup.com

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TABLE: A

EQUIVALENT POPULATION

Project Name: **Millbrook South East Subdivision**
File: 19121
Date: Mar. 2022

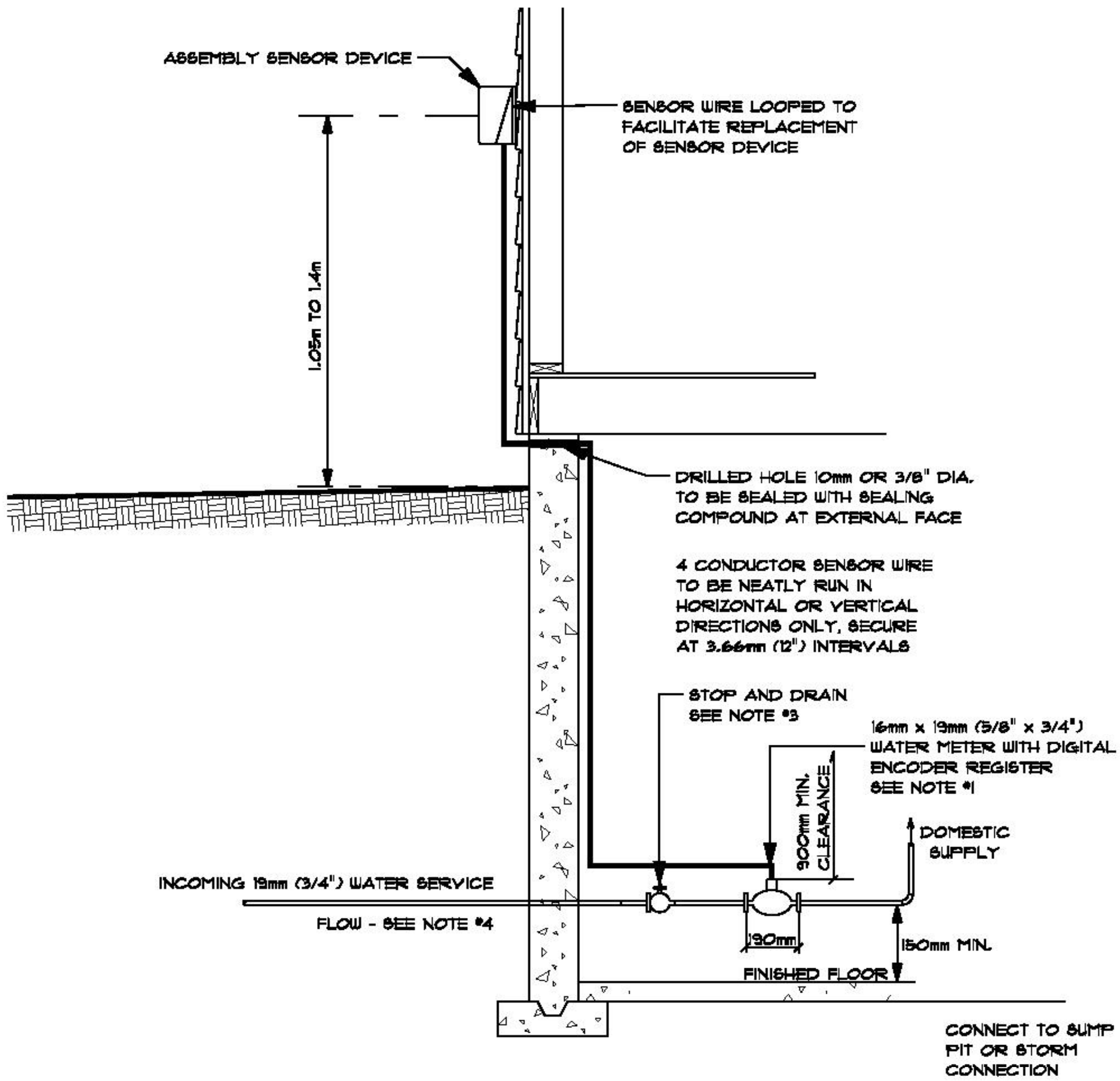
PROPOSED DRAFT PLAN

Land Use	Area (Hectares)	Criteria	No. of Units	Equivalent Population
Detached Dwellings	6.24	3.50 persons per unit	128	448
Street Townhomes	1.28	3.50 persons per unit	48	168
Commercial	1.30	3.68 persons per hectare *		5
Stormwater Management Pond	1.62			
Natural Heritage Systems	15.46			
Parkland & Trails	0.36			
Roads & Road Widenings	3.22			
Total:	**29.48**		**176**	**621**

* Equates to sewage flow of 1.15 L/s/Ha and 450 L/person/day

APPENDIX "B"

Water Demand Calculations & Details



NOTES:

1 - METER SHALL BE 16mm (5/8") METER, REGISTRATION IN CUBIC METERES. 19mm (3/4") THREADED CONNECTIONS

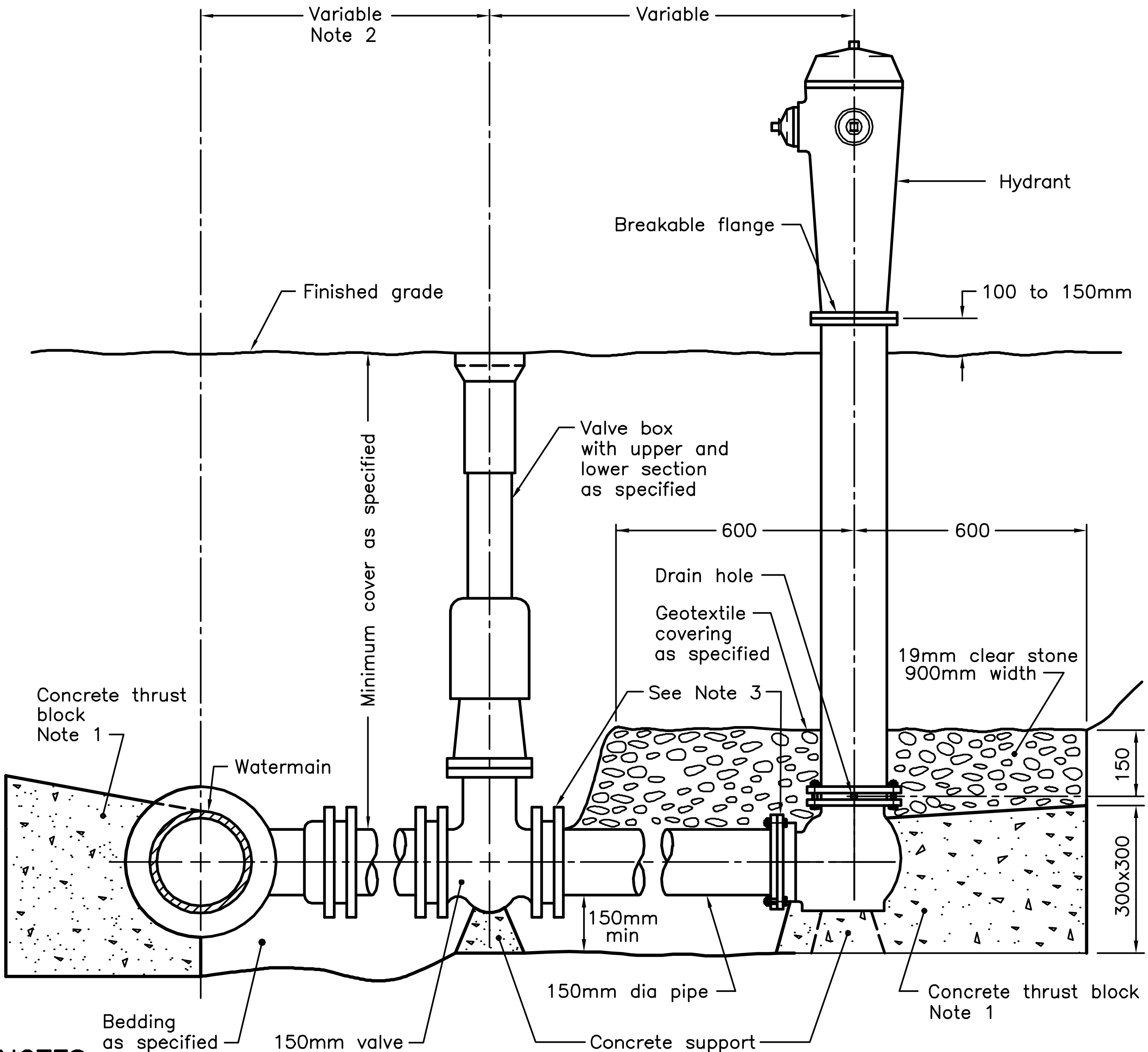
2 - SUPPLY AND INSTALL REMOTE READOUT DEVICE ON OUTSIDE WALL WITHIN 2.0m OF THE FROST WALL AND IN THE SAME SIDE AS THE HYDRO METER. REMOTE READOUT DEVICE SHALL BE SUITABLE FOR TOUCH READ AUTOMATED READING AND BULLING SYSTEM.

3 - STOP AND DRAIN VALVE TO BE THE SAME SIZE AS INCOMING PIPE

4 - IF HOT WATER TANK IS WITHIN 3.0m OF THE METER, A CHECK VALVE IS REQUIRED BETWEEN THE METER AND THE HOT WATER TANK.

5 - METER SHALL BE INSTALLED USING THREADED CONNECTIONS ONLY

TOWNSHIP OF CAVAN MONAGHAN	TYPICAL WATER METER INSTALLATION	STD. S7
	SCALE: NOT TO SCALE	
	DATE: AUGUST 2013	



NOTES:

1 All concrete thrust blocks shall be poured against undisturbed ground.
2 When specified, for watermains 400mm and less, locate valve within 1.0m of centreline of watermain. Retaining and restraining devices shall be utilized. For watermains 600mm and over, bolt valve with flanged end directly to flanged tee.
3 When specified, retaining and restraining devices shall be utilized, in addition to thrust blocks.
A Bond breaker shall be used between the concrete and the fittings and appurtenances.
B Bolts and nuts for buried flange to flange connections shall be stainless steel.
C When required, flange of standpipe extensions shall not be in frost zone.
D This OPSD shall be read in conjunction with OPSD 1103.010 and 1103.020.
E Backfill material within 500mm of service box shall be native or imported, as specified.
F All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2013	Rev 2
HYDRANT INSTALLATION		
	OPSD 1105.010	

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www.valdor-engineering.com

TABLE: B1

DOMESTIC WATER DEMAND CALCULATION

Project Name: **Millbrook South East Subdivision, Township of Cavan Monaghan**
File: 19121
Date: March 2022

Conditions:		
Residential Average Day Demand	450	L/person/day
Maximum Day Factor	2.0	
Peak Hour Factor	3.0	

PROPOSED DRAFT PLAN

Land Use	**Equivalent Population** (persons)	**Domestic Demand** (L/min)	**Maximum Day Demand** (L/min)	**Peak Hour Demand** (L/min)
Detached Dwellings	448	140.0	280.0	420.0
Street Townhomes	168	52.5	105.0	157.5
Commercial Block	5	1.6	3.1	4.7
Total	**621**	**194.1**	**388.1**	**582.2**

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TABLE: B2-1

CALCULATION OF REQUIRED FIRE FLOW

In accordance to Water Supply for Public Fire Protection, Fire Underwriters Survey 1999

Project Name: **Millbrook South East Subdivision**
File: 19121
Date: Mar-22

Notes: **DETACHED DWELLING**
Assume:
- 3,500 sq.ft total floor area
- interior unit for max exposure

Type of Construction - Wood Frame
$C =$ 1.5

Total Floor Area: 325 sq.m
$A =$ 325 sq.m
(Total Floor Area includes all storeys, but excludes basements at least 50 percent below grade)

$F = 220\ C\ \sqrt{A}$
$F =$ 5,949 L/min
$F =$ 6,000 (to nearest 1,000 Lmin)

Occupancy Factor

		Charge
Type:	Non-Combustible	-25%
	$f_1 =$	-25%

$F' = F \text{ x } (1+f_1)$
$F' =$ 4,500 L/min

Sprinkler Credit

		Charge
NFPA 13 Sprinkler Standard:	NO	0%
Standard Water Supply:	NO	0%
Fully Supervised System:	NO	0%
Total Charge to Fire Flow:	$f_2 =$	0%

Exposure Factor		Charge
Side 1 - Distance to Building (m):	0 to 3m	25%
Side 2 - Distance to Building (m):	0 to 3m	25%
Side 3 - Distance to Building (m):	3.1 to 10m	20%
Side 4 - Distance to Building (m):	3.1 to 10m	20%
	$f_3 =$	75% (maximum of 75%)

$F'' = F' + F' \text{ x} f_2 + F' \text{ x} f_3$
$F'' =$ 7,875 L/min

REQUIRED FIRE FLOW
$F'' =$ **8,000** **L/min** (to nearest 1,000 L/min)

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TABLE: B2-2

CALCULATION OF REQUIRED FIRE FLOW

In accordance to Water Supply for Public Fire Protection, Fire Underwriters Survey 1999

Project Name: **Millbrook South East Subdivision**
File: 19121
Date: Mar-22

Notes: **STREET TOWNHOMES DWELLING**
Assume:
- 2,500 sq.ft total floor area
- interior unit for max exposure

Type of Construction - Wood Frame
$C =$ 1.5

Total Floor Area: 233 sq.m
$A =$ 233 sq.m
(Total Floor Area includes all storeys, but excludes basements at least 50 percent below grade)

$F = 220\,C\,\sqrt{A}$
$F =$ 5,037 L/min
$F =$ 5,000 (to nearest 1,000 Lmin)

Occupancy Factor

		Charge
Type:	Non-Combustible	-25%
	$f_1 =$	-25%

$F' = F \times (1+f_1)$
$F' =$ 3,750 L/min

Sprinkler Credit

		Charge
NFPA 13 Sprinkler Standard:	NO	0%
Standard Water Supply:	NO	0%
Fully Supervised System:	NO	0%
Total Charge to Fire Flow:	$f_2 =$	0%

Exposure Factor		Charge
Side 1 - Distance to Building (m):	0 to 3m	25%
Side 2 - Distance to Building (m):	0 to 3m	25%
Side 3 - Distance to Building (m):	3.1 to 10m	20%
Side 4 - Distance to Building (m):	3.1 to 10m	20%
	$f_3 =$	75% (maximum of 75%)

$F'' = F' + F' \times f_2 + F' \times f_3$
$F'' =$ 6,563 L/min

REQUIRED FIRE FLOW
$F'' =$ **7,000** **L/min** (to nearest 1,000 L/min)

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TABLE B-3: REQUIRED FIRE FLOW CALCULATION

In accordance to Water Supply for Public Fire Protection, Fire Underwriters Survey 1999

Project Name: **Millbrook Subdivision Phase 1**
File: 13152
Date: August 2014

Notes:

Type of Construction - Fire Resistive
$C =$ 0.6

For fire-resistive buildings with 1-hour fire rating, the area shall be the total area of the largest floor plus 25% of each of the two immediately adjoining floors (assuming vertical openings and exterior vertical communications are properly protected):

Floor	Area (sq.m)	%
Mechanical Penthouse	437	0%
Level 21-22	581	0%
Level 20	600	0%
Level 17-19	663	0%
Level 16	666	0%
Level 13-15	682	0%
Level 12	647	0%
Level 10-11	682	0%
Level 9	666	0%
Level 6-8	663	0%
Level 5	666	0%
Level 4	798	0%
Level 3	681	25%
Level 2	1,178	25%
Ground	1,300	100%
$A =$	1,765	sq.m

$F = 220\,C\,\sqrt{A}$
$F =$ 5,545 L/min
$F =$ 6,000 (to nearest 1,000 Lmin)

Occupancy Factor

		Charge
Type:	Non-Combustible	-25%
	$f_1 =$	-25%

$F' = F \text{ x } (1+f_1)$
$F' =$ 4,500 L/min

Sprinkler Credit

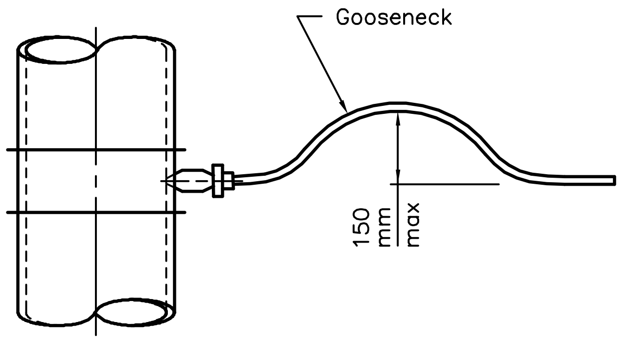
		Charge
NFPA 13 Sprinkler Standard:	YES	-30%
Standard Water Supply:	NO	0%
Fully Supervised System:	NO	0%
Total Charge to Fire Flow:	$f_2 =$	-30%

Exposure Factor

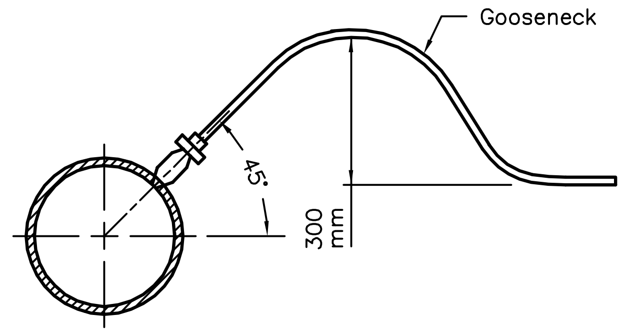
		Charge
North Side - Distance to Building (m):	3 to 10m	20%
East Side - Distance to Building (m):	30 to 45m	5%
South Side - Distance to Building (m):	0 to 3m	25%
West Side - Distance to Building (m):	3 to 10m	20%
	$f_3 =$	70% (maximum of 75%)

$F'' = F' + F' \text{ x } f_2 + F' \text{ x } f_3$
$F'' =$ 6,300 L/min

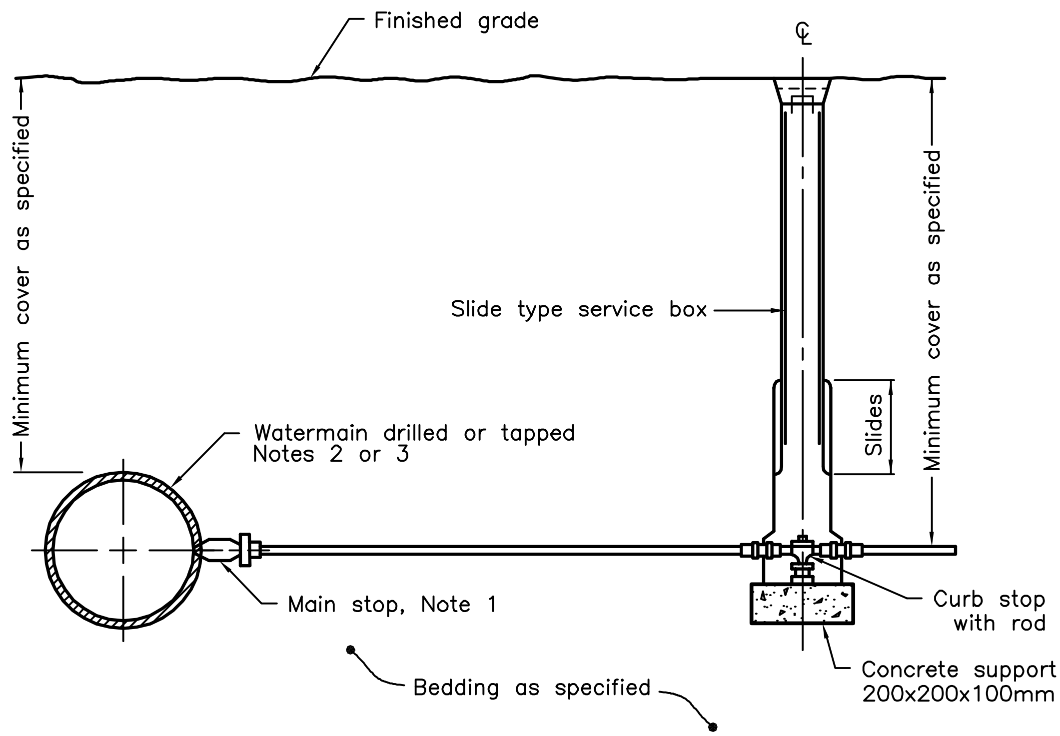
REQUIRED FIRE FLOW
***F''* = 6,000 L/min** (to nearest 1,000 L/min)



HORIZONTAL GOOSENECK



VERTICAL GOOSENECK OPTION



VERTICAL SECTION

NOTES:

1 For plastic service pipes, install main stop at 15° above horizontal with a minimum 1.2m long gooseneck.

2 Direct tap ductile iron pipe with approved tool with standard AWWA inlet thread.

3 Service connections to plastic watermains shall be made using service saddles or factory made tees.

A When specified, the vertical gooseneck option shall be used.

B Couplings shall not be permitted unless the service length exceeds 20m between the main stop and curb stop.

C All water services shall be installed 90° to the longitudinal axis of the watermain.

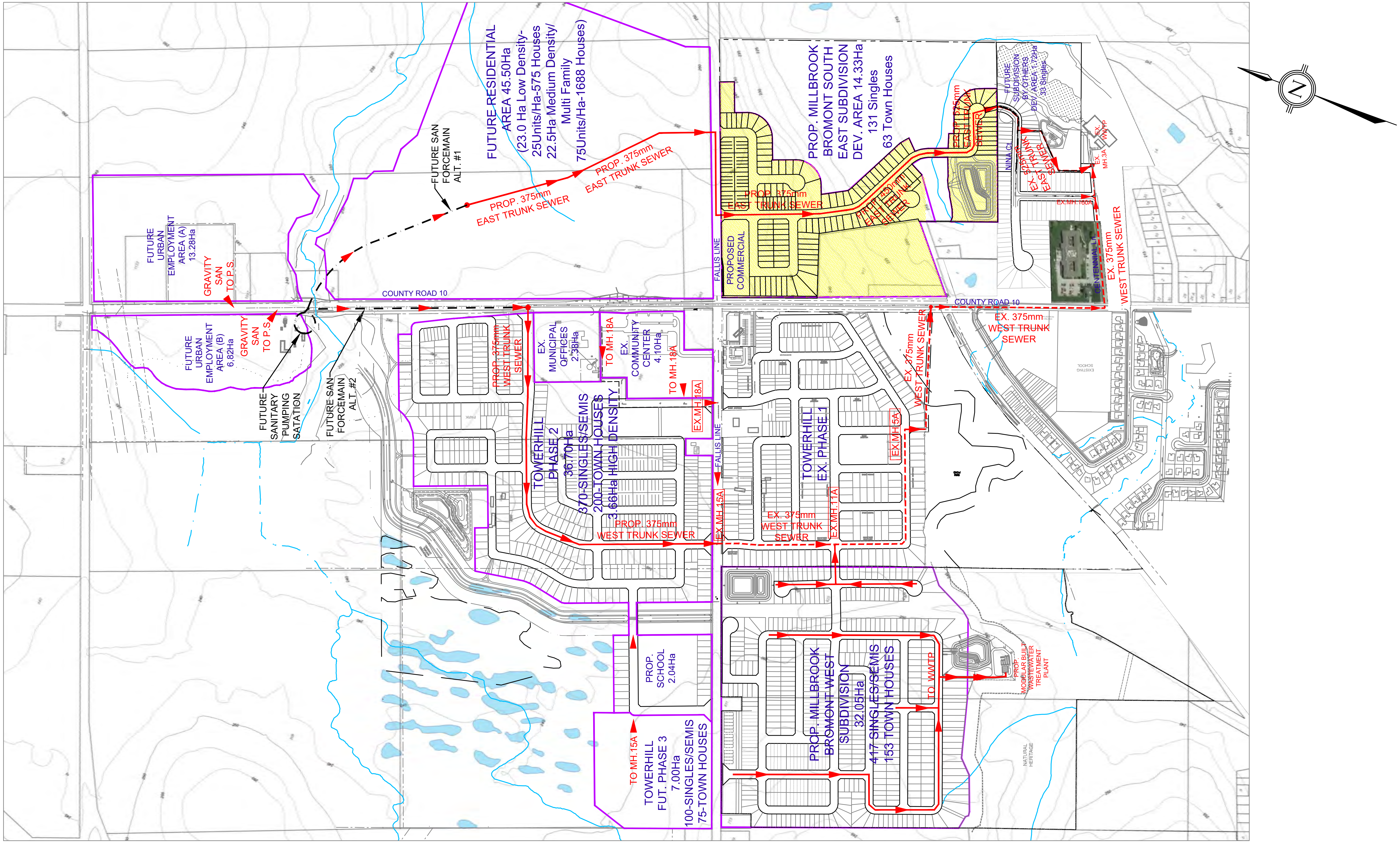
D Backfill material within 500mm of service box shall be native or imported, as specified.

E All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2013	Rev 3
WATER SERVICE CONNECTION 19 and 25mm DIAMETER SIZES	**OPSD 1104.010**	

APPENDIX "C"

Wastewater Servicing Details & Calculations



MILLBROOK SOUTH EAST SUBDIVISION

SANITARY DRAINAGE PLAN

DRAWN BY	V.L.
CKD. BY	D.G.
DATE	March, 2022

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
741 ROWNTREE DAIRY ROAD, SUITE 2, WOODBRIDGE, ONTARIO, L4L 5T9
TEL (905)264-0054, FAX (905)264-0069
E-MAIL: info@valdor-engineering.com
www.valdor-engineering.com

SCALE	PROJECT	
N.T.S.	19121	FIGURE C

Residential			Commercial		
Average Daily Flow	450	l/person/d	Average Daily F	1.15	l/s/ha
Peaking factor, M	2.00	min.	Peaking factor,	2.5	
	4.50	max.			

Infiltration rate	0.28	l/s/ha
Pipe Velocities	0.75	m/s min
	3.00	m/s max

Township of Cavan Monaghan

S A N I T A R Y S E W E R D E S I G N S H E E T

Project Name: Millbrook North Sanitary Trunk Sewer

Project No: 17125

SANITARY FLOW

Consultant:

Valdor Engineering Inc.

741 Rowntree Dairy Road, Suite 2, Woodbridge, Ontario, L4L 5T9

Tel: 905-264-0054 Fax: 905-264-0069 info@valdor-engineering.com

Design: **V. Lider** Date: April, 2021

Checked: **D. Giugovaz, P.Eng**

Approved: **P. Zourntos, P.Eng**

Sheet:

Residential		
Density - Detached	3.5	p.p.u.
Density - Townhome	2.4	p.p.u.
Density - Medium	180	p.p.ha.
Density - High	300	p.p.ha.

School		
Average Daily Flow	0.35	l/s/100 students
Peaking factor, M	1.50	

$M = 1 + \frac{14}{4+\sqrt{P}}$ where P in 1000's

$Q(p) = \frac{P \times q \times M}{86.4}$ in (l / s)

$Q(i) = i \times A$ in (l / s)

$Q(d) = Q(p) + Q(i)$ in (l / s)

Street	from M.H.	to M.H.	Developable Area (ha)	No. of Detached Dwellings (Units)	No. of Street Townhomes (Units)	No. of Medium Density (ha.)	No. of High Density (ha.)	Industrial Commercial Institutional (ha.)	Population (person)	Accum. Population P (person)	Peaking Factor M	Industrial Institutional Commercial Flow (l/s)	Peak Flow Q(p) (l/s)	Accum. Area (ha)	Extraneous Flow Q(i) (l/s)	Design Flow Q(d) (l/s)	Pipe Length L (m)	Pipe Diameter d (mm)	Pipe Slope S (%)	Nominal Pipe Full Flow Cap. Q(f), (l/s)	Pipe Full Flow Vel. V(f) (m/s)	Actual Velocity V(a) (m/s)	Q(d) / Q(f)	COMMENTS
FUT. DEV. NORTH COMMERCIAL	PLUG	MH.15AA	20.10	0	0		0	20.100	0	0	2.50	23.12	50.25	20.100	5.63	55.88	50.0	375	0.3%	96.0	0.87	0.90	0.58	
FUT. TOWERHILL PHASE 2 (Residential)	PLUG	MH.15AA	34.66	370	200		3.660	0.000	2873.0	2873.0	3.46	0.00	51.75	34.66	9.70	61.45	50.0	375	0.3%	96.0	0.87	0.92	0.64	
FUT. TOWERHILL PHASE 2 (School)	PLUG	MH.15AA	2.04	0	0		0.000	2.040	0.0	0.0	1.50	2.80	4.20	2.04	0.57	4.77	50.0	375	0.3%	96.0	0.87	0.45	0.05	
FUT. TOWERHILL PHASE 3	PLUG	MH.15AA	7.00	100	75		0.000	0.000	530.0	530.0	3.96	0.00	10.93	7.00	1.96	12.89	50.0	375	0.3%	96.0	0.87	0.61	0.13	
TOTAL FUT. FLOW to EX.MH.15A (N. Developments)	MH.15AA	EX.MH.15A	63.80	470	275		3.660	22.140	3403.0	3403.0	3.40	25.92	114.63	63.80	17.86	132.49	50.0	375	0.3%	96.0	0.87	0.88	1.38	
EX. Municipality&Community (To MH.18A-OLD Criteria)	EX.MH.24A	EX.MH.18A	6.480	0	0		0	6.480	0	0	2.00	0.75	1.50	6.480	1.81	3.31	80.0	200	1.3%	38.0	1.21	0.74	0.09	
EX. FALLIS LINE (To MH 15A-OLD Criteria)	EX.MH.18A	EX.MH.15A	3.150	30	0		0	0.000	78.0	78.0	4.00	0.00	1.63	3.150	0.88	2.51	60.0	200	1.0%	32.8	1.04	0.62	0.08	
EX. TOWERHILL PHASE 1 (To MH.11A-OLD Criteria)	EX.MH.15A	EX.MH.11A	4.970	87	0		0	0.000	226.2	226.20	4.00	0.00	4.71	14.600	4.09	8.80	60.0	375	0.3%	96.0	0.87	0.54	0.09	
TOTAL FLOW TO EX. MH.11A	EX.MH.15A	EX.MH.11A	78.4	587	275		3.660	28.620	3707.2	3707.2	3.36	26.67	120.88	78.4	21.95	142.83	60.0	375	0.3%	96.0	0.87	0.88	1.49	
FUT. SOUTH WEST SUDIVISION	PLUG	MH.11A	32.050	417	153		0	0	1826.7	1826.7	3.62	0.00	34.40	32.050	8.97	43.38	60.0	250	1.0%	59.5	1.21	1.32	0.73	
TOTAL FUT. FLOW TO EX. MH.10A (Highlands Blvd.)	EX.MH.11A	EX.MH.10A	110.5	1004	428		3.660	28.620	5533.9	5533.9	3.20	26.67	148.29	110.5	30.93	179.22	60.0	375	0.3%	96.0	0.87	0.88	1.87	
EX. TOWERHILL PHASE 1 (To MH 5A-OLD Criteria)	EX.MH.11A	EX.MH.5A	17.400	245	0		0	0.000	637.0	637.00	4.00	0.00	13.27	17.400	4.87	18.14	60.0	375	0.3%	96.0	0.87	0.67	0.19	
TOTAL FUTURE FLOW TO EX. MH.1A	EX.MH.5A	EX.MH.1A	127.9	1249	428		3.660	28.620	6170.9	6170.9	3.16	26.67	157.48	127.9	35.80	193.28	60.0	375	2.7%	288.1	2.61	2.80	0.67	
EX.Tupper St./County Road 10 (Old Criteria)	EX.MH.1A	EX.MH.22A	5.260	51	0		0	0	132.6	132.6	4.00	0.00	2.76	5.260	1.47	4.24	50.0	375	0.9%	166.3	1.51	0.64	0.03	
TOTAL FUTURE FLOW TO EX. MH.22A	EX.MH.1A	EX.MH.22A	133.1	1300	428		3.660	28.620	6303.5	6303.5	3.15	26.67	159.38	133.1	37.27	196.65	60.0	375	0.9%	166.3	1.51	1.53	1.18	
EX.Centennial Lane (Old Criteria)	EX.MH.22A	EX.MH.3A	8.130	241	0		0	0	483.6	483.6	4.00	0.00	10.08	8.130	2.28	12.35	50.0	375	0.61%	136.9	1.24	0.77	0.09	
TOTAL FUTURE FLOW TO EX. MH.3A (WEST DEV.)	EX.MH.22A	EX.MH.3A	141.2	1541	428		3.660	28.620	6787.1	6787.1	3.12	26.67	166.22	141.2	39.55	205.77	60.0	375	0.61%	136.9	1.24	1.26	1.50	
FUTURE NORTH EAST SUBDIVISION (NORTH FALLIS LINE	MH200A	MH.300A	47.700	575	1688		0	0	6063.7	6063.7	3.17	0.00	100.00	47.700	13.36	113.36	60.0	375	1.0%	175.3	1.59	1.69	0.65	
EASTCOMMERCIAL (SOUTH FALLIS LINE)	MH.350A	MH.300A	3.11	0	0		0	3.110	0	0	2.50	3.58	7.78	3.110	0.87	8.65	50.0	200	1.0%	32.8	1.04	0.88	0.26	
SOUTH EAST SUBDIVISION (SOUTH FALLIS LINE)	MH.300A	MH.SA-6A	9.520	108	62		0.000	0.000	526.8	6590.5	3.13	0.00	107.50	60.330	16.89	133.04	60.0	375	1.0%	175.3	1.59	1.75	0.76	
FUTURE SUBDIVISION (BY OTHERS)	MH.SA-6A	EX.MH.3A	1.720	33	0		0.000	0.000	115.5	6706.0	3.12	0.00	109.13	62.050	17.37	135.15	60.0	375	1.0%	175.3	1.59	1.75	0.77	

No.	REVISION	DATE	AUTH
2.000			
1.000			

Township of Cavan Monaghan
Engineering and Public Works Department

SANITARY SEWER DESIGN SHEET

SCALE: N.T.S.

DRAWN BY: DWG.

VALDOR ENGINEERING INC.
741 Rowntree Dairy Road, Suite 2, Woodbridge, ON L4L 5T9
Tel: 905-264-0054 Fax: 905-264-0069 info@valdor-engineering.com
www.valdor-engineering.com

TABLE: C1

WASTEWATER FLOW CALCULATIONS

Project Name: **Millbrook South East Subdivision, Township of Cavan Monaghan**
File: 19121
Date: Mar-22

Conditions:

Residential Average Daily Flow: 450 L/person/day

Residential Peaking Factor: $K_H = 1 + \frac{14}{4+\sqrt{P}}$ where K_H = Harmon Peaking Factor (max. 4.5, min. 2.0)
p = population in thousands

Extraneous Flow (I): 0.28 L/s/ha. (infiltration)

Design Flow (Q_D): $Q \times K_H + I$

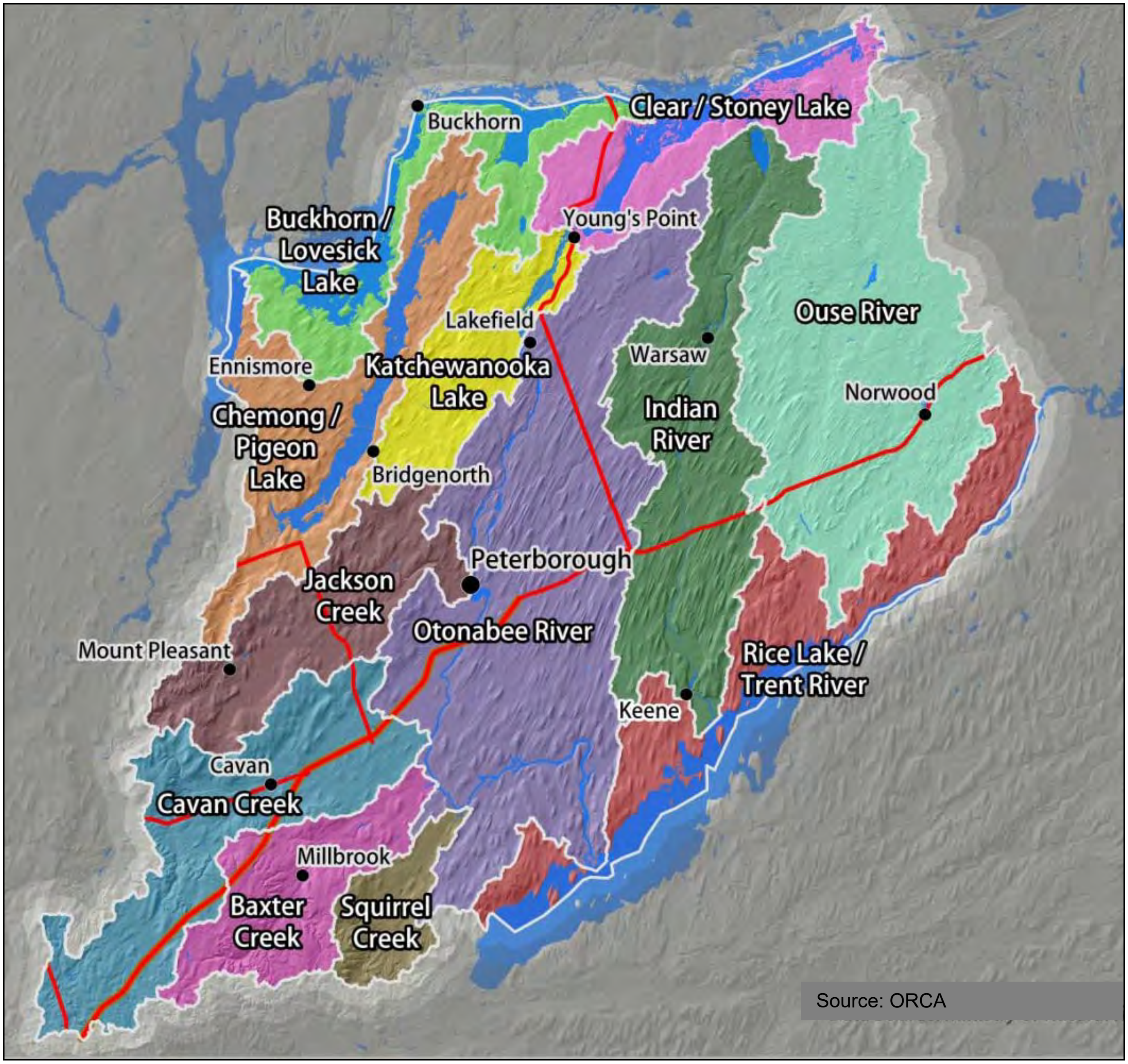
Commercial/instituational Average Daily Flow: 1.15 L/s/ha
Commercial/Instituational Peaking Factor: 2.5

PROPOSED DRAFT PLAN

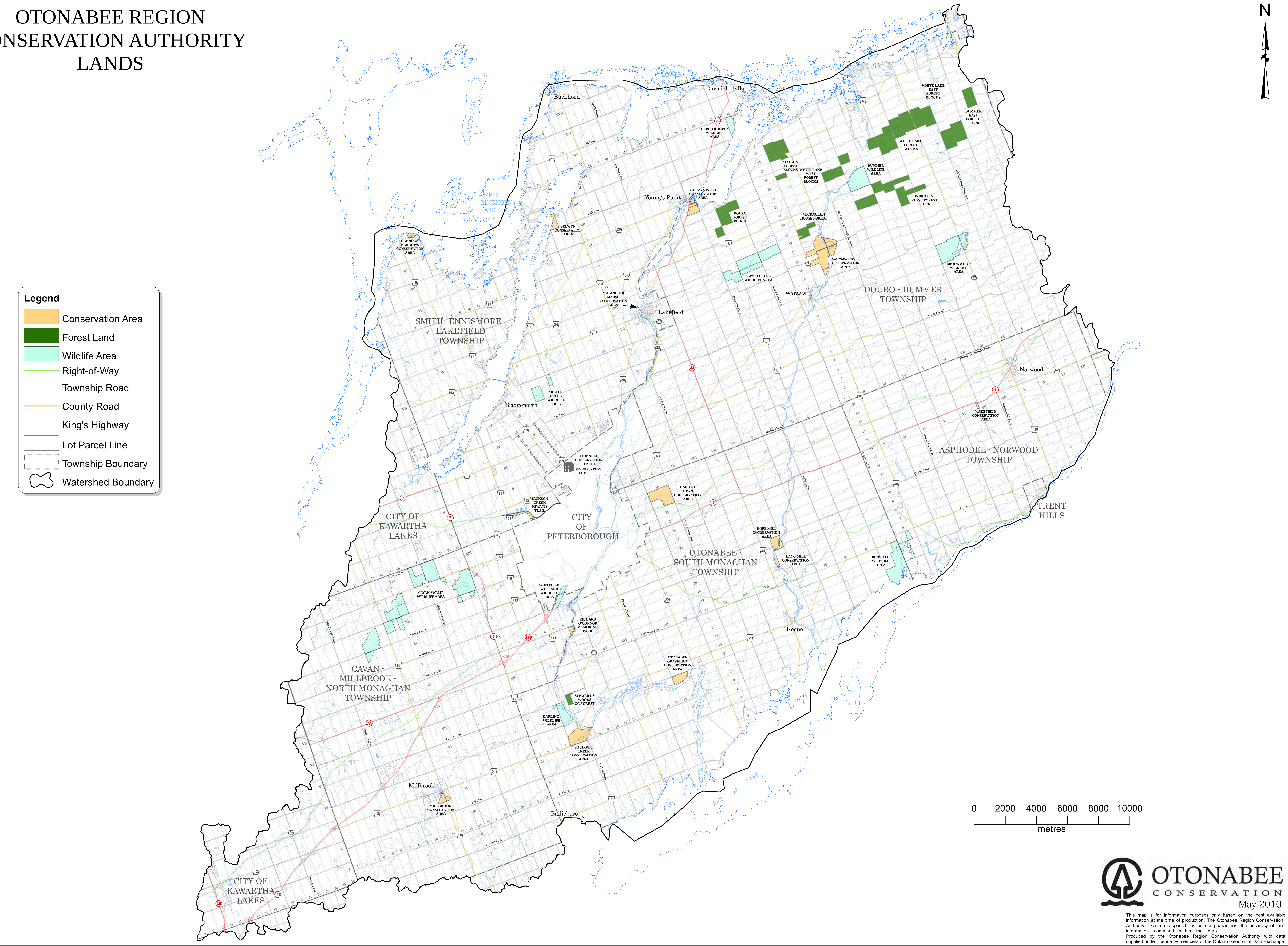
Land Use	Area (ha.)	Equivalent Population (persons)	Average Daily Flow (L/s)	Harmon Peaking Factor	Peak Daily Flow (L/s)	Extraneous Flow (L/s)	Total Flow (L/s)
Detached Dwellings	6.29	448	2.33	4.00	9.33	1.76	11.09
Street Townhomes	1.28	168	0.88	4.17	3.65	0.36	4.01
Commercial Block	1.30	5	1.50	2.50	3.74	0.36	4.10
Roads	3.26					0.91	0.91
Total	**12.13**	**621**	**4.70**		**16.72**	**3.40**	**20.12**

APPENDIX "D"

Storm Drainage Details


Buckhorn
Clear / Stoney Lake
Buckhorn / Lovesick Lake
Young's Point
Lakefield
Ouse River
Warsaw
Ennismore
Katchewanooka Lake
Norwood
Chemong / Pigeon Lake
Indian River
Bridgenorth
Peterborough
Jackson Creek
Otonabee River
Mount Pleasant
Rice Lake / Trent River
Keene
Cavan
Cavan Creek
Millbrook
Baxter Creek
Squirrel Creek
Source: ORCA

OTONABEE REGION CONSERVATION AUTHORITY LANDS



OTONABEE
CONSERVATION

May 2010

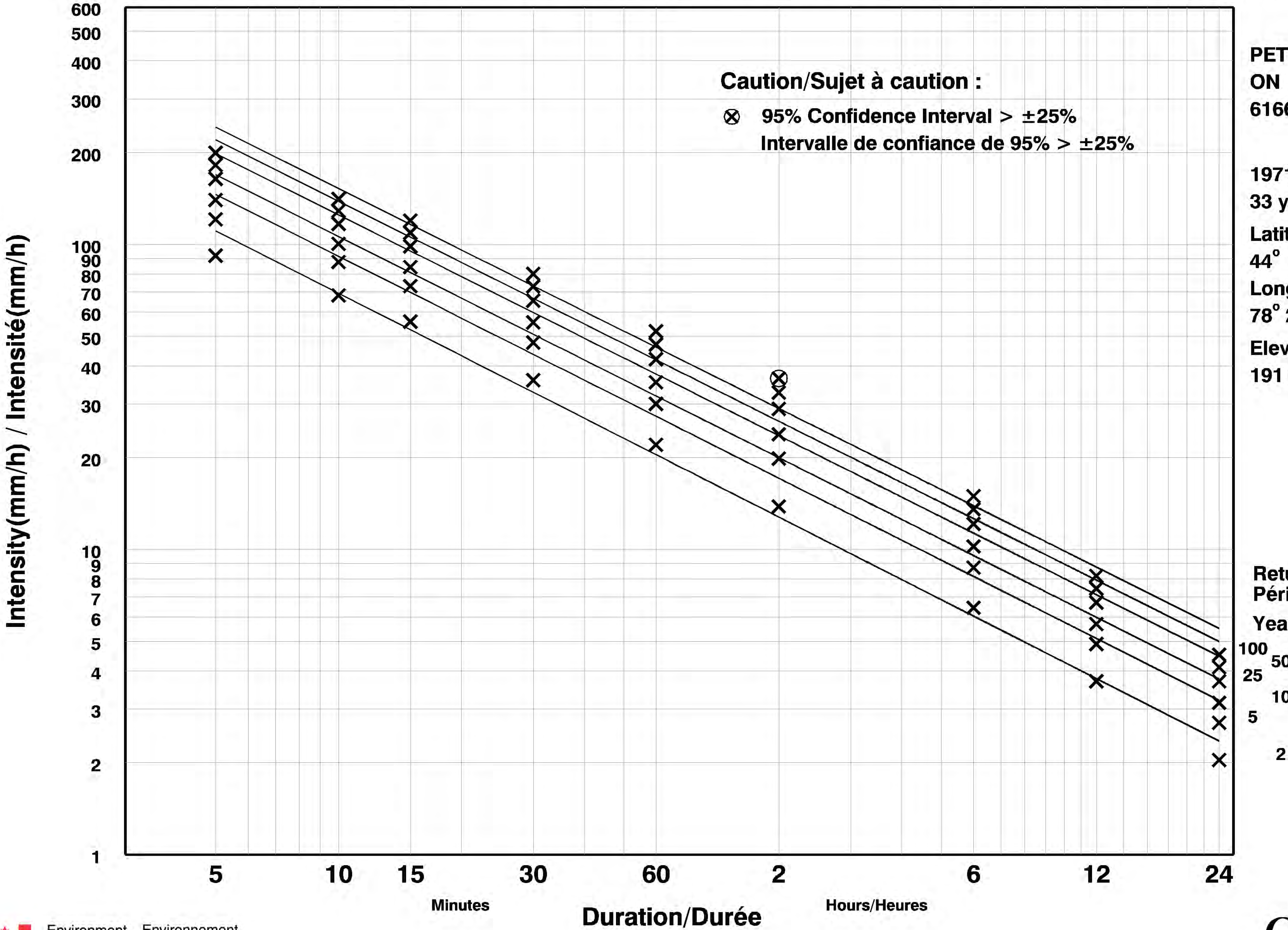
This map is for information purposes only based on the best available information at the time of production. The Otonabee Region Conservation Authority takes no responsibility for, nor guarantees, the accuracy of the information contained within the map.

Produced by the Otonabee Region Conservation Authority with data supplied under licence by members of the Ontario Geospatial Data Exchange.

Short Duration Rainfall Intensity-Duration-Frequency Data

2014/12/21

Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée



PETERBOROUGH A
ON
6166418

1971 - 2006
33 years / ans

Latitude
44° 14'N
Longitude
78° 22'W
Elevation / Altitude
191 m

Return Periods/
Périodes de retour
Years / ans

Intensity Duration Frequency Statistics for Peterborough

Location - Peterborough Airport 2014 Data

Rainfall Intensity = a/(Tc+b)^c

Tc = Time of Concentration

2 Year Return Period		
a	b	c
583.351	6.010	0.773
Duration	**Intensity**	
5	92.0	
10	68.2	
15	56.0	
30	35.9	
60	22.1	
120	13.9	
360	6.4	
720	3.7	
1440	2.0	

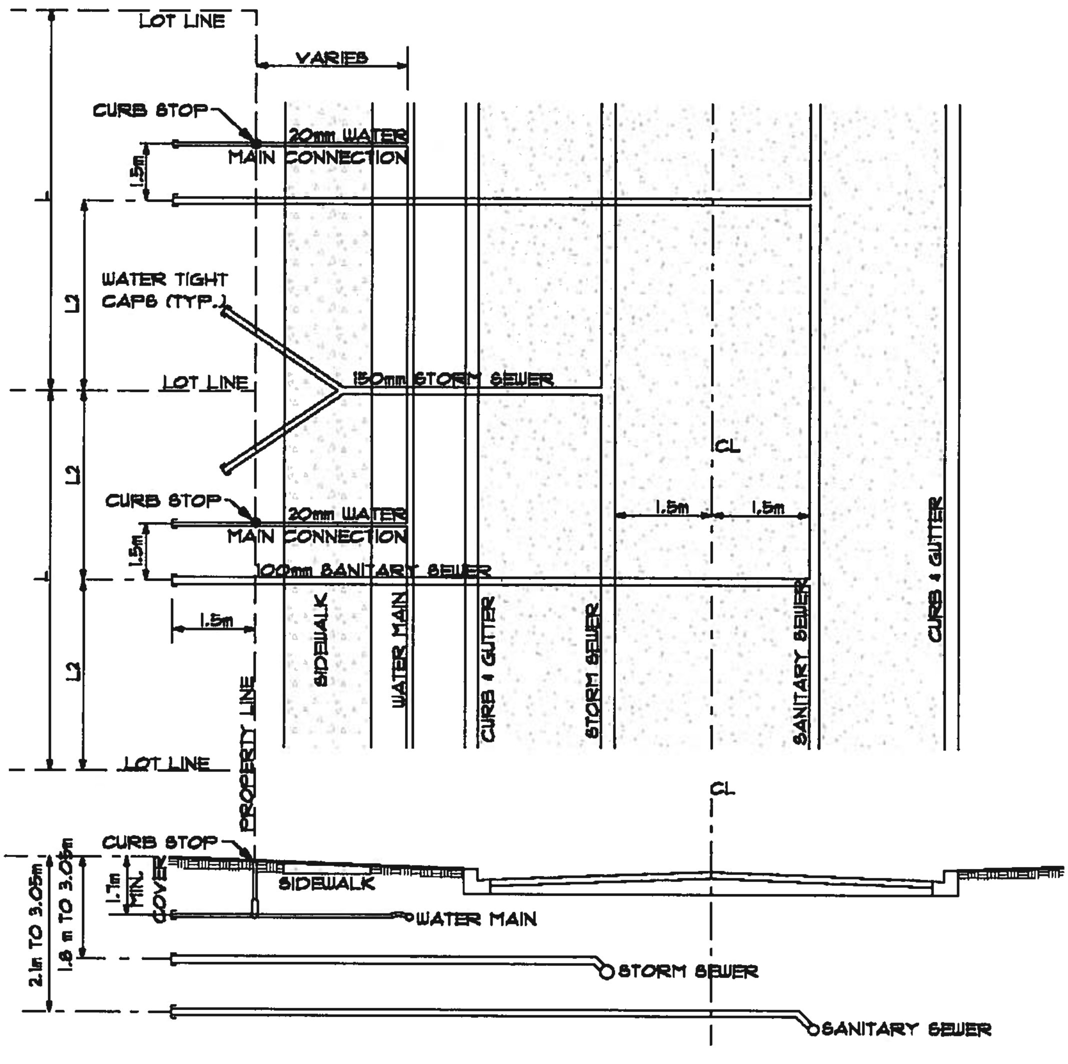
5 Year Return Period		
a	b	c
843.799	7.500	0.783
Duration	**Intensity**	
5	121.0	
10	87.7	
15	73.1	
30	47.8	
60	30.1	
120	19.9	
360	8.7	
720	4.9	
1440	2.7	

10 Year Return Period		
a	b	c
1034.243	8.265	0.791
Duration	**Intensity**	
5	140.2	
10	100.7	
15	84.5	
30	55.6	
60	35.4	
120	23.9	
360	10.2	
720	5.7	
1440	3.1	

25 Year Return Period		
a	b	c
1263.414	9.012	0.795
Duration	**Intensity**	
5	164.4	
10	117.0	
15	98.8	
30	65.5	
60	42.1	
120	29.0	
360	12.2	
720	6.7	
1440	3.7	

50 Year Return Period		
a	b	c
1468.915	9.751	0.801
Duration	**Intensity**	
5	182.3	
10	129.1	
15	109.4	
30	72.9	
60	47.1	
120	32.7	
360	13.6	
720	7.5	
1440	4.1	

100 Year Return Period		
a	b	c
1696.952	10.502	0.808
Duration	**Intensity**	
5	200.2	
10	141.1	
15	120.0	
30	80.2	
60	52.0	
120	36.4	
360	15.0	
720	8.2	
1440	4.5	



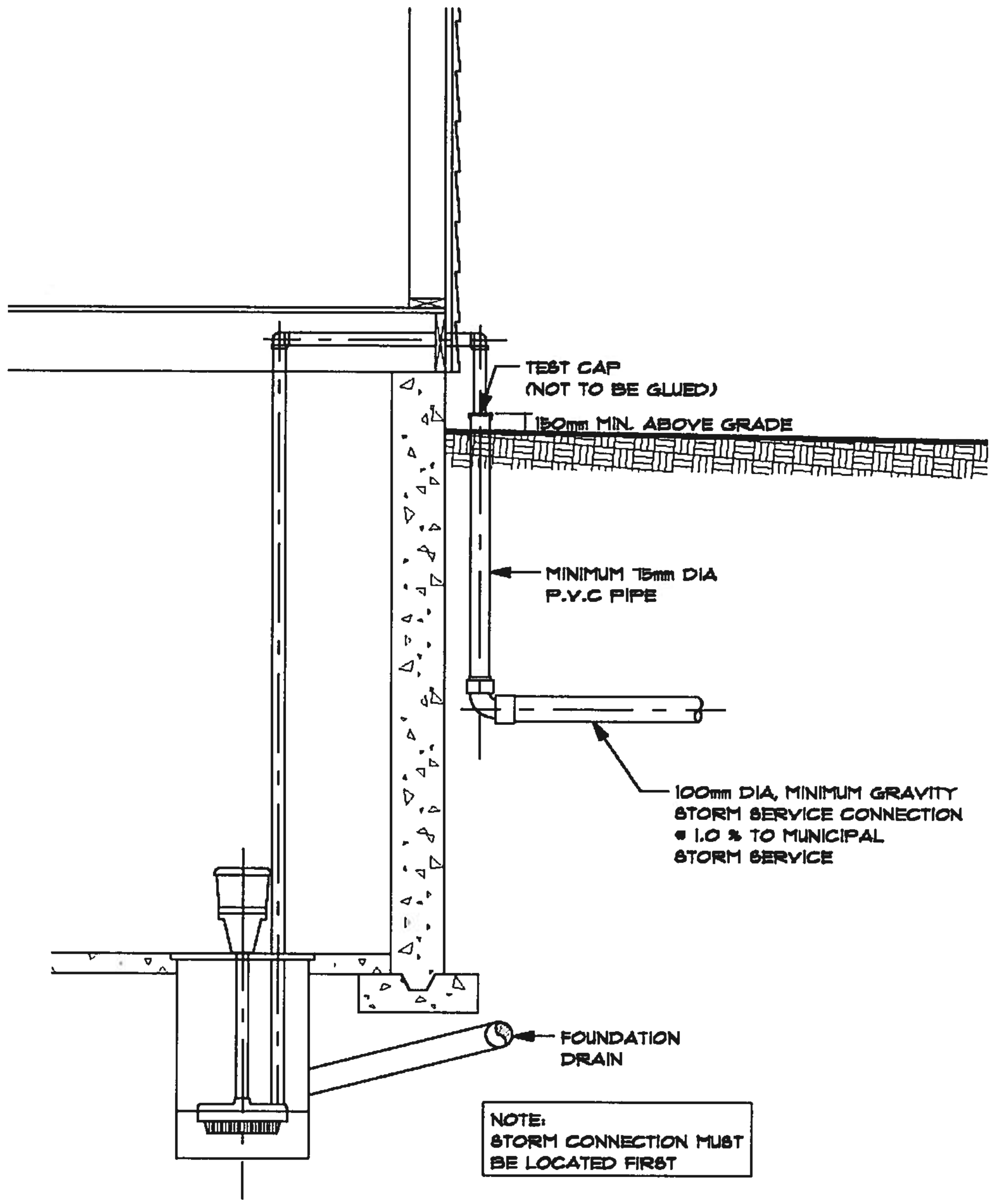
NOTES:
1. WATERTIGHT CAPS ON ALL SERVICES.
2. ALL DIMENSIONS SHOWN ARE CENTRE TO CENTRE.
3. STORM PIPE MATERIALS IS TO BE PVE SDR 28 AND WHITE IN COLOUR
4. L = FRONTAGE OF ONE UNIT

TOWNSHIP OF CAVAN MONAGHAN

STORM SERVICE RESIDENTIAL SERVICE CONNECTION

SCALE: NOT TO SCALE

DATE: AUGUST 2013



TOWNSHIP OF CAVAN MONAGHAN	SUMP PUMP TO STORM SEWER CONNECTION	STD. S2
	SCALE: NOT TO SCALE	
	DATE: AUGUST 2013	

Worksheet for ROW Ponding for Street A

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Channel Slope	0.03000	m/m
Discharge	0.99	m³/s
Section Definitions		

Station (m)	Elevation (m)
0+00	0.25
0+05	0.15
0+06	0.00
0+10	0.10
0+14	0.00
0+15	0.15
0+20	0.25

Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+00, 0.25)	(0+05, 0.15)	0.035
(0+05, 0.15)	(0+06, 0.00)	0.013
(0+06, 0.00)	(0+10, 0.10)	0.013
(0+10, 0.10)	(0+14, 0.00)	0.013
(0+14, 0.00)	(0+15, 0.15)	0.035
(0+15, 0.15)	(0+20, 0.25)	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

Normal Depth	0.11	m

Worksheet for ROW Ponding for Street A

Results

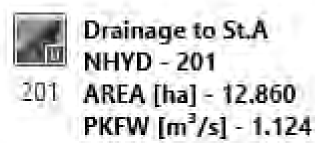
Elevation Range	0.00 to 0.25 m	
Flow Area	0.58	m²
Wetted Perimeter	9.44	m
Hydraulic Radius	0.06	m
Top Width	9.42	m
Normal Depth	0.11	m
Critical Depth	0.16	m
Critical Slope	0.00528	m/m
Velocity	1.72	m/s
Velocity Head	0.15	m
Specific Energy	0.26	m
Froude Number	2.21	
Flow Type	Supercritical	

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	0.11	m
Critical Depth	0.16	m
Channel Slope	0.03000	m/m
Critical Slope	0.00528	m/m



VO5 Model Schematic – Ponding at Street A

```
==========================================================================================
==============
==========================================================================================
==============

       V    V   I    SSSSS  U   U    A    L             (v 6.2.2006)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M  OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```



```
                   *****  D E T A I L E D   O U T P U T *****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DATE: 03/10/2022                           TIME: 10:49:58

USER:


COMMENTS: ____________________________________________________________


------------------------------------------------------------------------------------------
--------------
  ************************************************
  ** SIMULATION : AES_06hr_005yr                **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\PYung\AppD
|                  |              ata\Local\Temp\
|                  |              77effeab-87b7-420f-a9d0-8e757d3be1bf\8f03ed2d
| Ptotal= 52.41 mm |    Comments: AES_06hr_005yr
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                 0.25    0.00 |  2.00   17.82 |  3.75    7.34 |  5.50    1.05
                 0.50    1.05 |  2.25   17.82 |  4.00    4.19 |  5.75    1.05
                 0.75    1.05 |  2.50   48.21 |  4.25    4.19 |  6.00    1.05
                 1.00    1.05 |  2.75   48.21 |  4.50    2.10 |  6.25    1.05
                 1.25    1.05 |  3.00   13.62 |  4.75    2.10 |
                 1.50    6.29 |  3.25   13.62 |  5.00    1.05 |
                 1.75    6.29 |  3.50    7.34 |  5.25    1.05 |

------------------------------------------------------------------------------

--------------------
| CALIB            |
| STANDHYD (  0201)|   Area    (ha)=  12.86
|ID= 1 DT= 5.0 min |   Total Imp(%)=  70.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       9.00         3.86
     Dep. Storage     (mm)=       2.00         5.00
     Average Slope     (%)=       2.00         2.00
     Length            (m)=     292.80        40.00
     Mannings n          =      0.013        0.250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                            ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
               0.083    0.00 | 1.667    6.29 | 3.250   13.62 |  4.83    1.05
               0.167    0.00 | 1.750    6.29 | 3.333    7.34 |  4.92    1.05
               0.250    0.00 | 1.833   17.82 | 3.417    7.34 |  5.00    1.05
               0.333    1.05 | 1.917   17.82 | 3.500    7.34 |  5.08    1.05
               0.417    1.05 | 2.000   17.82 | 3.583    7.34 |  5.17    1.05
               0.500    1.05 | 2.083   17.82 | 3.667    7.34 |  5.25    1.05
               0.583    1.05 | 2.167   17.82 | 3.750    7.34 |  5.33    1.05
               0.667    1.05 | 2.250   17.82 | 3.833    4.19 |  5.42    1.05
               0.750    1.05 | 2.333   48.21 | 3.917    4.19 |  5.50    1.05
               0.833    1.05 | 2.417   48.21 | 4.000    4.19 |  5.58    1.05
               0.917    1.05 | 2.500   48.21 | 4.083    4.19 |  5.67    1.05
               1.000    1.05 | 2.583   48.21 | 4.167    4.19 |  5.75    1.05
               1.083    1.05 | 2.667   48.21 | 4.250    4.19 |  5.83    1.05
               1.167    1.05 | 2.750   48.21 | 4.333    2.10 |  5.92    1.05
               1.250    1.05 | 2.833   13.62 | 4.417    2.10 |  6.00    1.05
               1.333    6.29 | 2.917   13.62 | 4.500    2.10 |  6.08    1.05
               1.417    6.29 | 3.000   13.62 | 4.583    2.10 |  6.17    1.05
               1.500    6.29 | 3.083   13.62 | 4.667    2.10 |  6.25    1.05
               1.583    6.29 | 3.167   13.62 | 4.750    2.10 |

     Max.Eff.Inten.(mm/hr)=      48.21        16.66
                over (min)        5.00        20.00
     Storage Coeff.  (min)=       5.29 (ii)   19.75 (ii)
     Unit Hyd. Tpeak (min)=       5.00        20.00
     Unit Hyd. peak  (cms)=       0.21         0.06
                                                          *TOTALS*
     PEAK FLOW       (cms)=       1.03         0.11         1.124 (iii)
     TIME TO PEAK    (hrs)=       2.75         2.92           2.75
     RUNOFF VOLUME    (mm)=      50.41        10.88          34.60
     TOTAL RAINFALL   (mm)=      52.41        52.41          52.41
     RUNOFF COEFFICIENT   =       0.96         0.21           0.66


       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  53.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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       V    V   I    SSSSS  U   U    A    L             (v 6.2.2006)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M  OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```

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                   *****  D E T A I L E D   O U T P U T *****


  Input   filename: C:\Program Files (x86)\Visual OTTHYMO 6.2\VO2\voin.dat
  Output  filename: C:\Users\PYung\AppData\Local\Civica\VH5\ec2d128f-cd40-49c7-9652-
18c6ee52e553\c127b6f3-f020-43f8-a81d-4f22222846e2\scenar
  Summary filename: C:\Users\PYung\AppData\Local\Civica\VH5\ec2d128f-cd40-49c7-9652-
18c6ee52e553\c127b6f3-f020-43f8-a81d-4f22222846e2\scenar


DATE: 03/10/2022                           TIME: 10:49:58

USER:



COMMENTS: ____________________________________________________________


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  ************************************************
  ** SIMULATION : AES_06hr_100yr                **
  ************************************************


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|    READ STORM    |    Filename: C:\Users\PYung\AppD
|                  |              ata\Local\Temp\
|                  |              77effeab-87b7-420f-a9d0-8e757d3be1bf\2873ba7d
| Ptotal= 89.91 mm |    Comments: AES_06hr_100yr
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                 0.25    0.00 |  2.00   30.57 |  3.75   12.59 |  5.50    1.80
                 0.50    1.80 |  2.25   30.57 |  4.00    7.19 |  5.75    1.80
                 0.75    1.80 |  2.50   82.71 |  4.25    7.19 |  6.00    1.80
                 1.00    1.80 |  2.75   82.71 |  4.50    3.60 |  6.25    1.80
                 1.25    1.80 |  3.00   23.37 |  4.75    3.60 |
                 1.50   10.79 |  3.25   23.37 |  5.00    1.80 |
                 1.75   10.79 |  3.50   12.59 |  5.25    1.80 |

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| CALIB            |
| STANDHYD (  0201)|   Area    (ha)=  12.86
|ID= 1 DT= 5.0 min |   Total Imp(%)=  70.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       9.00         3.86
     Dep. Storage     (mm)=       2.00         5.00
     Average Slope     (%)=       2.00         2.00
     Length            (m)=     292.80        40.00
     Mannings n          =      0.013        0.250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                              ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    0.00 | 1.667   10.79 | 3.250   23.37 |  4.83    1.80
                0.167    0.00 | 1.750   10.79 | 3.333   12.59 |  4.92    1.80
                0.250    0.00 | 1.833   30.57 | 3.417   12.59 |  5.00    1.80
                0.333    1.80 | 1.917   30.57 | 3.500   12.59 |  5.08    1.80
                0.417    1.80 | 2.000   30.57 | 3.583   12.59 |  5.17    1.80
                0.500    1.80 | 2.083   30.57 | 3.667   12.59 |  5.25    1.80
                0.583    1.80 | 2.167   30.57 | 3.750   12.59 |  5.33    1.80
                0.667    1.80 | 2.250   30.57 | 3.833    7.19 |  5.42    1.80
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                0.750    1.80 | 2.333   82.71 | 3.917    7.19 |  5.50    1.80
                0.833    1.80 | 2.417   82.71 | 4.000    7.19 |  5.58    1.80
                0.917    1.80 | 2.500   82.71 | 4.083    7.19 |  5.67    1.80
                1.000    1.80 | 2.583   82.71 | 4.167    7.19 |  5.75    1.80
                1.083    1.80 | 2.667   82.71 | 4.250    7.19 |  5.83    1.80
                1.167    1.80 | 2.750   82.71 | 4.333    3.60 |  5.92    1.80
                1.250    1.80 | 2.833   23.37 | 4.417    3.60 |  6.00    1.80
                1.333   10.79 | 2.917   23.37 | 4.500    3.60 |  6.08    1.80
                1.417   10.79 | 3.000   23.37 | 4.583    3.60 |  6.17    1.80
                1.500   10.79 | 3.083   23.37 | 4.667    3.60 |  6.25    1.80
                1.583   10.79 | 3.167   23.37 | 4.750    3.60 |

     Max.Eff.Inten.(mm/hr)=      82.71        46.43
                over (min)        5.00        15.00
     Storage Coeff.  (min)=       4.27 (ii)   13.86 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=       0.23         0.08
                                                           *TOTALS*
     PEAK FLOW       (cms)=       1.77         0.36          2.114 (iii)
     TIME TO PEAK    (hrs)=       2.75         2.83          2.75
     RUNOFF VOLUME    (mm)=      87.91        29.10         64.39
     TOTAL RAINFALL   (mm)=      89.91        89.91         89.91
     RUNOFF COEFFICIENT   =       0.98         0.32          0.72

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  53.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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APPENDIX "E"

Flood Plain Analysis

VALDOR ENGINEERING INC.
File: 19121
Date: March 2022

Table E.1-A: VO Model Parameters - Floodplain

Catchment	Area (ha)	VO5 Routine	TIMP	XIMP	CN II	CN*	IA (mm)	Tp (hr)
401	28.90	StandHyd	0.60	0.45	61	53	5.0	-
402	15.49	NasHyd	-	-	62	58	8.6	0.31
403	23.90	NasHyd	-	-	63	59	8.0	0.36
Total	**68.29**							

VALDOR ENGINEERING INC.
File: 19121
Date: March 2022

Table E.1-B: VO Model Parameters - Floodplain

Catchment	Area (ha)	VO5 Routine	TIMP	XIMP	CN II	CN*	IA (mm)	Tp (hr)
401	28.90	StandHyd	0.60	0.45	61	53	5.0	-
402	15.49	NasHyd	-	-	62	58	8.6	0.31
403A	5.63	NasHyd	-	-	62	56	7.4	0.21
403B	4.34	NasHyd	-	-	55	48	9.8	0.21
201	12.86	StandHyd	0.80	0.70	61	53	5.0	-
202	4.27	StandHyd	0.60	0.45	61	53	5.0	-
203	1.39	StandHyd	0.50	0.50	61	53	5.0	-
204	2.28	StandHyd	0.50	0.30	61	53	5.0	-
205	0.25	NashHyd	-	-	61	53	5.0	0.03
206	0.08	NashHyd	-	-	55	48	10.0	0.06
Total	**75.49**							

VALDOR ENGINEERING INC.
File: 19121
Date: March 2022

Table E.2: Calculation of CN Values, Initial Abstractions and Runoff Coefficients - Floodplain

Catchment	Area (ha)	Land Use and Land Cover		CN II	Area Weighted CN II	IA (mm)	Area Weighted IA (mm)	C-Value	Area Weighted C-Value
		Type	Area (ha)						
402	15.49	Forest (HSG 'B')	12.64	55	**62**	**10**	**8.6**	0.30	**0.39**
		Meadow (HSG 'B')	0.00	58		**8**		0.40	
		Row Crops (HSG 'B')	0.00	81		**7**		0.65	
		Open Space (HSG 'B')	0.55	61		**5**		0.25	
		Other Impervious	2.30	98		**2**		0.95	
403	23.90	Forest (HSG 'B')	9.16	55	**63**	**10**	**8.0**	0.30	**0.43**
		Meadow (HSG 'B')	5.60	58		**8**		0.40	
		Row Crops (HSG 'B')	5.99	75		**7**		0.65	
		Open Space (HSG 'B')	2.36	61		**5**		0.25	
		Other Impervious	0.79	98		**2**		0.95	
403A	5.63	Forest (HSG 'B')	2.82	55	**62**	**10**	**7.4**	0.30	**0.38**
		Meadow (HSG 'B')	0.46	58		**8**		0.40	
		Row Crops (HSG 'B')	0.00	81		**7**		0.65	
		Open Space (HSG 'B')	1.63	61		**5**		0.25	
		Other Impervious	0.72	98		**2**		0.95	
403B	4.34	Forest (HSG 'B')	3.95	55	**55**	**10**	**9.8**	0.30	**0.31**
		Meadow (HSG 'B')	0.39	58		**8**		0.40	
		Row Crops (HSG 'B')	0.00	81		**7**		0.65	
		Open Space (HSG 'B')	0.00	61		**5**		0.25	
		Other Impervious	0.00	98		**2**		0.95	

VALDOR ENGINEERING INC.
File: 19121
Date: March 2022

Table E.3: Calculation of Time to Peak - Floodplain

Catchment	A Area (ha)	C Runoff Coefficient (Area Weighted)	L (m) Catchment Length	Highest Elevation (m)	Lowest Elevation (m)	S (%) Catchment Slope	[1,2]T_c Method	T_c (min)	[1,2]T_p (hr)
402	15.49	0.39	830.0	247.00	218.00	3.49	Airport	28.0	0.31
403	23.90	0.43	725.0	255.00	209.50	6.28	Bransby-Williams	32.2	0.36
403A	5.63	0.38	113.0	222.75	209.50	11.73	Airport	11.1	0.21
403B	4.34	0.31	320.0	255.00	209.50	14.22	Airport	19.2	0.21
206	0.08	0.30	14.0	214.10	213.34	5.43	Airport	5.6	0.06

Uplands Method

Catchment 403A - Channelized flow portion

Landtype	Length	Cu	Slope	V (m/s)	Tc (s)	Tp (s)	Tp (hr)
Grassed Waterway	522	4.6	0.060823755	1.13	460.13	308.28	0.09

Notes:

1) T_p calculation for catchments with C < 0.40 is based on the Airport Formula:
$$T_c = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S_w^{\ 0.33}}$$

2) T_p calculation for catchments with C > 0.40 is based on the Bransby-Williams Formula:
$$T_c = \frac{(0.057)(L)}{(S_w)^{0.2}(A)^{0.1}}$$

$$T_P = 0.67 T_c$$

3) T_p for Catchment 403A is a combination of uplands method along the road ditch and overland flow for the rest of catchment

VALDOR ENGINEERING INC.
File: 19121
Date: March 2022

Table E.4: HEC-RAS Output - Pre-Development

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Watercourse	17	Regional	5.19	215.00	215.63		215.73	0.008964	1.4	3.9	13.46	0.73
Watercourse	16	Regional	5.19	214.60	215.41		215.48	0.004404	1.35	7.45	27.24	0.55
Watercourse	15	Regional	5.19	214.20	215.08	215.08	215.25	0.010378	1.96	4.24	17.79	0.82
Watercourse	14	Regional	5.19	213.80	214.68	214.68	214.87	0.009938	2.21	4.58	14.73	0.82
Watercourse	13	Regional	5.19	213.60	214.38	214.38	214.58	0.011501	2.11	3.76	12.36	0.87
Watercourse	12	Regional	5.19	213.20	213.95	213.95	214.11	0.008896	1.92	5.34	26.72	0.76
Watercourse	11	Regional	5.19	213.00	213.69		213.77	0.004118	1.39	6.38	16.62	0.54
Watercourse	10	Regional	5.19	212.60	213.36	213.36	213.57	0.011143	2.35	4.4	14.13	0.87
Watercourse	9	Regional	5.19	211.80	212.40	212.36	212.56	0.0092	1.88	4.33	13.99	0.79
Watercourse	8	Regional	5.19	211.40	212.07	212.07	212.26	0.010621	2.04	3.71	12.5	0.82
Watercourse	7	Regional	5.19	211.00	211.59	211.59	211.71	0.009557	1.82	6.05	25.63	0.77
Watercourse	6	Regional	5.19	210.20	210.97	210.97	211.10	0.011112	2.17	6.03	22.42	0.82
Watercourse	5	Regional	5.19	210.00	210.59	210.59	210.76	0.01075	2	4.53	16.34	0.84
Watercourse	4	Regional	5.19	209.40	210.10	210.06	210.26	0.009256	1.82	4.07	14.03	0.77
Watercourse	3	Regional	5.19	209.20	209.90	209.90	210.06	0.010445	2.03	5.1	19.76	0.81
Watercourse	2	Regional	5.19	209.00	209.55	209.55	209.67	0.011322	1.94	6.29	27.62	0.85
Watercourse	1	Regional	5.19	208.80	209.16	209.16	209.22	0.022181	1.96	7.78	48.67	1.08

VALDOR ENGINEERING INC.
File: 19121
Date: March 2022

Table E.5: HEC-RAS Output - Post-Development												
Reach	**River Sta**	**Profile**	**Q Total**	**Min Ch El**	**W.S. Elev**	**Crit W.S.**	**E.G. Elev**	**E.G. Slope**	**Vel Chnl**	**Flow Area**	**Top Width**	**Froude # Chl**
			(m3/s)	**(m)**	**(m)**	**(m)**	**(m)**	**(m/m)**	**(m/s)**	**(m2)**	**(m)**	
Watercourse	17	Regional	6.25	215.00	215.67		215.78	0.008853	1.5	4.53	14.77	0.74
Watercourse	16	Regional	6.25	214.60	215.46		215.53	0.004402	1.42	8.95	30.12	0.56
Watercourse	15	Regional	6.25	214.20	215.14	215.14	215.31	0.009643	2.02	5.43	20.68	0.81
Watercourse	14	Regional	6.25	213.80	214.74	214.74	214.94	0.010002	2.33	5.51	16.4	0.83
Watercourse	13	Regional	6.25	213.60	214.44	214.44	214.65	0.011023	2.21	4.58	13.58	0.87
Watercourse	12	Regional	6.25	213.20	214.03	214.03	214.16	0.007283	1.86	7.51	32.02	0.7
Watercourse	11	Regional	6.25	213.00	213.73		213.82	0.004666	1.54	7.09	17.3	0.58
Watercourse	10	Regional	6.25	212.60	213.45	213.45	213.63	0.008865	2.27	5.87	15.84	0.79
Watercourse	9.5	Regional	6.25	211.99	213.27		213.28	0.000372	0.58	15.59	20.54	0.18
Watercourse	8.5	Regional	6.25	211.80	213.17	212.56	213.24	0.001555	1.28	5.96	12.97	0.36
Watercourse	8		Culvert									
Watercourse	7.5	Regional	6.25	211.20	211.93	211.93	212.24	0.014111	2.55	2.94	14.22	0.97
Watercourse	6.5	Regional	6.25	210.79	211.35		211.44	0.010207	1.82	7.32	22.25	0.8
Watercourse	6	Regional	6.25	210.20	211.01	211.01	211.15	0.011002	2.25	7.11	23.65	0.83
Watercourse	5	Regional	6.25	210.00	210.64	210.64	210.82	0.010639	2.1	5.39	17.17	0.85
Watercourse	4	Regional	6.25	209.40	210.15	210.12	210.32	0.01001	1.99	4.7	15.22	0.81
Watercourse	3	Regional	6.25	209.20	209.95	209.95	210.12	0.010349	2.13	6.18	22.03	0.82
Watercourse	2	Regional	6.25	209.00	209.59	209.59	209.71	0.011473	2.04	7.41	29.79	0.86
Watercourse	1	Regional	6.25	208.80	209.18	209.18	209.25	0.022797	2.07	8.76	49.07	1.1

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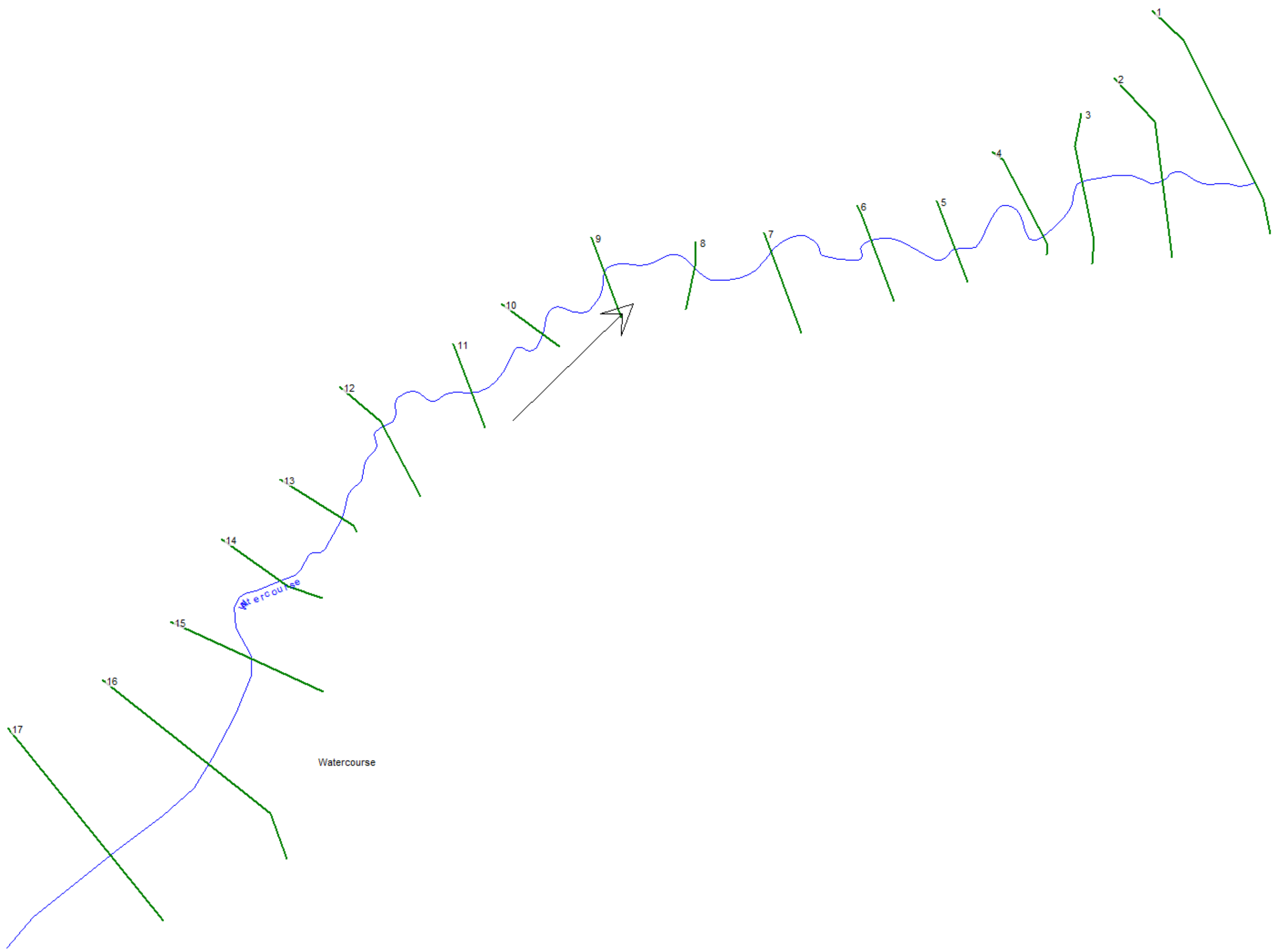
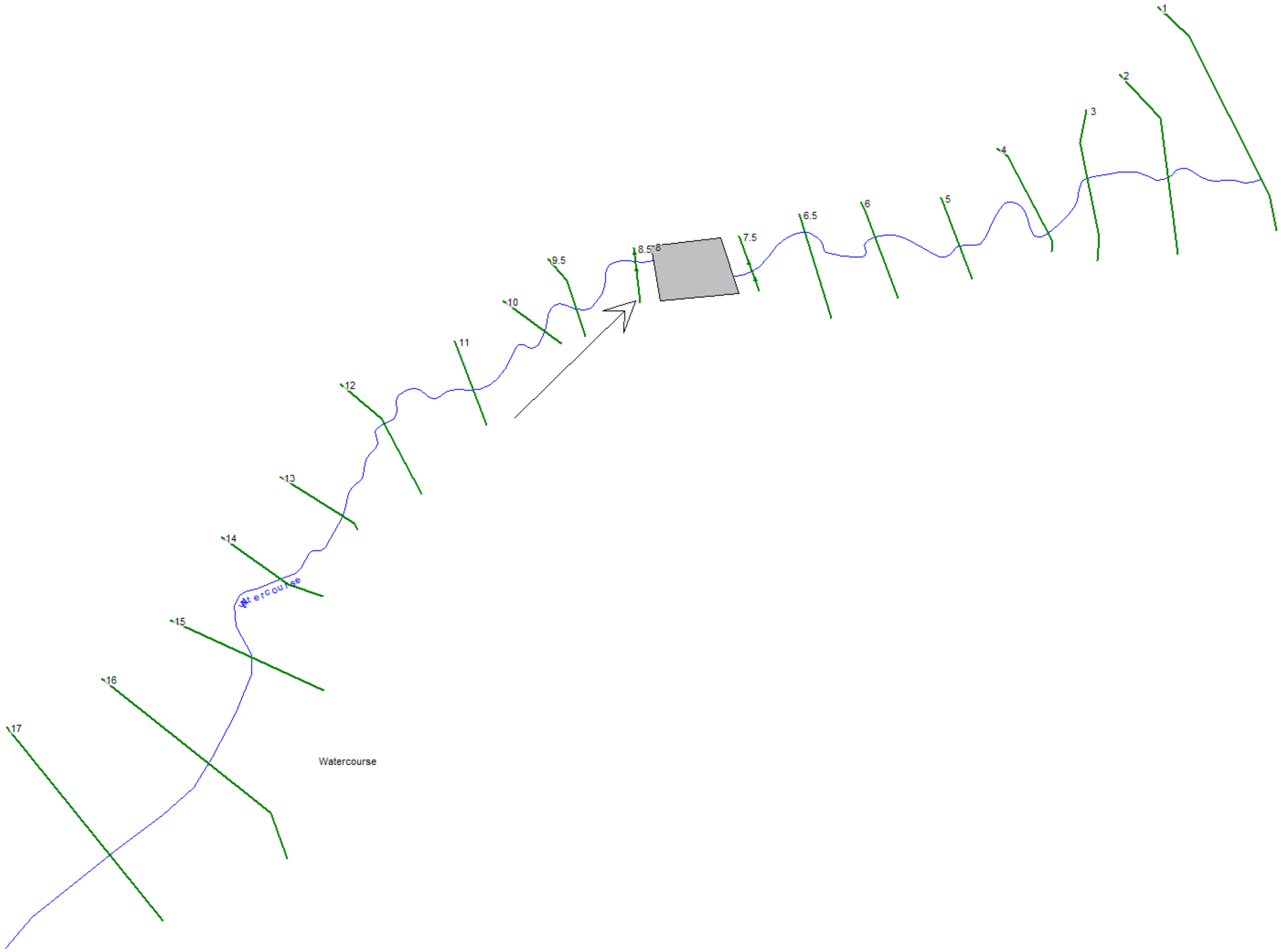
Date: March 2022



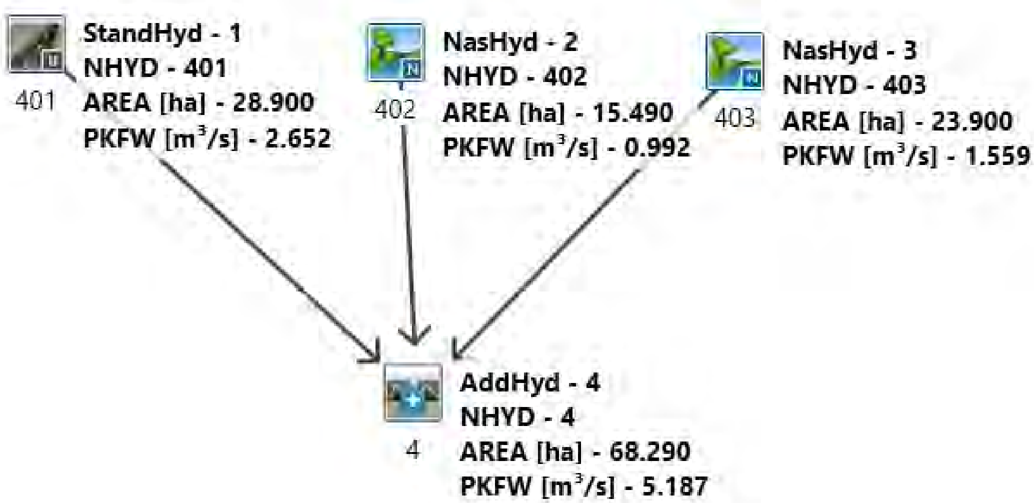
HEC-RAS Model Schematic – Pre-Development

Valdor Engineering Inc.
File: 19121
Date: March 2022



HEC-RAS Model Schematic – Post-Development

Valdor Engineering Inc.
File: 19121
Date: March 2022



VO5 Model Schematic – Pre-Development Floodplain Drainage Area