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TOWN OF CALEDON
PLANNING
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May 27, 2026

FUNCTIONAL SERVICING REPORT

Proposed Amelia Street Subdivision

West Side of Main Street, North of Queen Street
Community of Alton
Town of Caledon (21T-24001C)
Region of Peel

April 2022
Rev: February 2024
Rev: April 2026

Prepared For: **Mount Nicholas Holdings, Inc.**

File: **17121**

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VALDOR WATER RESOURCES

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1.0 INTRODUCTION

Valdor Engineering Inc. has been retained by Mount Nicholas Holdings, Inc. to provide consulting engineering services for the proposed residential subdivision located on a 5.84 hectare parcel located west of Main Street and north of Queen Street in the Alton Community of the Town of Caledon as illustrated in **Figure 1**.

1.1 Existing Conditions

The subject site is currently vacant and has rolling topography with tree cover. The site is bounded to the east by residential lots fronting Main Street and to the south by residential lots fronting Amelia Street. The site is bounded to the north and west by a forested area. The subject site has access to Amelia Street to the south as well as access to Main Street to the east via the untravelled road allowances of Spring Street and Nicholas Street.

1.2 Proposed Development

The proposed development consists of 15 residential lots for detached dwellings which will be serviced by the northly extension of Amelia Street to connect to Main Street via the untravelled road allowance of Spring Street. A large woodlot on the west side of the site will be preserved and will be contained within Lot 7.

The configuration of the proposed development is illustrated in **Figure 2**. The development statistics and the equivalent population data are summarized in **Table 1**. A copy of the draft plan of subdivision indicating the configuration of the lots is included in **Appendix "A"** together with calculation of the equivalent population.

Table 1. Development Statistics

Land Use	**Area** (Ha)	**Residential Units** (No.)
Residential Lots	5.44	15
Road Allowances & 0.30m Reserves	0.39	
TOTAL	**5.84**	**15**

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 engineering staff at the Town of Caledon, Region of

Peel and Credit Valley Conservation, as well as visits to the site. The conceptual design is documented on a series of large size preliminary plans which are contained in a pocket at the rear of this report. This document provides guidance for detailed engineering design of the subdivision.

2.0 WATER SERVICING

The Region of Peel is responsible for the treatment and distribution of water within the Town of Caledon as well as the City of Brampton and the City of Mississauga. The Region's South Peel Drinking Water System is a lake-based system that supplies a population of over 1.3 million people in the urban area drawing water from Lake Ontario after which it is treated at either the Lakeview or Lornepark water treatment facilities. The rural communities of Caledon including Palgrave, Caledon East, Caledon Village, Cheltenham Village, Inglewood as well as Alton are serviced by well based water systems which are operated by the Region of Peel.

The community of Alton is serviced by the Caledon Village – Alton Drinking Water System. This system consists of wells in both Caledon Village and in Alton which are interconnected to form a single system which includes a water treatment plant and service a population of approximately 4,900. The subject site is located within Water Pressure Zone AV12. 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 Region of Peel engineering design standards which includes the following parameters:

Residential Average Day Demand:	280 L/person/day
Maximum Day Factor:	2.0
Peak Hour Factor	3.0

A detailed tabulation of the domestic water demand calculation is detailed in **Table B1 o**f **Appendix "B"**. The demands are summarized in **Table 2** below.

Table 2. Domestic Water & Fire Flow Demand

Land Use	Equivalent Population (Persons)	Average Day Demand (L/min)	Maximum Day Demand (L/min)	Peak Hour Demand (L/min)	Fire Flow (L/min)	Maximum Day Plus Fire Flow (L/min)	Maximum Day Plus Fire Flow (L/s)
Detached Dwellings	60	11.7	23.3	35.0	5,000	5,023	83.7

2.2 Local Watermains & Service Connections

The local water distribution system within the subdivision will consist of watermains of 150mm diameter. This internal water system will connect to the existing 200mm diameter

watermain on the Main Street and the existing 150mm diameter watermain on Amelia Street at Nicholas Street.

Based on Ontario Building Code (OBC 2012) regulations (7.6.3.4.(1) and (5) and Table 7.6.3.4), the houses 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 Region and will be installed in the basement of each dwelling unit with a remote readout device located on the exterior ground floor wall of the unit. A copy of the standard water service connection detail are included in **Appendix "B"**. The location of the existing and proposed watermains are indicated in on the **Functional Servicing Plan**.

2.3 Fire Protection

The fire flow required for the proposed detached dwelling units and commercial buildings 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.

Based on the calculations, the minimum fire suppression flow required for the detached dwellings is 5,000 L/min. The preliminary fire flow calculation is shown in **Table B2** contained in **Appendix "B"**. In accordance with the Region of Peel standards, this flow must be available at the nearest hydrant with a minimum pressure of 140 KPa.

Fire hydrants will be provided along the road at a 150m spacing in accordance with the Region of Peel design criteria. A copy of the standard fire hydrant connection detail is included in **Appendix "B"**. The location of the existing proposed fire hydrants are indicated on the **Functional Servicing Plan**.

3.0 WASTEWATER SERVICING

The Region of Peel operates and maintains approximately 3,500 Km of sanitary sewer including those within the urban areas of Caledon as well as those in the City of Brampton and the City of Mississauga. Wastewater treatment for these urban areas is provided by the Clarkson and G.E. Booth Lakeview Wastewater Treatment facilities located on the shore of Lake Ontario. Caledon's Inglewood Community is serviced by the Inglewood Communal Wastewater Treatment Station which discharges to the Credit River. Homes in the balance of Caledon's rural communities, including Alton are serviced by individual private on-site sewage systems.

The following sections detail the sizing of the private on-site sewage systems in accordance with the Ontario Building Code.

3.1 Wastewater Design Flow

In accordance with Part 8 of the Ontario Building Code (OBC) the daily design flow for a dwelling is to be calculated based on the floor area of the dwelling, the number of bedrooms and the number of fixture units.

Although architectural house plans are not yet available, for the purpose of confirming servicing feasibility, it is assumed that the proposed detached dwellings will have four bedrooms, 278 m^2 (3,000 sq.ft) of floor area with 24.5 fixture units. Based on these parameters the design sewage flow has been calculated to be 2,800 L/day as summarized in **Table 3**. The calculation of the design flow is provided in **Table C1** and the listing of fixture units is provided in **Table C2** which are both included in **Appendix "C".**

A Preliminary Geotechnical Investigation prepared by Sirati & Partners Consultants Limited determined that the native soils across the lots consist of sand, sandy silt or silty sand. A Hydrogeological Investigation prepared by Palmer Environmental Consulting Group Inc. includes a Septic System Servicing Assessment with percolation rates. Based on this assessment the preliminary sizing of septic systems have been based on a percolation rate ("T" time) of 28 min/cm. Excerpts from the Preliminary Geotechnical Investigation and the Hydrogeological Investigation are included in **Appendix "C"**.

Table 3. Wastewater Flow

House Size:	278 m^2 (3,000 sq.ft.), 4 bedrooms, 24.5 fixture units
Design Flow, Q:	2,800 L/day
Native Soils:	Sand, Sandy Silt, Silty Sand
Percolation Rate:	28 minutes / cm
Tertiary Treatment Septic System:	
Anaerobic Digester Tank	5,664 L
Waterloo Biofilter® Tank	Model AD-7480
Area Bed: Stone Layer:	37.3 sq.m.
Sand Layer:	196.0 sq.m.

3.2 Septic System Components

In order to provide high quality treatment and reduce nitrate loading, tertiary treatment septic systems are proposed for all of the lots within the subdivision. There are several alternative treatment systems available which have been approved by the Ministry of Municipal Affairs & Housing (MMAH) and recognized in the OBC. Alternative treatment systems designed as "Treatment Units" other than septic tanks must meet the requirements of Section 8.6.2.2 of the OBC and must produce tertiary quality effluent. One

such technology providing tertiary treatment is the Waterloo Biofilter® manufactured by Waterloo Biofilter Systems Inc.

Although there are several commercially available and OBC approved tertiary treatment systems, the following is a summary of the sizing of a Waterloo Biofilter® system:

Anaerobic Digester Tank

A 100mm diameter gravity sanitary drain will convey sewage flows to the Waterloo Biofilter Anaerobic Digester tank from plumbing fixtures on the main floor of the dwelling and above. Fixtures in the basement will require an internal sewage ejector to pump flow up to the gravity sanitary drain.

In accordance with Section 8.2.2.3 of the OBC the tank volume is to be a minimum of twice the daily design flow as follows:

Minimum Tank Size =	2,800 L/day x 2 =	5,600 L
Available Anaerobic Digester Tank Size =		5,664 L

Pump Tank

Given that the effluent from the anaerobic digestor mut be pumped to the treatment unit a pump tank is required. Based on a design flow of 2,800 L/day a Waterloo Biofilter® tank model PT-3000 is required.

Tertiary Treatment Unit - Waterloo Biofilter®

The Waterloo Biofilter® is an aerobic trickling filter that uses an absorbent synthetic filter material developed by researchers at the University of Waterloo and was incorporated into the OBC in 1998. Effluent is applied intermittently over modules of plastic foam pieces (patented biofilter medium) contained in wire mesh baskets. This synthetic media is a support for microbiological growth, and these microorganisms are responsible for the aerobic breakdown of the wastewater. Approximately 50% of the effluent exiting the unit is pumped back to the septic tank, while the other half is directed to a disposal bed.

In accordance with the manufacturer's guidelines, based on a design flow of 2,800 L/day the Waterloo Biofilter® model AD-7480 is required.

Subsurface Disposal: Area Bed

An area bed dispersal system is typically utilized with alternative treatment systems for subsurface disposal. The area bed is to be comprised of a stone layer overlying a sand layer where the stone layer is to be a minimum of 250mm in depth, wrapped with a permeable geo-textile fabric, and comprised of stone meeting the requirements of either Subclause 8.7.3.3.(1)(b)(i) or (ii) of the OBC. Distribution pipes having 75mm diameter are to be spaced evenly within the stone layer with spacing not exceeding 1.2m. The sand layer is to be a minimum of 300mm in depth having a percolation rate ("T" time) of 6 to 10 min/cm. The sand

layer covers the entire area bed. The sizing of the stone layer and sand layer is as follows:

Stone Layer:

Given that the daily design flow is less than 3,000 L, the loading on the surface of the stone layer is calculated as follows:

Minimum Surface Area = Q / 75 = 2,800 / 75 = 37.3 m^2

Sand Layer:

This sand layer is to extend over an area which is calculated as follows:

Minimum Surface Area = Q x T / 400 = 2,800 x 28 / 400 = 196.0 m^2

The area bed absorption system also includes a mantle extending a minimum of 15m beyond the distribution pipe allows an additional area for the effluent to seep laterally into the native underlying soils.

A summary of the tertiary treatment sizing is provided in **Table 3**. The location of the area beds are indicated on the **Functional Servicing Plan** and the **Functional Grading Plan**. The septic bed sizing calculations are included in **Appendix "C"**.

4.0 STORM CONVEYANCE SYSTEM

The subject site is located within the Credit River watershed which encompasses approximately 860 km^2 of area. The Credit River is almost 90 km long and meanders southeast from its headwaters in Orangeville, Erin and Mono, through nine municipalities including Brampton, eventually draining into Lake Ontario at Port Credit in Mississauga.

In particular, the property is located within the Shaw's Creek sub-watershed which is one of twenty sub-watersheds of the Credit River watershed. The Shaw's Creek sub-watershed, referred to as Tributary 17 is approximately 78 sq. km in area and is located partly within the Town of Caledon, the Town of Erin, the Township of East Garafraxa, and the Town of Orangeville. All of the sub-watersheds within the Credit River watershed are under the jurisdiction of the Credit Valley Conservation Authority (CVC). A plan of the watershed is included in **Appendix "D"**.

In accordance with Town 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

In accordance with the Town's 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 Town's rainfall intensity duration frequency (IDF). Based on this data the rainfall intensity for the 5- and 100-year rainfall events is calculated as follows:

$$I_5 = \frac{1593}{(t+11)^{0.8789}} \quad I_{100} = \frac{4688}{(t+17)^{0.9624}}$$

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 IDF curve data detailed in the Town's Engineering Standards is included in **Appendix "D"**. A schematic design of the minor system is illustrated in on the **Functional Servicing Plan** (Dwg. FSP-1).

The sizing of street catchbasins, ditch inlet catchbasins and rear lot catchbasins will be completed at the detailed engineering design stage. The spacing of street catchbasins will be in accordance with Town standards.

Total capture will be provided at two locations:

- The low point on Amelia Street, just south of Main Street, will provide total capture for all drainage in the northern portion of the proposed development. Total capture will be achieved via two double catchbasins in sags. The storm sewer will be sized to convey the 100-year flow for this drainage area at detailed design.

- Due to grading constraints, it is not possible to construct a low point at the Amelia Street and Nicholas Street intersection in order to capture drainage from the majority of the site. To ensure total capture at this location, catchbasins with high-capacity grates will be installed within the gutters. These catchbasins will be connected directly to the proposed on-site SWM control (box culvert storage tank).

The design of this total capture catchbasins will be completed at the detailed design stage.

There is a large external drainage area to the south of the subject site that drains through the development area. It is proposed that external flows will by-pass the SWM controls. To achieve this, drainage from this external area will be intercepted by cut-off swales directing flow to DICBs, which will in turn convey flow to a separate storm sewer running in parallel with the storm sewer servicing the proposed development, bypassing the on-site SWM controls. The two storm sewer systems will combine at the intersection of Amelia Street and Nicholas Street and then flow will be conveyed along Nicholas Street to Main Street in the form of a concrete box culvert.

Based on the Municipal Class Environmental Assessment Environmental Study Report Final prepared by R.V. Anderson Associates Limited and the Alton Village Drainage Study, Phase 2, prepare by R.J. Burnside & Associates Limtied, flow from the subject site, and the upstream drainage area, is to be directed to the existing 750mm diameter culvert under Main Street. In this regard, the proposed box culvert along Nicholas Street will discharge to the Main Street ditch just upstream of the existing culvert as indicated on the **Functional**

Servicing Plan (Dwg FSP-1) . Due to the relatively shallow depth of the existing culvert, the proposed box culvert will have reduced cover. The final size and alignment of the box culvert will be design and the detailed subdivisdion engineering stage.

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 one of two total capture locations discussed above. The major system flow route is illustrated in **Functional Grading Plan** (Dwg FGP-1).

4.3 Foundation Drainage

It is anticipated that the proposed dwellings will have conventional basements which will have foundation weeping tile. Given the high-water level within the proposed storm sewer due to the on-site SWM controls, gravity connections will not be provided. Sump pumps will therefore be required. The sump pumps are to discharge over splash pads to grassed areas. The sump pumps are not to discharge to the septic system.

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.

5.0 FLOODPLAIN

As per the CVC regulation mapping provided in **Appendix "D"**, the subject site is not located within the Shaw's Creek floodplain. As such, no floodplain mitigation measures are required.

6.0 STORMWATER MANAGEMENT

6.1 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. As per the Town's request, references to the findings and recommendations from the Alton Village Drainage Study - Phase 2 Report prepared by R.J. Burnside & Associates Limited was made. Excerpts from this report can be found in **Appendix "E"**.

6.1.1 Pre-Development

The existing site within the project area (*Catchment 101*, 2.65 ha) consists of forested lands draining east to an existing culvert under Main Street, just north of Nicholas Street.

Site elevations within the project area vary from approximately 423.5 m at the rear of Lot 7, to approximately 404.0 m at the south-eastern limit of the site where the

Amelia Street extension will meet the existing street. The existing slopes range from approximately 7% to approximately 28%.

Figure 3 shows the pre-development condition storm drainage plan for the subject site.

6.1.2 Post-Development

The subject site will be developed into an estate residential subdivision consisting of 15 detached-home lots, a road and environmental zones.

The majority of the proposed development area (*Catchment 201*, 2.02 ha) will drain to the proposed street and will be controlled by a box culvert storage tank located under the street. The box culvert will provide quantity control and will discharge to an Oil/Grit Separator which will provide quality control. This flow will be directed to the existing 750 mm culvert under Main street and will follow the same drainage patterns as the findings of the Alton Village Drainage Study - Phase 2 Report.

The rear of Lots 1-4 and the area south of the Nicholas Street intersection (Lot 15 and part of Lot 14) will drain uncontrolled (*Catchment 202*, 0.63 ha).

Figure 4 shows the post-development condition storm drainage plan for the subject site.

6.1.3 External Areas

The external drainage area to the south of the proposed development (*Catchment 301*, 5.19 ha) will drain to cut-off swales located at the rear of Lots 5-15 and to a separate storm sewer that will bypass the box culvert storage tank. The drainage patterns for the external drainage area to the west of the proposed development (*Catchment 302*, 2.24 ha) will generally remain untouched, but will be conveyed under the Amelia Street extension via a proposed culvert with rip-rap sized for the 100 year storm for erosion protection. Further details will be provided at detailed design.

6.2 Stormwater Management Design Criteria

The proposed SWM facilities shall be designed to provide the following levels of control as per the requirements of the Ministry of the Environment (MOE), Credit Valley Conservation (CVC) and the Town of Caledon:

- **Quality Control**: Water quality control shall be provided to achieve Enhanced (Level 1) treatment of stormwater runoff (80% TSS removal).
- **Erosion Control**: Erosion control shall be provided to ensure zero runoff from the first 5 mm of precipitation over the entire site area.
- **Flood Control**: Flood storage and control shall be provided to maintain peak outflows from the site at or below pre-development levels for the 2-year through 100-year design storm events.

- **Water Balance**: Pre-development annual infiltration rates are to be maintained under post-development conditions.

6.3 Quantity Control

6.3.1 Pre-Development Condition

The development area is occupied by mostly forested areas with grassed lawn and single detached areas as depicted in **Figure 3**. Based on the various surface types the pre-development composite runoff coefficient was calculated to be 0.15. The pre-development peak flows are calculated in **Table E2** contained in **Appendix "E"** and summarized in the first row of **Table 4**.

Table 4: Summary of Peak Flow Rates

Condition	Runoff Coefficient	Peak Flows (L/s)					
		2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Pre-Development	0.15	97.6	124.9	152.7	178.1	200.6	223.7
Post-Development-Unmitigated	0.48	304.7	389.8	476.8	556.1	626.2	698.5
Post-Development-Mitigated	0.48	96.9	122.3	146.7	169.2	187.9	207.2

6.3.2 Post-Development Condition: Un-Mitigated

Based on a review of the Draft plan, the post-development storm drainage areas and composite runoff coefficients for the development area were determined and summarized in **Figure 4**. The surfaces include the impervious roofs, roads and lawn areas.

Based on these surfaces, the proposed development has a higher level of imperviousness than the pre-development condition and the post-development runoff coefficient was calculated to be 0.48. The post-development unmitigated peak flow calculation is provided in **Tables E3-A** and **F3-B** contained in **Appendix "E"** and summarized in the second row of **Table 4**.

6.3.3 Post-Development Condition: Mitigated

A comparison of the first and second rows of **Table 4** indicates that an increase in peak flows is anticipated and therefore stormwater mitigation measures are required.

In order to achieve the target discharge rates, an on-site stormwater detention system consisting of a box culvert storage tank under Amelia Street and orifice control is proposed to provide on-site detention. Ground surface detention is not required.

The stormwater quantity control was modelled using the modified rational method. This method calculates the storage volume using the composite runoff coefficient and the allowable release rate based on rainfall intensities using the Town's standard IDF data. Through an iterative process, a combination of orifice diameters, high water levels (HWL) and detention volumes were analyzed to optimize the SWM system.

Using an orifice diameter of 133 mm, the post-development 2- to 100-year peak flows can be controlled to the pre-development flows, as indicated in the third row of **Table 4**. Based on this, the required detention volume is 509.92 m^3, which is less than the provided detention volume of 513.00 m^3. The stormwater detention calculations are provided in **Table E4** to **Table E6**, all of which are contained in **Appendix "E"** together with the summary **Table E1**. The location of the detention tank and the control device are illustrated in **Functional Servicing Plan (Dwg. FSP-1**).

The type of SWM detention pipe (box culvert, circular or elliptical) will be reviewed at the detailed subdivision engineering design stage for consideration with respect to future flushing and maintenance.

6.4 Quality Control

Based on the Stormwater Management Planning & Design Manual prepared by the Ministry of the Environment (March 2003), various levels of treatment are defined with the goal to maintain or enhance existing aquatic habitat based on the total suspended solids (TSS) removal efficiency.

Based on the municipal criteria for the area, storm water quality control for the subject site is to be designed to achieve "Enhanced" protection level (Level 1 treatment) which provides 80% TSS removal. In order to provide stormwater quality control for the subject site, catch basin infiltration trenches pretreated with goss traps in conjunction with an oil / grit separator is proposed. Separators are generally implemented on relatively small sites and are typically in the form of a pre-cast concrete maintenance hole with a deep sump with a special insert which diverts low flows to a lower chamber to capture and store oil and grit from the storm drainage discharge from the site. The insert diverts high flow away from the lower chamber to ensure that captured pollutants do not scour or re-suspend.

For this application a *Stormceptor*® type oil / grit separator manufactured by Hanson Pipe and Products Inc. has been selected. This separator has been sized in accordance with the manufacturer's recommendations using the software PCSWMM for *Stormceptor*. Based on the simulation, a *Stormceptor*® Model EFO4 has been chosen which, based on the site area and imperviousness, has the capability of providing a TSS removal rate of 45%. Details of the oil grit separator sizing output and sections is provided in **Appendix "E"**. The oil / grit separator has been located downstream of the orifice control such that it receives only controlled flows in order to maximize its removal efficiency. The location of the oil / grit separator is illustrated in **Functional Servicing Plan (Dwg. FSP-1**).

The infiltration tanks will provide a TSS removal rate of 70%. With a treatment train approach, a total TSS removal of 83.5% is achieved, which meets the water quality criteria of 80%. Detailed calculations can be found in **Table E.7** in **Appendix "E"**.

6.5 Erosion Control

As per the requirements of the Credit Valley Conservation Authority, the proposed site is to ensure zero runoff from the first 5 mm of precipitation over the entire site area in order to mitigate potential erosion downstream. As calculated in **Table F1** in **Appendix "F"**, a minimum storage volume of 44 m^3 is required.

An infiltration volume of 44.40 m^3 will be provided in seven infiltration trenches located underneath Amelia Street. These infiltration trench is discussed in further detail in *Section 7.4*. The erosion control target for the site is therefore being achieved.

6.6 SWM Inspection & Maintenance

The proposed stormwater management system requires regular inspection and maintenance to ensure the infrastructure function as designed. The suggested inspection and maintenance procedures are as follows:

Catchbasins & CB Manholes:	Keep catchbasin grates clear of debris. Inspect silt depth in sumps regularly. Clean sump when reaches 50% full.
Orifice Device:	Inspect orifice devices periodically. Ensure orifice device clear of blockage. Clean sump when accumulates over 50% of debris.
Oil Grit Separator:	Inspect and maintain the oil / grit separator unit in accordance with the owner's manual which will be provided by the manufacturer upon completion of the installation.
Infiltration Trench:	The infiltration trench will receive clean runoff from the rear lawn areas and the external forested area to the south, with little anticipated sediment accumulation. In order to ensure the proper functioning of the DICBs located to either end of the infiltration trench, they should be periodically inspected to ensure they are clear of debris.

Maintenance should be performed immediately if an abnormality is observed such as slow catchbasin draining rate.

7.0 SITE WATER BALANCE

In accordance with the requirements of the Credit Valley Conservation, an annual site water balance assessment was completed by Palmer 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 (*Hydrogeological Investigation; Palmer*, January 2022). Excerpts from the *Hydrogeological Investigation Report* regarding the water balance analysis (*Section 4 Water Balance*) are included in **Appendix "F"**. The findings of the *Hydrogeological Assessment* water balance analysis are summarized below. Refer to **Appendix "F"** for the detailed water balance calculations.

7.1 Pre-Development Infiltration Volume

For the pre-development condition, a total estimated infiltration volume of 14,181 m³/year was determined.

7.2 Post-Development Infiltration Volume (Unmitigated)

For the post-development condition, without infiltration enhancements, a total estimated infiltration of 12,706 m³/year was determined. This corresponds to a total infiltration deficit of 1,475 m³/year, a 10% decrease in annual infiltration compared to the pre-development condition.

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 an infiltration trench. The design of these LID measures are presented below.

7.3 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 AB, 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 3.3.1 – Hydraulic Conductivity* Testing and Infiltration Rate Estimates (*Hydrogeological Investigation; Palmer*, January 2022) the infiltration rate varies between 30 and 76 mm/hr, the infiltration rate of the soils are considered acceptable. Assuming the lowest measured infiltration rate of 30 mm/hr to be conservative, the infiltration rate of the soils are considered acceptable.

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

  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.

  As per the draft plan, all lots have an average minimum roof area of 140 m^2. There are a total of 15 single detached residential lots, each with an applicable roof area of 100 m^2, for a total applicable roof area of 1,500 m^2 (0.15 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 901.6 mm/yr. The total roof area applicable to roof downspout disconnection infiltration is 0.15 ha. The precipitation to applicable roof areas is therefore 1,352 m^3/yr (0.15 ha x 901.6 mm/yr = 1,352 m^3/yr). Based on an evaporation rate of 10% of incident precipitation for impervious areas, as per the *Geotechnical Investigation Report*, the surplus for the applicable roof area is therefore 1,217 m^3/yr (1,352 m^3/yr x (1 – 0.1) = 1,217 m^3/yr).

Assuming an infiltration rate of 50%, the infiltration volume due to roof downspout disconnections is therefore 609 m^3/yr (1,217 m^3/yr x 0.50 = 609 m^3/yr).

7.4 Proposed Infiltration BMP – Infiltration Trenches

Roof downspout disconnection will account for an annual infiltration volume of 609 m^3/yr. The remaining deficit of 866 m^3/yr (1,475 m^3/yr – 609 m^3/yr = 866 m^3/yr) will be infiltrated through the installation of infiltration trenches underneath the street. The infiltration trenches wil be comprised of a trench lined with filer fabric and filled with clear stone. The voids between the stone will provide the required storage volume for the water that is to be infiltrated. The street CB infiltration trenches will be located beneath the street and will be supplied by runoff that will have been pre-treated by goss traps installed in the CBs.

With regards to the groundwater table, the elevations were measured in monitoring wells that were drilled in conjunction with the Hydrogeological Investigation prepared by Palmer. dated January 24, 2022. Based on this data the bottom of the proposed infiltration trenches will be at least 1.0 m above the groundwater level.

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 0.40. **Tables F.3** provides a preliminary sizing of the CB infiltration trenches underneath the street, and further details will be provided at detailed design.

With the implementation of the proposed infiltration trenches, an additional infiltration capacity of 1,360 m^3/yr is achieved.

7.5 Post-Development Infiltration Volume (Mitigated)

With the inclusion of the roof downspout disconnection and infiltration trench LID measures, the total post-development infiltration volume will be 14,675 m^3/yr (12,706 m^3/yr + 609 m^3/yr + 1360 m^3/yr = 14,675 m^3/yr), which is 103.4% of the annual pre-development infiltration volume (14,181 m^3/yr). The pre-development infiltration volumes will therefore be maintained under post-development conditions. The location of the infiltration trenches is indicated in **Figure 5** and a preliminary infiltration detail is provided in **Figure 6**.

8.0 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.

8.1 Municipal Roads

Vehicular access to the subject site will be provided by road connection to Amelia Street, Main Street and Nicholas Street which are all under the jurisdiction of the Town of Caledon.

The proposed municipal road will be constructed to urban standards which includes an asphalt pavement, crowned with 2% cross fall and edged with concrete curb and gutter. The proposed 20.0m wide road allowance will facilate an 8.7m wide pavement in accordance with the Town's standard urban cross section. The longitudinal slope of the road will generally be 0.75% with some length of road ranging up to 6.00% slope. A copy of the Town's standard road cross section is included in **Appendix "G"**.

8.2 Driveways

Each dwelling will have an attached garage and driveway. It is anticipated that the slope of driveways is to be within the range of 2.0% to 6.0% in accordance with Town standards and will be designed at the in conjunction with the individual site grading plans at the building permit stage.

9.0 GRADING

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

9.1 Grading Criteria

The subject site is to be graded in accordance with the Town grading criterion which dictates that road grades are to range from 0.75% to 6.0% (8.0% for 18.5m ROW) and that sodded yard areas are to range from 2.0% to 5.0%. For large grade differentials, a maximum slope 4H : 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.

9.2 Preliminary Design

Based on the topographic survey, the proposed subdivision configuration and the Town's criteria, a preliminary grading design has been prepared and is provided on the **Functional Grading Plan** (Dwg FGP-1). The preliminary grading design, considered the following factors:

- Achieve the Town's lot grading criteria.
- Meet the Town's vertical road design parameters.

- Minimize the depth of cut and of fill.
- Minimize the requirement for retaining walls.
- Match existing grades along the adjacent properties and road allowances.
- Provide an overland flow route to direct drainage to a safe outlet.
- Provide sufficient cover over the storm sewer.
- Match the existing grade along the limit of the woodland buffer.

Due to the existing topography, retaining walls will be required within the Town's existing road allowances for Spring Street, Amelia Street and Nicholas Street. These retaining walls can be constructed using systems that do not require tie backs. No retaining walls are required within the proposed road allowance within the subject site. We have added cross sections to the Functional Grading Plan to illustrate the extent of the retaining walls. It may be possible to reduce / eliminate some of the retaining walls if 4:1 slopes can be incorporated within the boulevards. The grading design will be refined at the detailed subdivision engineering design stage.

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 in an effort to achieve a balance. Based on the preliminary design, no significant difficulties are anticipated in achieving the municipal grading design standards or an earthworks balance.

9.3 Permitting

A review of the Regulation Mapping indicates that the subject site is not located within an area that is regulated by the CVC, however, this will be confirmed at the detailed engineering design stage in conjunction with the finalization of the storm drainage and stormwater management design.

10.0 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. It is of particular concern for this site given the proximity of the site to a watercourse.

The impact of construction on the environment is recognized by the Greater Golden Horseshoe Area Conservation Authorities. Their document titled Erosion & Sediment Control Guidelines for Urban Construction (ESC Guideline) 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:

10.1 Control Measures

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

- **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, 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.

The proposed sediment controls are illustrated on the **Preliminary Erosion & Sediment Control Plan (Dwg ESC-1)**.

10.2 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 silt fencing around the limits of grading prior to topsoil stripping.
3. Strip topsoil from the lots and road allowance, stockpile where indicated and install silt fence around the perimeter.
4. Rough grade the road allowance 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 and install sediment controls at street catchbasins and ditch inlet catchbasins.
3. Construct the road including granular base, base curb and base courase asphalt.

Stage 3 – House Construction

1. Excavate for house foundations, backfill and frame the dwellings.
2. Once the houses are cladded, install the septic systems.
3. Pave the driveway and stabilize the lot with topsoil and sod.
4. Remove silt fencing on a phased basis as areas are stabilized.

10.3 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.

11.0 UTILITIES

While some external upgrades may be necessary by the utility providers, it is anticipated that utilities such as hydro (Hydro One Networks Inc.), natural gas (Enbridge Gas Distribution Inc.), cable television (Rogers Cable Inc.), and telephone service (Bell Canada) will be available to service the subject development. As per standard practice in subdivisions, utilities will be installed underground. Co-ordination with the local hydro authority 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 as outlined in the Town's Design Standards. 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.

12.0 SUMMARY

Based on the analysis contained herein, the proposed residential subdivision can be adequately serviced (watermain, wastewater and storm) in accordance with the standards of the Town of Caledon, Region of Peel and the Credit Valley Conservation Authority design criteria as follows:

Water

- The community of Alton is serviced by the Caledon Village – Alton Drinking Water System which is owned and operated by the Region of Peel. This system consists of several wells, a water treatment plant, watermains and fire hydrants.
- The subject site will be serviced by a looped watermain having a connection to the existing 200mm watermain on Main Street and a connection to the the existing 150mm diameter watermain on Amelia Street at Nicholas Street.
- The proposed development will have a fire flow plus maximum day demand of 83.7 L/s.

Waste Water

- There are no municipal sanitary sewers in the community of Alton and therefore each lot within the development will be serviced by a private on-site sewage system.
- In order to provide high quality treatment and reduce nitrate loading, tertiary treatment septic systems are proposed for all of the lots within the subdivision.

Storm Drainage

- The subject site is located within the Shaw's Creek sub-watershed which is located in the Credit River watershed.
- In accordance with Town criteria, the subject site will be serviced by minor system comprised of a municipal storm sewer sized for the 5-year storm event.
- The major system will be comprised of an overland flow route which will convey the development's runoff from rainfall events in excess of the capacity of the municipal storm sewer to the 100-year total capture locations.

Floodplain

- Based on a review of CVC mapping, the subject site is not located within the flood plain of Shaw's Creek and therefore no flood proofing measures are required.

Stormwater Management

- In order to address the stormwater management criteria, quantity, quality, water balance and erosion control measures will be implemented in the form of a stormwater detention tank, and oil/grit separator and an infiltration trench, as follows:
- Enhance (Level 1) quality control is being achieve for the site, with a TSS removal of 83.5% for the site development area using a treatment train approach involving infiltration trenches in conjunction with an OGS.
- The 5 mm runoff volume will be infiltrated within infiltration trenches. The erosion control target for the site is therefore achieved.

- Quantity control is provided for the 2- through 100-year storm events to meet site pre-development flow targets. Runoff storage is provided by a stormwater detention tank. Discharge from the tank is controlled by an orifice plate discharging to the storm sewer.
- 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 roof top disconnections and infiltration trenches.

Vehicular Access

- Vehicular access to the subject site will be provided by road connection to Amelia Street, Main Street and Nicholas Street which are all under the jurisdiction of the Town of Caledon.
- The proposed 20.0m wide road allowance will facilate an 8.7m wide pavement with curbs in accordance with the Town's standard urban cross section.

Grading

- As is typical with subdivision development, earthmoving will be required to achieve the proposed subdivision grading necessary to meet the criteria of the Town. A detailed analysis of the earthworks will be conducted at the detailed design stage to optimize the cut and fill volumes. Based on the preliminary grading design, given the rolling topography of the site, some 3:1 slopes and retaining walls will be required to provide the necessary grading transitions, however, it is anticipated that the Town's municipal grading design standards can be acheived.
- A review of the Regulation Mapping indicates that the subject site is not located within an area that is regulated by the CVC, however, this will be confirmed at the detailed engineering design stage in conjunction with the finalization of the storm drainage and stormwater management design.

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 (2019). The ESC plans are to be prepared at the detailed engineering design stage and are to reflect the various construction stages.

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.

13.0 REFERENCES & BIBLIOGRAPHY

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- Sirati & Partners Consultants Limited, **Preliminary Geotechnical Investigation**, Sept 6, 2017.
- Palmer Environmental Consulting Group Inc., **Hydrogeological Impact Study**, January 24, 2022.
- R.V. Anderson Associates Limited, **Village of Alton – Main Street North & Queen Street West Municipal Class EA Environmental Study Report Final**, February 17, 2022.
- R.J. Burnside & Associates Limtied, **Alton Village Drainage Study, Phase 2**, December 2019.

Respectfully Submitted,

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Senior Project Manager

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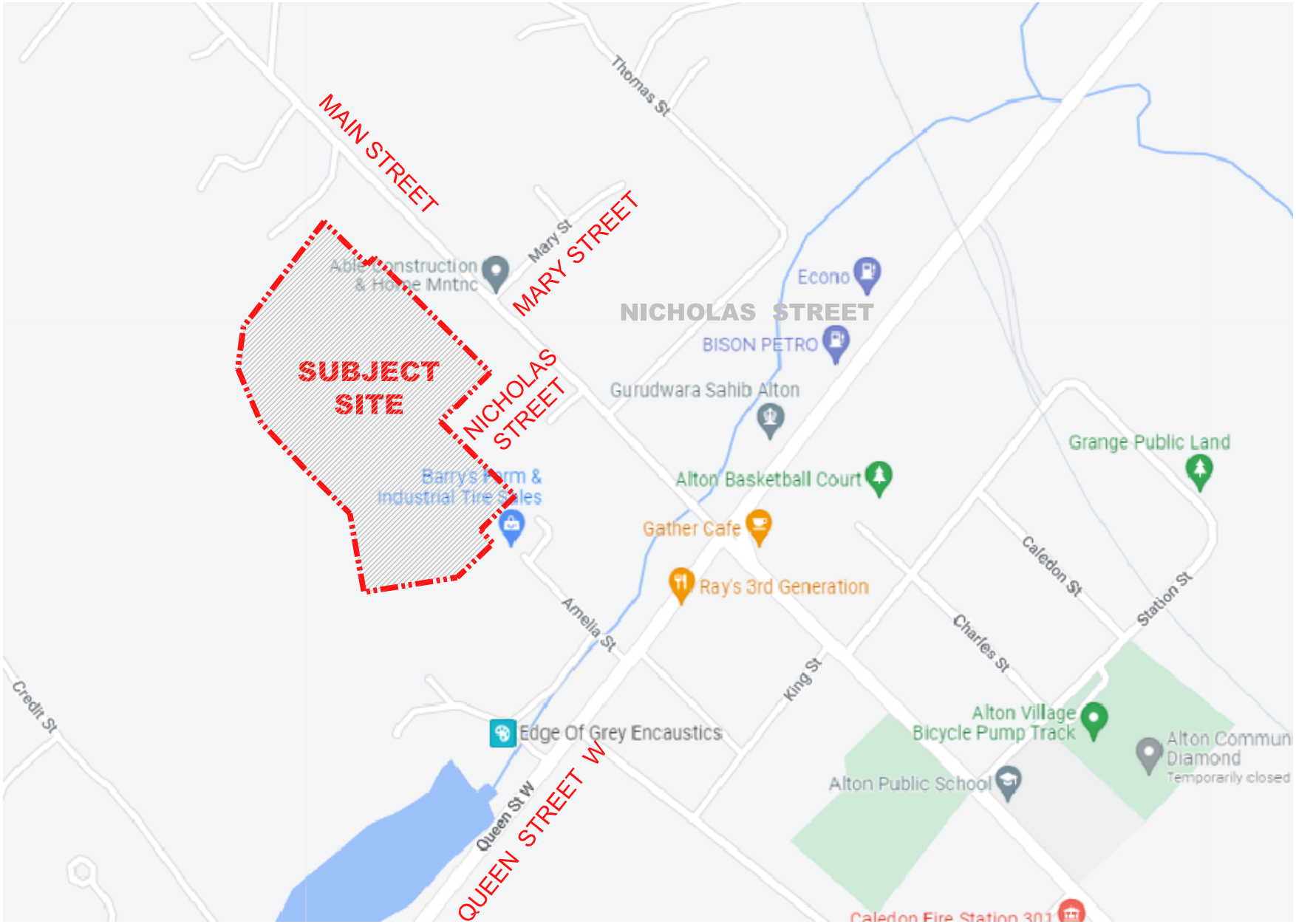

LEGEND

PROPERTY BOUNDARY

AMELIA STREET SUBDIVISION

TOWN OF CALEDON
REGION OF PEEL

PROPOSED SITE LOCATION

VALDOR ENGINEERING INC.
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SCALE	N.T.S.	CKD. BY	D.A.G.	DWG.	FIGURE 1
DATE	FRBRUARY 2024	DRAWN BY	D.N.Z.	PROJECT	171221

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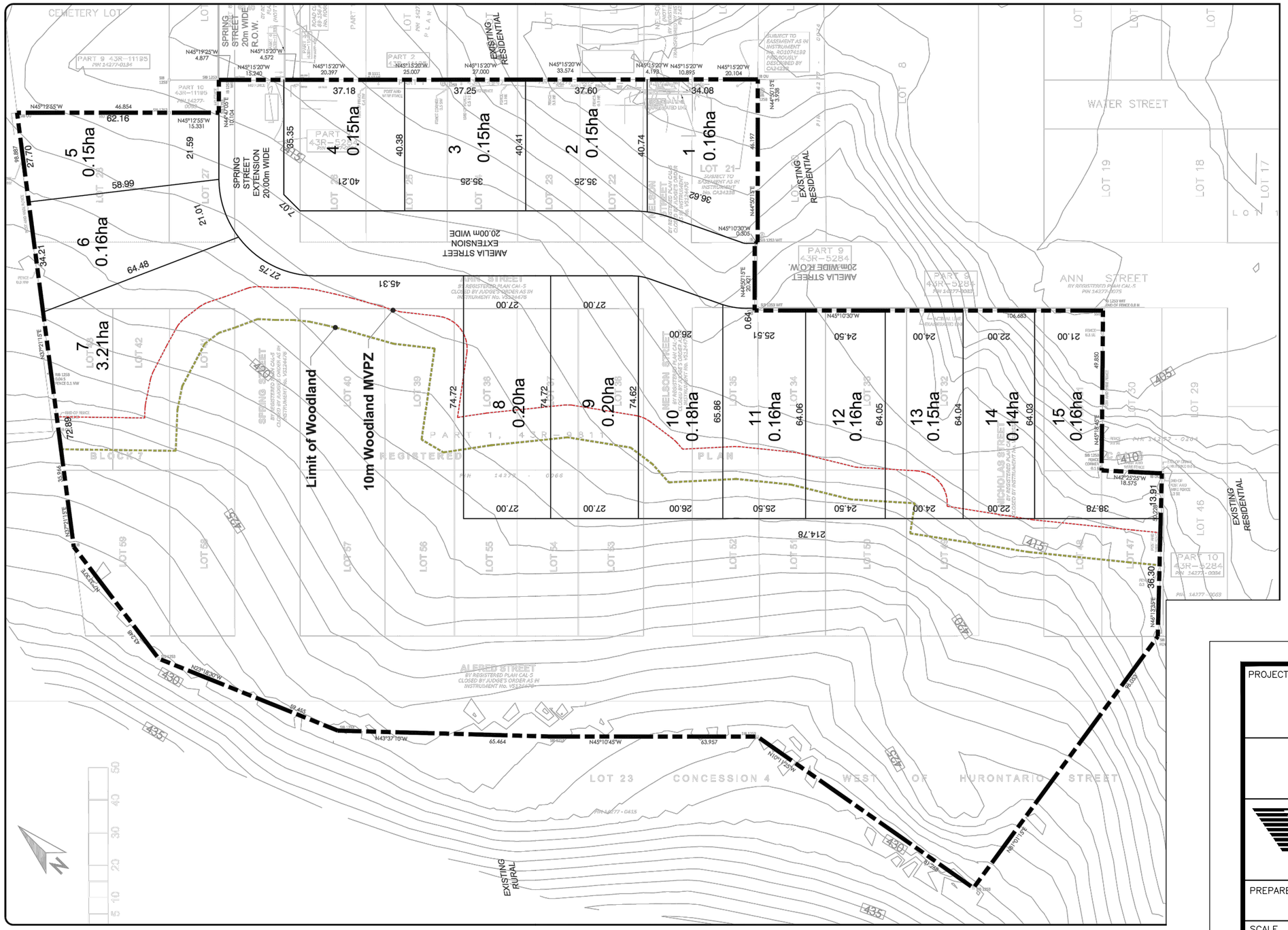

PROJECT

AMELIA STREET SUBDIVISION

TOWN OF CALEDON, REGION OF PEEL

DRAFT PLAN

VALDOR ENGINEERING INC.
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PREPARED BY D.N.Z.	CKD. BY D.A.G.
SCALE 1:2000	DATE FEBRUARY 2024
PROJECT 17121	DWG. FIGURE 2

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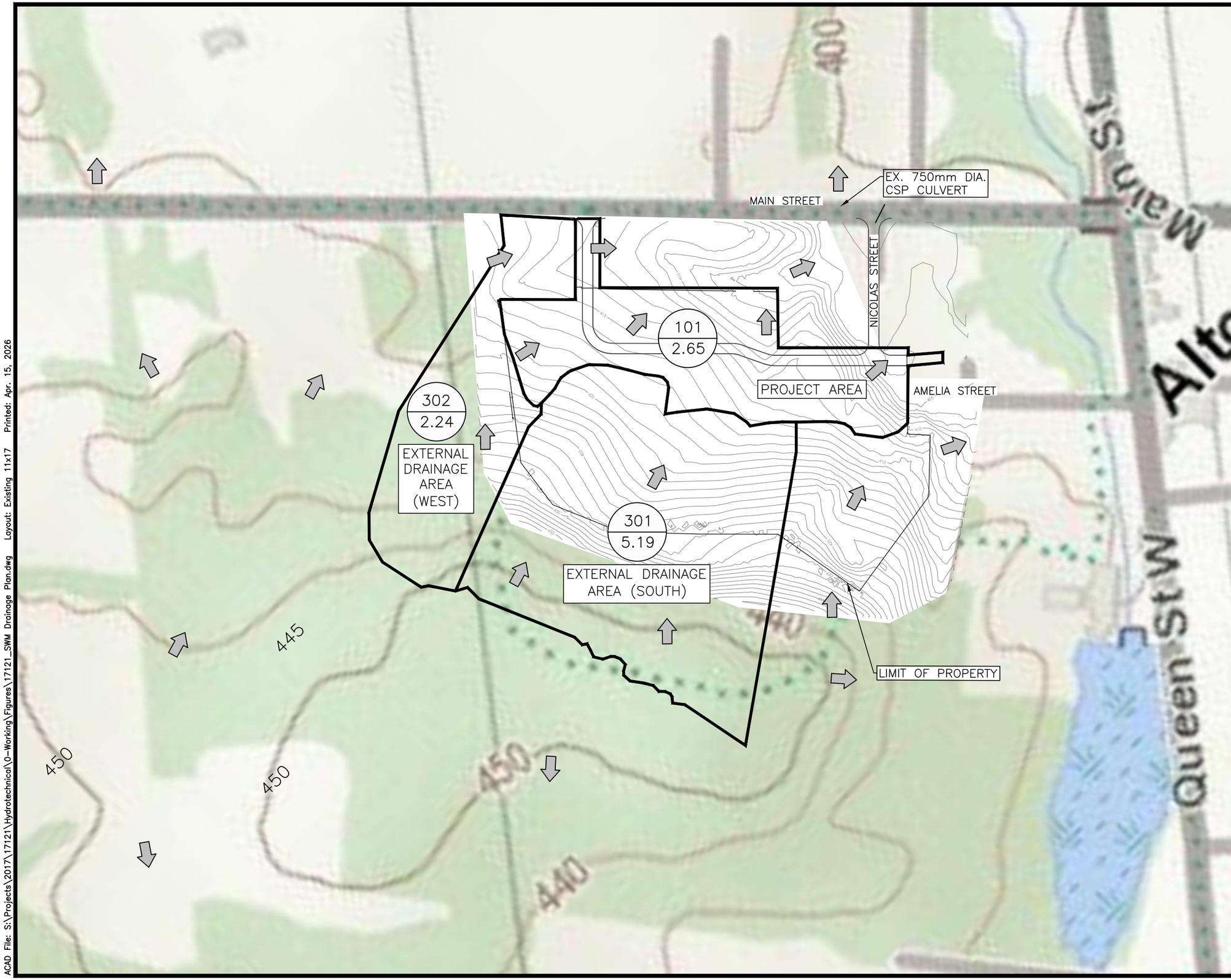

LEGEND

101 — CATCHMENT ID
10.00 — AREA (HA)

DRAINAGE BOUNDARY

OVERLAND FLOW

PROJECT
AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
REGION OF PEEL

TITLE
SWM DRAINAGE PLAN
PRE-DEVELOPMENT

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PREPARED BY P.Y.	CHECKED BY D.G.
SCALE 1:1500	DATE APR. 2026
PROJECT 17121	**FIGURE 3**

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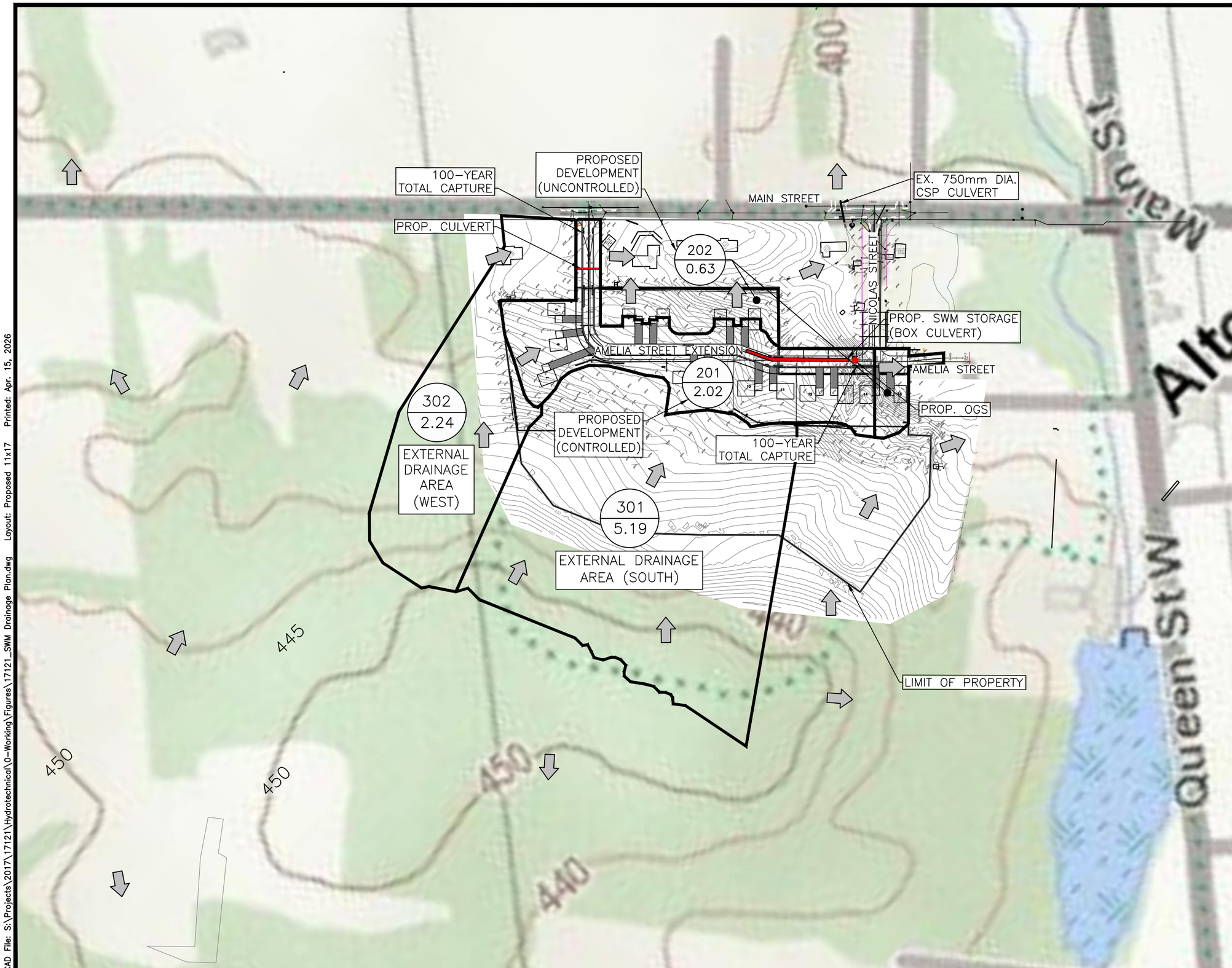

LEGEND

201 / 10.00	CATCHMENT ID / AREA (HA)
	DRAINAGE BOUNDARY
	OVERLAND FLOW

PROJECT
AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
REGION OF PEEL

TITLE
SWM DRAINAGE PLAN POST-DEVELOPMENT

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PREPARED BY P.Y.	CHECKED BY D.G.
SCALE 1:1500	DATE APR. 2026
PROJECT 17121	**FIGURE 4**

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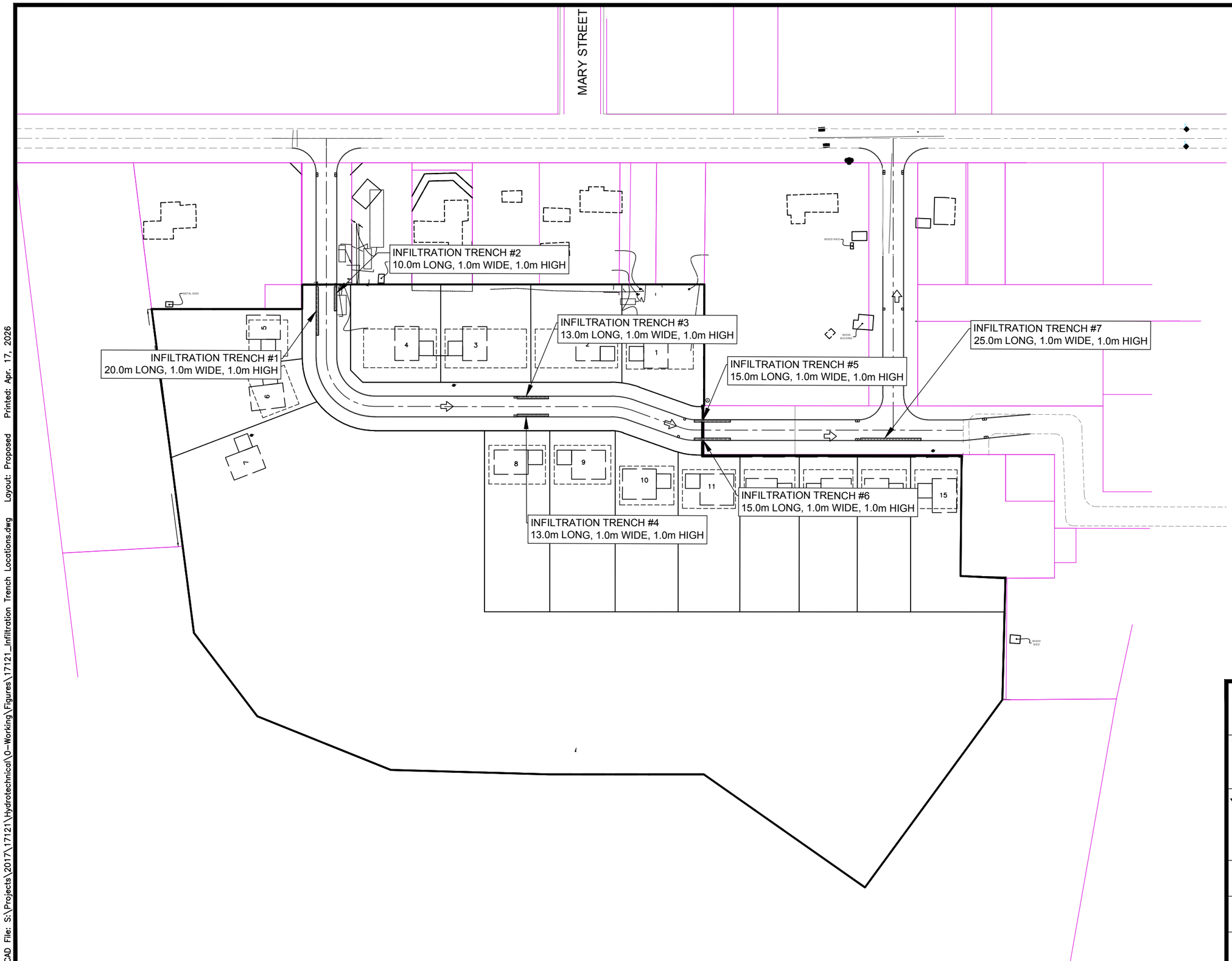

LEGEND

201	CATCHMENT ID
10.00	AREA (HA)

DRAINAGE BOUNDARY

OVERLAND FLOW

PROJECT
AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
REGION OF PEEL

TITLE
INFILTRATION TRENCH LOCATIONS

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PREPARED BY P.Y.	CHECKED BY D.G.
SCALE 1:1500	DATE APR. 2026
PROJECT 17121	**FIGURE 5**

ACAD File: S:\Projects\2017\17121\Hydrotechnical\0-Working\Figures\17121_Infiltration Trench Locations.dwg Layout: Proposed Printed: Apr. 17, 2026
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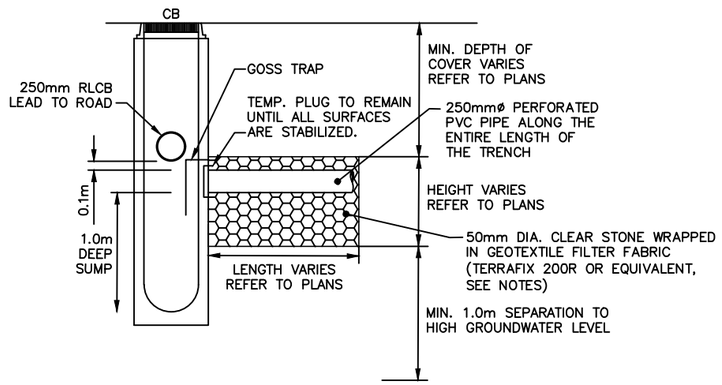

TRENCH PROFILE
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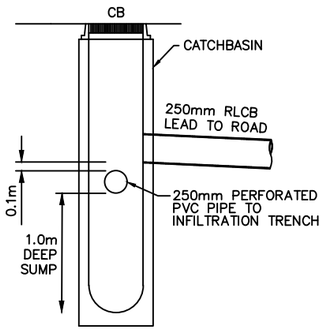

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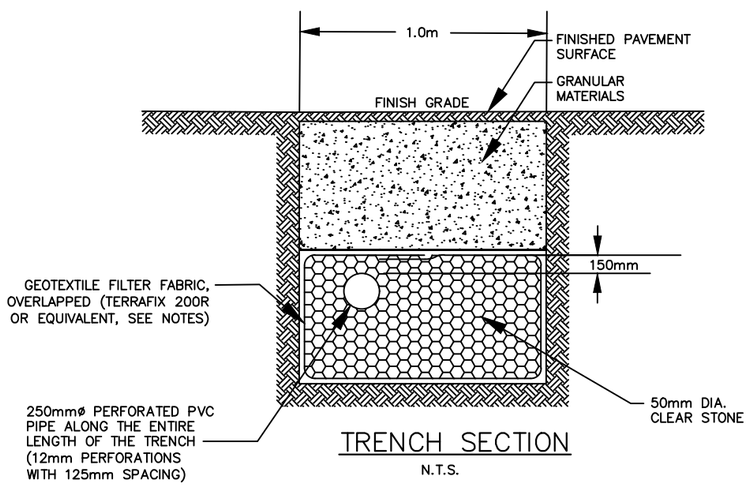

TRENCH SECTION
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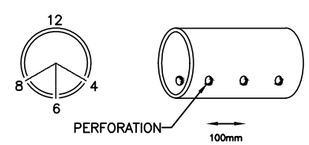


250mm PERFORATED PVC PIPE (SDR-35).
25mm PERFORATIONS AT APPROXIMATELY 4, 6, AND 8 O'CLOCK POSITIONS WHEN LAID.

PERFORATED PIPE DETAIL
N.T.S.

CB INFILTRATION TRENCH DETAILS

N.T.S.

LEGEND

201 — CATCHMENT ID
10.00 — AREA (HA)

DRAINAGE BOUNDARY

OVERLAND FLOW

PROJECT
AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
REGION OF PEEL

TITLE
INFILTRATION TRENCH DETAIL

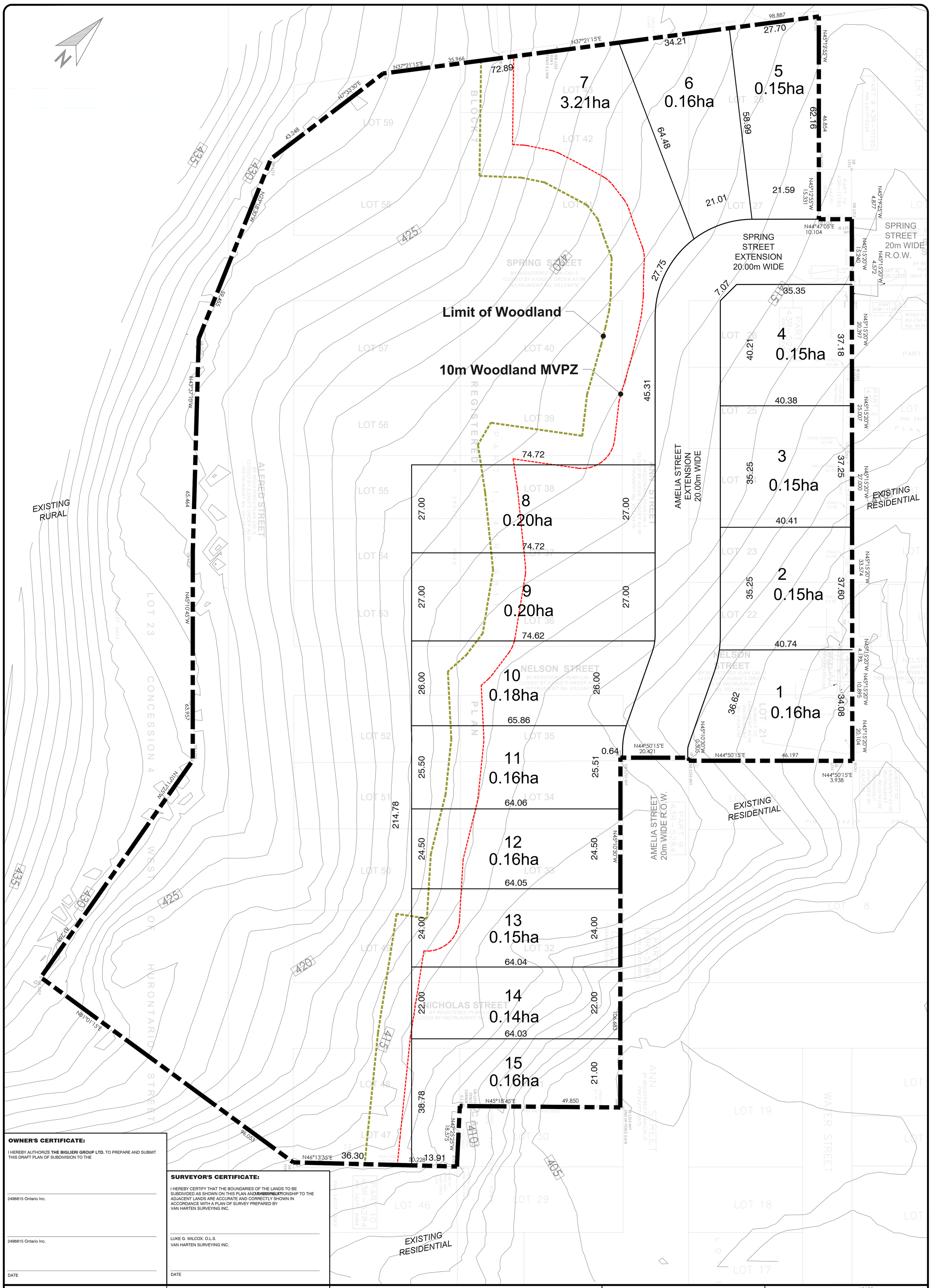
VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
571 Chrislea Road, Unit 4, 2nd Floor Vaughan, Ontario, L4L 8A2
TEL (905)264-0054, FAX (905)264-0069
E-MAIL: info@valdor-engineering.com
www.valdor-engineering.com

PREPARED BY	CHECKED BY
P.Y.	D.G.
SCALE 1:1500	DATE APR. 2026
PROJECT 17121	**FIGURE 6**

ACAD File: S:\Projects\2017\17121\Hydrotechnical\0-Working\Figures\17121_Infiltration Trench Locations.dwg Layout: Infiltration trench detail Printed: Apr. 15, 2026

APPENDIX “A”

Draft Plan & Equivalent Population Calculation



OWNER'S CERTIFICATE:

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

2496815 Ontario Inc.

2496815 Ontario Inc.

DATE

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 VAN HARTEN SURVEYING INC.

LUKE G. WILCOX. O.L.S.
VAN HARTEN SURVEYING INC.

DATE

TITLE:

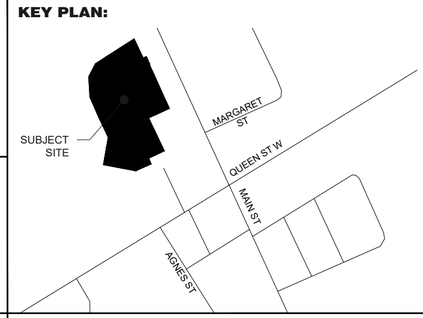
DRAFT PLAN OF SUBDIVISION

LEGAL DESCRIPTION:

PLAN OF SURVEY OF
PART LOT 23, CONCESSION 4,
WEST OF HURONTARIO STREET,
(GEOGRAPHIC TOWNSHIP OF CALEDON)
AND
PART OF LOTS 21, 22, 23, 24, 25, 26, 27 31, 32, 33, 34, 35, 47 & 59,
ALL OF LOTS 28, 36, 37, 38, 39, 40, 41, 42, 43, 48, 49, 50, 51, 52, 53, 54, 55, 56, 57 & 58, AND
PARTS OF SPRING STREET,
NELSON STREET,
NICHOLAS STREET,
ALFRED STREET AND
ANN STREET
BLOCK 7,
REGISTERED PLAN CAL-5
TOWN OF CALEDON
REGIONAL MUNICIPALITY OF PEEL

ALTON VILLAGE
CALEDON, ONTARIO

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) MUNICIPAL WATER AND SEWAGE AVAILABLE
(i) TOPSOIL, FILL, SAND, SANDY SILT, SILTY SAND, SILT, SAND AND GRAVEL
(j) SEE PLAN
(k) MUNICIPAL WATER AND SEWAGE AVAILABLE
(l) SEE PLAN
NOTE: CONTOURS RELATE TO CANADIAN GEODETIC DATUM

Schedule of Land Use

Description	Lot / Block No.	Residential Units	Area (ha)
Single Detached Residential			
Min, Lot Are 0.14ha	1-15	15	5.48
ROW	AMELIA STREET EXTENSION		0.40
Total		15	5.88

APPROVAL STAMP

REVISIONS

No.	Description	Date	Int.
3			
2			
1			

PROJECT No.:	16420
DATE:	March 7, 2024
SCALE:	1:500
DRAFTED BY: EC	CHECKED BY:
DRAWING No.:	**DP-01**

BIGLIERI GROUP

2472 Kingston Rd, Scarborough
21 King Street W, Suite 1502, Hamilton
(416) 693-9155
thebiglierigroup.com

VALDOR ENGINEERING INC.
571 Chrislea Road, Unit 4, Woodbridge, ON, L4L 8A2
Tel: 905-264-0054 Fax: 905-264-0069 info@valdor-engineering.com
www.valdor-engineering.com

TABLE: A1

EQUIVALENT POPULATION CALCULATION

Project Name: **Amelia Street Extension, Alton, Town of Caledon**
File: 17121
Date: March 2026

Land Use	Density	Number of Units	Equivalent Population
Detached Dwellings	4.2 persons per unit	15	63.0
		Total Equivalent Population (persons):	**63**

APPENDIX “B”

Water Demand Details & Calculations

VALDOR ENGINEERING INC.
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www.valdor-engineering.com

TABLE: B1

WATER DEMAND CALCULATION

Project Name: Amelia Street Subdivision, Alton, Town of Caledon
File: 17121
Date: March 2026

Demand Critera:

	Base Demand		Peaking Factors	
Residential	280	L/capita/day	Max Day	2.00
			Peak Hour	3.00

	Equivalent Population	Average Day (L/day)	Average Day (L/min)	Max Day (L/min)	Peak Hour (L/min)
Detached Dwellings	63	17,640	12.3	24.5	36.8

VALDOR ENGINEERING INC.
571 Chrislea Road, Unit 4, Woodbridge, ON, L4L 8A2
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TABLE: B2

REQUIRED FIRE FLOW CALCULATION

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

Project Name: Amelia Street Subdivision, Alton, Town of Caledon
File: 17121
Date: April 2022

Notes: 3,000 sq.ft. Detached Dwelling

Type of Construction - Ordinary Construction

C = 1.0

Total Floor Area:	278.7	sq.m
A =	279	sq.m

(Total Floor Area includes all storeys, but excludes basements at least 50 percent below grade)

$F = 220\, C \sqrt{A}$

F = 3,673 L/min

F = 4,000 (to nearest 1,000 Lmin)

Occupancy Factor

		Charge
Type:	Limited Combustible	-15%
	f_1 =	-15%

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%

$F' = F \times (1+f_1) \times (1+f_2)$

F' = 3,400 L/min

Exposure Factor

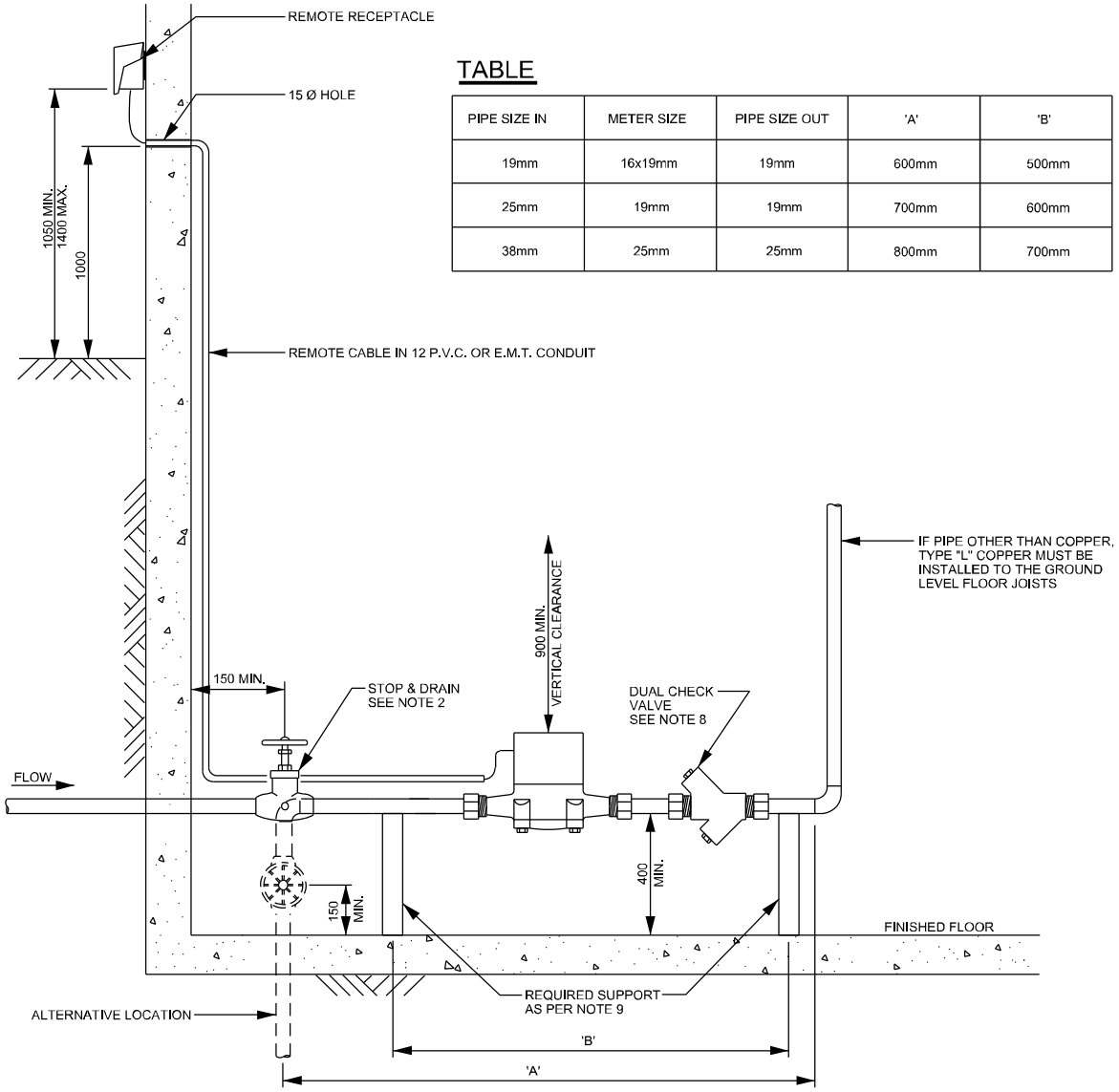
		Charge	
Side 1 - Distance to Building (m):	30.1 to 45m	5%	Front
Side 2 - Distance to Building (m):	30.1 to 45m	5%	Rear
Side 3 - Distance to Building (m):	3.1 to 10m	20%	Right Site
Side 4 - Distance to Building (m):	3.1 to 10m	20%	Left Site
	f_3 =	50%	(maximum of 75%)

$F'' = F' \times (1+f_3)$

F'' = 5,100 L/min

REQUIRED FIRE FLOW

F'' = **5,000** **L/min** (to nearest 1,000 L/min)



TABLE

PIPE SIZE IN	METER SIZE	PIPE SIZE OUT	'A'	'B'
19mm	16x19mm	19mm	600mm	500mm
25mm	19mm	19mm	700mm	600mm
38mm	25mm	25mm	800mm	700mm

NOTE

1. 175mm MINIMUM CLEARANCE BETWEEN WALL AND C/L OF PIPE AND 75mm HORIZONTAL CLEARANCE BETWEEN WALL AND METER.
2. STOP AND DRAIN TO BE THE SAME SIZE AS INCOMING PIPE.
3. IF HOT WATER TANK IS WITHIN 3.0m OF METER, THEN A CHECK VALVE IS REQUIRED BETWEEN METER AND HOT WATER TANK.
4. ALL COPPER PIPING AFTER THE STOP AND DRAIN SHALL BE TYPE "L" COPPER.
   PIPING FOR METER TO BE RUN HORIZONTALLY & METER TO BE INSTALLED ON HORIZONTAL PIPING ONLY.
5. WHERE THE INCOMING PIPE IS OTHER THAN COPPER, 500mm OF HORIZONTAL TYPE "L" COPPER PIPE (AS PER ABOVE TABLE).
6. METER SIZE TO BE ONE PIPE SIZE SMALLER THAN INCOMING SERVICE SIZE.
7. METERS MUST NOT BE LOCATED BEHIND FURNACES, WATER TANKS, ETC.
8. WHERE REQUIRED, DUAL CHECK VALVE BACKFLOW PREVENTER IS TO BE INSTALLED DOWNSTREAM OF THE METER
9. IF PLUMBING RISER/WATER SERVICE IS PLASTIC, SUPPORTS SHALL BE REQUIRED FOR METER ASSEMBLY AREA.
   TO AVOID INSTALLING SUPPORTS, RISER SHALL BE COPPER AND ATTACHED TO LOWER FLOOR LEVEL JOISTS

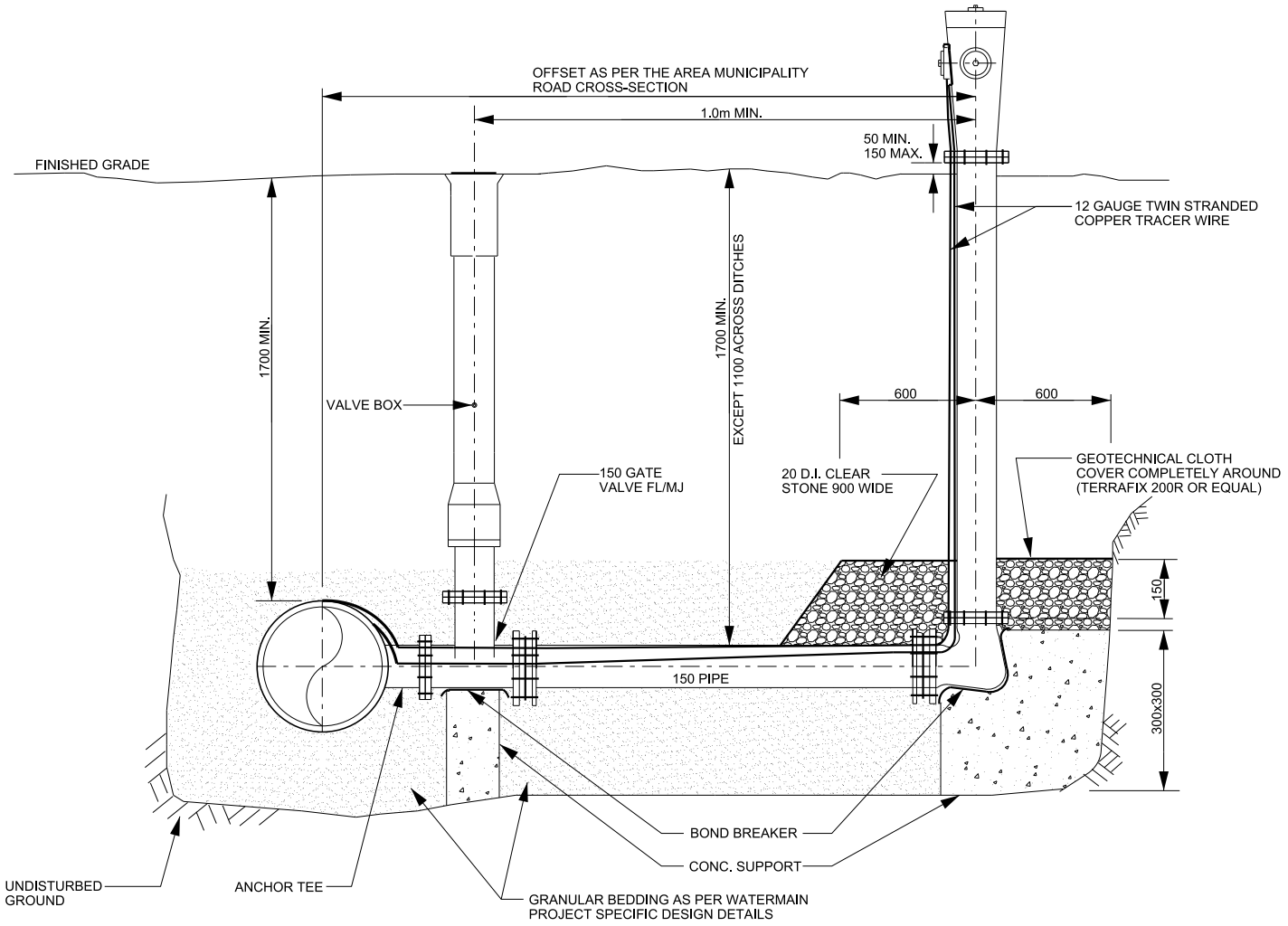
Region of Peel
Working for you

PUBLIC WORKS
STANDARD DRAWING

REV. DATE: MARCH 2017

APPROVED BY	DRAWN BY
J.K.	AINLEY GROUP - RDP
STD. DWG. NUMBER	SCALE
1-4-7	N.T.S.

16mm x 19mm TO 25mm METER INSTALLATION
FOR 25mm 'K' COPPER WATER SERVICE IN BUILDING

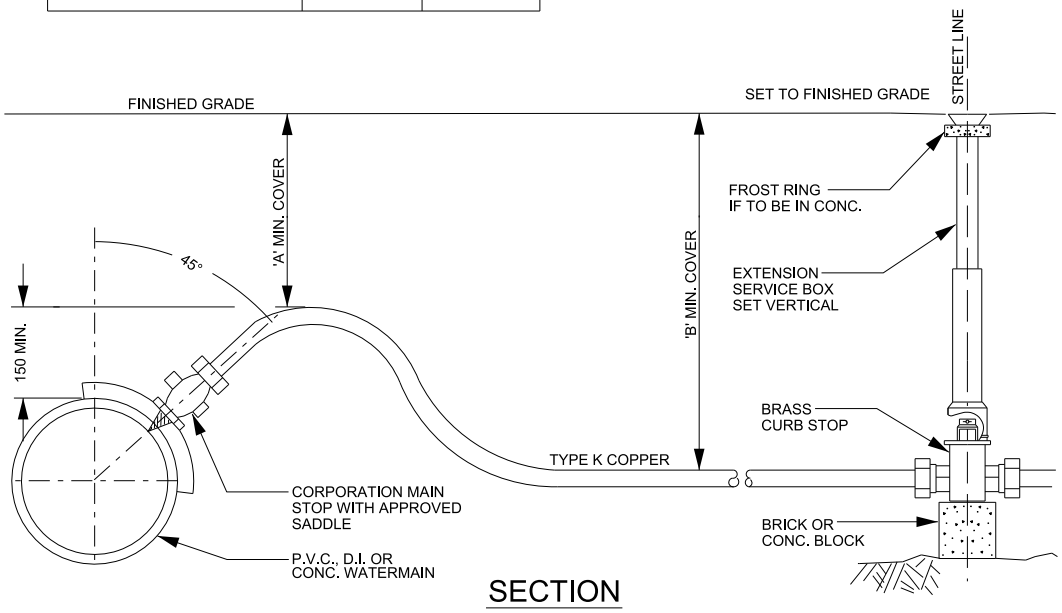


NOTE

1. IF THE WATERMAIN IS NOT AT THE STANDARD OFFSET, THE LOCATION OF THE HYDRANT TO BE AS PER CONSTRUCTION DRAWINGS.
2. BACKFILL TRENCH WITH GRANULAR 'B' COMPACTED TO 100% STD. PROCTOR DENSITY.
3. MECHANICAL RESTRAINTS REQUIRED ON ALL PVC HYDRANT LATERALS INCLUDING VALVES AND FITTINGS AS PER REGION STANDARD DRAWING 1-5-9.
4. ALL PIPING, FITTINGS, VALVES, APPURTENANCES AND MECHANICAL RESTRAINTS TO BE c/w DENSO PASTE, DENSO MASTIC AND DENSO TAPE OR APPROVED EQUAL, APPLIED TO MANUFACTURER'S RECOMMENDATIONS.

Region of Peel — Working for you	PUBLIC WORKS STANDARD DRAWING	REV. DATE: APRIL 2014
		APPROVED BY: A.P. / DRAWN BY: AINLEY GROUP
HYDRANT SETTING FOR D.I. OR P.V.C. PIPE		STD. DWG. NUMBER: 1-6-1 / SCALE: N.T.S.

	'A'	'B'
ON UNIMPROVED ROADS	1950	2100
ON CURB AND GUTTER ROADS	1500	1700



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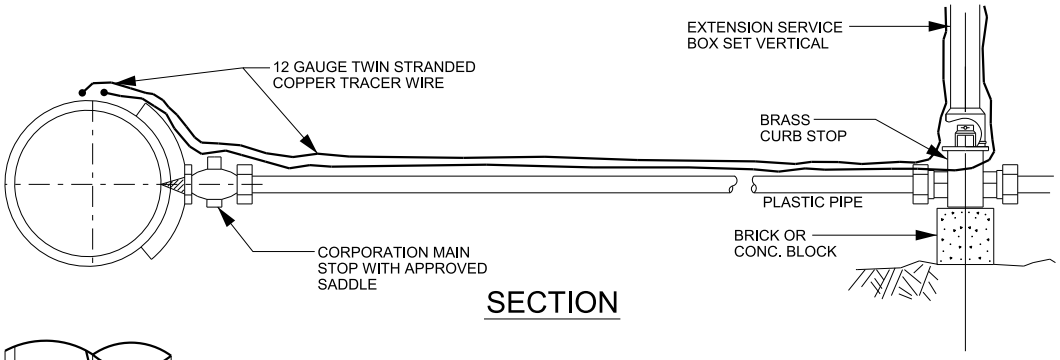

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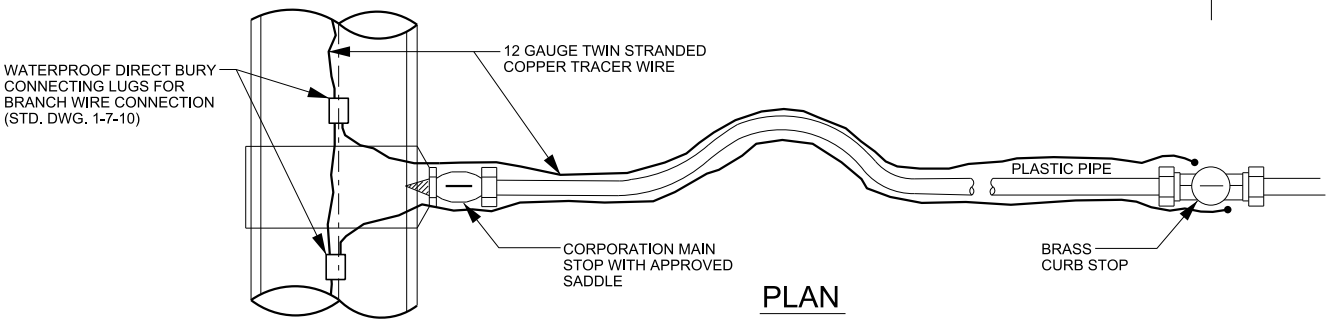

PLAN

NOTE

1. ALL SERVICES TO BE AT 90° TO THE WATERMAIN UNLESS OTHERWISE SPECIFIED.
2. WATER SERVICE TO BE A MIN. OF 1100mm UNDER THE BOTTOM OF DITCH.
3. SERVICE BOX TO BE SET TO FINISH GRADE.
4. NO DIRECT TAPPING OF PVC WATERMAINS. ALL CONNECTIONS TO PVC PIPE TO BE MADE USING AN APPROVED WIDE-BAND SERVICE SADDLE.
5. TRACER WIRE TO BE INSTALLED AS PER REGION STANDARDS.
6. ALL WATER SERVICE MATERIALS SHALL BE STORED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS.

Region of Peel
Working for you

PUBLIC WORKS
STANDARD DRAWING

REV. DATE: APRIL 2014

APPROVED BY	DRAWN BY
A.P.	**AINLEY GROUP**
STD. DWG. NUMBER	SCALE
1-7-1	**N.T.S.**

WATER SERVICE CONNECTION

APPENDIX "C"

Wastewater Servicing Details & Calculations

VALDOR ENGINEERING INC.

File: 17121
April 2022

TABLE: C1

PROJECT: **Amelia Street Subdivision**
Community of Alton
Town of Caledon

SEPTIC SYSTEM SIZING

House Data	
House Model	
Number of bedrooms	**4**
Floor Area (ft^2)	**3,000**
Floor Area (m^2)	278.7
Fixture Count (see attached calcs)	**24.5**

Soils Data	
t-time for native soil (min/cm)	**28**
t-time for fill (min/cm)	**10**

Daily Design Flow*		
Dwelling Flow (L/day) up to 5 bedrooms		2,000
Additional Flow:		800
i) for over 5 bedrooms	0	
ii) area - A	800	
area - B	0	
area - C	0	
iii) fixtures over 20 units	225	
Peak Daily Design Flow (L/day)		2,800
Total Peak Daily Design Flow (L/day)		**2,800**
Septic Tank Capacity (L)	Minimum:	5,600
(Available Sizes 4500L, 6800L, 9000L, 13500L)	Use:	**6,800**

* based on Ontario Building Code Table 8.2.1.3.A

Dwelling design flows (L/day)*		
a) 1 Bedroom	1	750
b) 2 Bedroom	2	1100
c) 3 Bedroom	3	1600
d) 4 Bedroom	4	2000
e) 5 Bedroom	5	2500
f) Additional for:		
i) each bedroom over 5		500
ii) A) each $10m^2$ (or part thereof) over $200m^2$ up to $400m^2$		100
B) each $10m^2$ (or part thereof) over $400m^2$ up to $600m^2$		75
C) each $10m^2$ (or part thereof) over $600m^2$, or		50
iii) each fixture unit over 20 fixture units		50

* part of Ontario Building Code Table 8.2.1.3.A

Loading Rates*	
Percolation Time of Soil	Loading Rates
(min/cm)	(L/m^2/day)
1 < t <= 20	10
20 < t <= 35	8
35 < t <= 50	6
t > 50	4

* part of Ontario Building Code Table 8.7.4.1.A

if t <= 20min - Trench In-Ground Bed		
		Peak Flow
Length of pipe (m)	L = [Q*t/200]	392.0
Approx Bed Area (m^2)	A = [L*1.6]	627.2
if t >20min - Raised Conventional Bed		
		Peak Flow
Length of pipe (m)	L = [Q*t/200]	140.0
Minimum Loading Area (m^2)		350.0
if t >20min - Raised Filter Bed		
		Peak Flow
Bed Area Required (m^2)		350.0
Minimum Contact Area (m^2)		92.2
Effective Filter Area Required (m^2)		37.3
BMEC Area Bed		
Minimum Stone Layer (m^2)		37.3
Minimum Sand Layer (m^2)		196.0

VALDOR ENGINEERING INC.

File: 17121
April 2022

TABLE: C2

PROJECT: **Amelia Street Subdivision**
Community of Alton
Town of Caledon

FIXTURE COUNTS			
Fixture	**Load* (units)**	**Number of Fixtures**	**Calculated Units**
BASEMENT			
Kitchen sink	1.5		0.0
Dishwasher	1.0		0.0
Toilet	4.0		0.0
Bidet	1.0		0.0
Tub/shower	1.5		0.0
Shower	1.5		0.0
Lavatory	1.0		0.0
Whirlpool	1.5		0.0
Other sink	1.5		0.0
Clothes washer	1.5	1	1.5
Laundry tub	1.5	1	1.5
Bathroom group	6.0		0.0
2" Floor Drain	2.0	1	2.0
3" Floor Drain	3.0		0.0
Other			0.0
Main Floor			
Kitchen sink	1.5	1	1.5
Dishwasher	1.0	1	1.0
Toilet	4.0	1	4.0
Bidet	1.0		0.0
Tub/shower	1.5		0.0
Shower	1.5		0.0
Lavatory	1.0	1	1.0
Whirlpool	1.5		0.0
Other sink	1.5		0.0
Clothes washer	1.5		0.0
Laundry tub	1.5		0.0
Bathroom group	6.0		0.0
Other			0.0
Second Floor			
Kitchen sink	1.5		0.0
Dishwasher	1.0		0.0
Toilet	4.0		0.0
Bidet	1.0		0.0
Tub/shower	1.5		0.0
Shower	1.5		0.0
Lavatory	1.0		0.0
Whirlpool	1.5		0.0
Other sink	1.5		0.0
Clothes washer	1.5		0.0
Laundry tub	1.5		0.0
Bathroom group	6.0	2	12.0
Other			0.0
TOTAL			**24.5**

* part of Ontario Building Code Table 7.4.9.3

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Tel: 647-795-8153 | www.pecg.ca

Hydrogeological Investigation

0 Nicholas Street, Alton Village, Town of Caledon, ON

Palmer Project #

1904308

Prepared For

Goodman, Solomon & Gold

January 24, 2022

5. Septic System Servicing Assessment

5.1 Suitability of Site Soils for Leaching Beds

Based on the SWRTs completed as part of this investigation and grain size distribution data obtained during the geotechnical investigation, the shallow soils appear to be suitable to allow for the construction of in-ground leaching beds. The average percolation rate of the subsurface soils is expected to be approximately 20.7 mm/hr when adjusted for a default safety factor of 2.5.

In order to use in-ground systems, a water table separation of approximately 1.2 to 1.5 m is required to meet the requirements under the Ontario Building Code (OBC, Ontario Regulation 332/12). Based on the depths to water table measured across the site on May 18, 2021, in the range of 1.75 to 3.19 mbgs, the use of partially raised beds in the extreme southeastern portion of the site near BH7 (i.e., vicinity of Lots 14 and 15 on the Site Plan, (**Appendix A**) should be considered due to the possibility of a higher water table under seasonal conditions. Additional soil and groundwater testing at later design stages is recommended.

5.2 Site Capacity Effects Assessment

In its comments on an earlier submission regarding the site, the CVC stated that it required (i) a septic impact assessment undertaken in accordance with the MOE's Design Guidelines for Sewage Works 2008 - Chapter 22. Large Subsurface Sewage Disposal Systems and (ii) that the concentration of nitrate in shallow groundwater entering nearby surface water/environmentally sensitive areas meet the CCME, 2012 standard of 3 mg/L nitrate-as-N for the protection of aquatic life. The CVC's first requirement leads to a need to develop a site-specific criterion at the site boundary to a value derived under the MECP's Guideline B7 ("Reasonable Use").

The background nitrate-as-N concentration was found to range from <0.020 to 3.10 mg/L with an average of 1.56 mg/L. The ODWQS for nitrate, applicable to daily flows less than 10,000 L/day, is 10 mg/L. Guideline B7 permits the concentration of a health-related parameter to increase by one-quarter of the difference between background and its ODWQS. This equates to 1.56 + 0.25 * (10 – 1.56) = 3.67 mg/L. It should be noted that only two of the four wells on-site were sampled and that the well with 3.10 mg/L of nitrate is BH6, located in the central portion of the site. The detected nitrate may originate at the septic tile field serving a large commercial development located west of the site (the septic tile field in question is located northwest of the site).

The specific plans for the development in terms of house sizes are unknown at present. It is assumed that each of the 15 lots will be occupied by a 3 bedroom house less than 200 m^2 in area. Under Table 8.2.1.3.a of the Ontario Building Code, the daily flow from each such home is 1,600 L/day or 24,000 L/day for the proposed development in its entirety. Given the uncertainty regarding specifics of the development and that the site-specific criterion of 3.67 mg/L nitrate-as-N at the site boundary is close to the CCME, 2012 standard of 3 mg/L nitrate-as-N, only an on-site assessment has been carried out. Off-site conditions will be assessed when more specific information regarding the proposed development is available.

The post-development infiltration is expected to be 14,181.4 m^3/year (38,853 L/day) once LID measures are put in place to maintain the pre-to-post development water balance as outlined in **Section 4.2.2,** above.

A MECP Procedure D-5-4 discharge assessment and Guideline B7 assessment have been completed to determine the loading that will result from the proposed system. Based on the septic systems outlined previously, the following nitrate loading calculation is provided:

Calculation of Nitrate (NO_3) Concentration in Groundwater:

$$[NO_3] = 3.67\,\frac{mg}{L} = \frac{Septic\ System\ Input + Supply\ Aquifer\ Input}{Available\ Dilution + Volume\ of\ Septic\ Effluent}$$

Where:

- *Septic System Input NO_3 [g/day] = Effluent [NO_3] x [Effluent Volume]*
  - *Allowable Effluent [NO_3] = the unknown being solved for*
  - *Effluent Volume per Day = 24,000 L/day*
- *Supply Aquifer Input NO_3 [g/day] = Background [NO_3] x Effluent Volume*
  - *Background [NO_3] = 1.56 mg/L*
  - *Effluent Volume per Day = 24,000 L/day*
- *Available Dilution [m^3/day] = 38,853 L/day*
  - *Daily infiltration over the site determined using a Thornthwaite & Mather Water Balance (1957) (**Table 8**)*

The resulting allowable concentration of nitrate-as-N in the septic effluent is 8.05 mg/L, which will require the use of tertiary treatment in the septic systems.

Conversion table from mm/hr to min/cm

Borehole	Test ID	Infiltration Rate mm/hr	mm/min	cm/min	min/cm	Desgin infiltration rate (safety factor 2.5) mm/hr	cm/min	min/cm
BH1		30.1	0.50	0.05	**19.93**	**12.0**	0.02	**49.8**
BH5		76	1.27	0.13	**7.89**	**30.4**	0.05	**19.7**
BH6		63.2	1.05	0.11	**9.49**	**25.3**	0.04	**23.7**
BH7		65.7	1.10	0.11	**9.13**	**26.3**	0.04	**22.8**
BH1	SS4	35.7	0.60	0.06	**16.81**	**14.3**	0.02	**42.0**
BH2	SS3	56.4	0.94	0.09	**10.64**	**22.6**	0.04	**26.6**
BH3	SS6	47.5	0.79	0.08	**12.63**	**19.0**	0.03	**31.6**
BH7	SS3	57.8	0.96	0.10	**10.38**	**23.1**	0.04	**26.0**
	Average	54.1	0.90	0.09	**11.10**	**21.6**	0.04	**27.8**

DRAFT REPORT ON
PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED NEW ALTON SUBDIVISION
QUEEN STREET
CALEDON, ONTARIO

Prepared for:

1029629 ONTARIO INC.

Prepared By:

SIRATI & PARTNERS CONSULTANTS LIMITED

Sirati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

750 Millway Avenue, Unit 8
Vaughan, Ontario L4K 3T7
Tel: 905.669.4477
Fax: 905.669.4488

Project: SP17-213-10
September 6, 2017

Sirati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH1

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/06/2017

REF. NO.: SP17-213-10
ENCL NO.: 2

SOIL PROFILE (m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) O UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** 220mm											
0.2	**FILL:** silty sand, trace clay, trace gravel, dark brown, very moist, very loose		1	SS	3							
0.8	**SANDY SILT TO SILY SAND:** trace clay, trace gravel, brown, moist, very loose		2	SS	3							
1.5	**SILT:** trace to some sand, trace clay, brown, wet, very loose		3	SS	2							
2.3	**SANDY SILT TO SILY SAND:** trace clay, trace gravel, brown, wet, very loose to loose		4	SS	4							0 57 30 13
			5	SS	2		W. L. 2.9 mBGL Jul 13, 2017					
4.6	**SAND:** trace silt, grey, wet, loose		6	SS	9							
5.1	**SANDY SILT TO SILY SAND:** trace clay, trace gravel, brown, wet, loose											
6.1	**SAND AND GRAVEL:** trace silt, some cobbles, brown, wet, very dense		7	SS	50/ 20mm							
6.4	**END OF BOREHOLE**											

Notes:
1) Monitoring well installed in the borehole upon completion.
2) Water level in the well was measured to be at 2.9 m depth on July 13, 2017.

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^3$ $\times^3$ Numbers refer to Sensitivity O ε=3% Strain at Failure

Sirati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH2

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/07/2017

REF. NO.: SP17-213-10
ENCL NO.: 3

(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
0.0	**TOPSOIL:** 250mm														
0.3	**FILL:** silty sand, trace topsoil, trace gravel, brown, moist, very loose		1	SS	3										
			2	SS	4										
1.5	**SAND AND GRAVEL:** trace silt, brown to dark brown, wet, very loose		3	SS	4							9	72	7	12
	dense at 2.3m		4	SS	34										
3.0	**FINE SAND:** trace to some silt, some gravel, occasional cobble/boulder, brown, wet, compact to very dense		5	SS	11										
			6	SS	45										
			7	SS	51										
7.6	**END OF BOREHOLE** Notes: 1) Auger refusal and terminated. 2) Water level at 1.8 m during drilling.		8	SS	50/ 25mm										

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^{3}$ $\times^{3}$ Numbers refer to Sensitivity ○ ε=3% Strain at Failure

SPCL SOIL LOG SP17-213-10 - ALTON SUBDIVISION.GPJ SPCL.GDT 8/29/17

Strati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH3

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/05/2017

REF. NO.: SP17-213-10
ENCL NO.: 4

(m) ELEV / DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** 250mm											
0.3	**FILL:** sandy silt, trace clay, brown, moist, very loose to loose		1	SS	2							
			2	SS	5							
1.5	**SANDY SILT:** trace clay, greyish brown, wet, very loose to loose		3	SS	4							
			4	SS	2							
			5	SS	9							
3.7	**SAND:** trace silt, brown, wet, loose to compact		6	SS	24							0 81 7 12
6.1	**SANDY SILT TO SILTY SAND:** trace clay, trace gravel,brown, wet, compact		7	SS	12							
	very dense below 7.6m		8	SS	23							
8.2	**END OF BOREHOLE** Notes: 1) Auger refusal and terminated. 2) Water level at 2.4 m during drilling.											

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^{3}$, $\times^{3}$ Numbers refer to Sensitivity ○ ε=3% Strain at Failure

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Sirati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH4

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/06/2017

REF. NO.: SP17-213-10
ENCL NO.: 5

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** 330mm						
0.3	**FILL:** silty sand, trace clay, brown, moist, very loose	1	SS	2			
		2	SS	2			
1.5	**FILL:** sand, some silt, trace topsoil, trace gravel, brown, moist, very loose	3	SS	1			
2.3	**SANDY SILT:** trace clay, trace gravel, greyish brown, very moist, very loose	4	SS	4			
3.0	**SILT:** trace to some clay, trace sand, greyish brown, wet, compact	5	SS	14			
4.6	**SAND:** trace to some silt, trace clay, brown, wet, compact	6	SS	16			
		7	SS	14			
7.6	**SILT:** trace to some clay, trace sand, grey, wet, compact	8	SS	23			
	very dense below 9.1m	9	SS	53			
9.7	**END OF BOREHOLE** Notes: 1) Water level at 2.4 m during drilling.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE & Sensitivity, × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: +³ ×³ Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

SPCL SOIL LOG SP17-213-10 - ALTON SUBDIVISION.GPJ SPCL.GDT 8/29/17

Strata & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH5

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/07/2017

REF. NO.: SP17-213-10
ENCL NO.: 6

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS / 0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:**250mm						
0.3	**FILL:** sand, some crusher limestone, brown to grey, moist, very loose	1	SS	2			
		2	SS	4			
1.6	**SANDY SILT TO SILTY SAND:** trace clay, brown, moist, compact	3	SS	10			
2.3	**SILTY SAND TO SAND:** trace gravel, brown, wet, compact	4	SS	14		W. L. 2.7 mBGL Jul 13, 2017	
3.0	**SANDY SILT TO SILTY SAND:** trace clay, brown, wet, compact to dense	5	SS	13			
4.6	**SANDY SILT TILL:** trace to some clay, occasional cobble/boulder, brown, wet, dense	6	SS	40			
6.1	**SAND AND GRAVEL:** some cobble/boulder, greyish brown, wet, dense to very dense	7	SS	30			
7.6	**END OF BOREHOLE** Notes: 1) Auger refusal and terminated. 2) Water level in the well was measured to be at 2.7 m depth on July 13, 2017.	8	SS	50/ 25mm			

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa) — NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3$ $\times^3$ Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

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Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH6

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/06/2017

REF. NO.: SP17-213-10
ENCL NO.: 7

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	ELEVATION
0.0	**TOPSOIL:**280mm				
0.3	**SAND:** trace to some gravel, trace silt, greyish brown, moist, very loose to loose	1	SS	2	
		2	SS	9	
1.5	**GRAVELLY SAND:** trace silt, greyish brown, moist, dense	3	SS	31	
2.3	**SILTY SAND:** trace clay, brown, wet, compact	4	SS	13	
3.0	**SILT TO SANDY SILT:** trace clay, trace gravel, brown, wet, compact	5	SS	26	W. L. 2.9 mBGL Jul 13, 2017
4.6	**SAND:** trace silt, grey, wet, compact	6	SS	16	
6.1	**SANDY SILT TILL:** trace to some clay, trace gravel, layer of crusher limestone, brown, wet, very dense	7	SS	55	
7.6	**SILTY SAND TILL:** trace clay, trace gravel, brown, wet, loose	8	SS	8	
	dense to very dense below 9.1m	9	SS	40	
10.4	**END OF BOREHOLE**	10	SS	50/ 25mm	

Notes:
1) Monitoring well installed in the borehole upon completion.
2) Water level in the well was measured to be at 2.9 m depth on July 13, 2017.

STRATA PLOT / GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa) — NATURAL UNIT WT (kN/m³) — REMARKS AND GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

SPCL SOIL LOG SP17-213-10 - ALTON SUBDIVISION.GPJ SPCL.GDT 8/29/17

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^3$, $\times^3$ Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Strati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH7

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/07/2017

REF. NO.: SP17-213-10
ENCL NO.: 8

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
0.0	**TOPSOIL:**280mm									
0.3	**FILL:** silty sand, trace topsoil, trace rootlets, brown, moist, loose	1	SS	6						
0.8	**FILL:** sand mixed with limestone, trace cobble, brown, moist, compact	2	SS	22						
1.5	**SILTY SAND:** trace sand, trace clay, brown, wet, loose to compact	3	SS	8	W. L. 1.3 mBGL Jul 13, 2017		0	79	13	8
		4	SS	13						
		5	SS	14						
		6	SS	19						
		7	SS	22						
		8	SS	8						
9.1	**SILTY CLAY:** trace sand, trace gravel, grey, wet, stiff	9	SS	8						
10.7	**CLAYEY SILT TILL:** some sand, trace gravel, grey, moist, hard	10	SS	36						
12.2	**INTERBEDDED SILT AND CLAYEY SILT:** trace sand, trace gravel, brown, moist, stiff	11	SS	13						
12.7	**END OF BOREHOLE**									

Notes:
1) Monitoring well installed in the borehole upon completion.
2) Water level in the well was measured to be at 1.3 m depth on July 13, 2017.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^3$ $\times^3$ Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Strati & Partners Consultants Ltd.
Geotechnical & Environmental Services
Engineering Solutions

LOG OF BOREHOLE BH8

PROJECT: Geotechnical and Hydrogeological Services
CLIENT: Derrick Libawski
PROJECT LOCATION: New Alton Subdivision Queen Street, Caledon, ON
DATUM: Geodetic
BH LOCATION: See Drawing 1

DRILLING DATA
Method: Hollow Stem Augers
Diameter: 200mm
Date: Jun/07/2017

REF. NO.: SP17-213-10
ENCL NO.: 9

(m) ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**TOPSOIL:** 280mm						
0.3	**FILL:** silty sand, trace topsoil, brown, moist, very loose	1	SS	3			
0.8	**SILTY SAND:** trace clay, brown, moist, very loose	2	SS	2			
1.5	**SAND:** trace to some silt, brown, moist, very loose	3	SS	4			
2.3	**SILTY SAND:** trace clay, trace gravel, brown, wet, compact	4	SS	10			
		5	SS	11			
4.6	**SANDY SILT:** trace clay, grey, wet, compact	6	SS	10			
		7	SS	12			
6.7	**SAND:** trace silt, grey, wet, compact						
7.6	**SAND AND GRAVEL WITH COBBLES:** some silt, brown, wet, compact to dense	8	SS	18			
		9	SS	38			
9.7	**END OF BOREHOLE**						

Notes:
1) Water level at 2.6 m during drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa) — NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES: +³ ×³ Numbers refer to Sensitivity; ○ ■=3% Strain at Failure

SPCL SOIL LOG SP17-213-10 - ALTON SUBDIVISION.GPJ SPCL.GDT 8/29/17

Design & Installation Guide

CAN/BNQ 3680-600

Class B-IV

Version 4.0
January 2020

Waterloo Biofilter Systems Inc.
143 Dennis Street, PO Box 400
Rockwood ON N0B 2K0
Tel: 519-856-0757 Fax: 519-856-0759
1-866-366-4329
info@waterloo-biofilter.com
www.waterloo-biofilter.com

CAN/BNQ 3680-600

2312

Certification

The Waterloo Biofilter is an onsite residential wastewater treatment system complying with the CAN/BNQ 3680-600 Standard, Certification No. 2312.

CAN/BNQ 3680-600

2312

Class of Treatment: B-IV (10 mg/L TSS, 10 mg/L $cBOD_5$).

Influent Temperature: Temperature condition *b)* – controlled to assure a minimum influent temperature of 10°C.

Scope: The CAN/BNQ 3680-600 standard applies only to onsite residential wastewater treatment technologies used to treat wastewater from isolated dwellings.

It is not the intention of the standard to apply to water reuse technologies, or technologies for other buildings similar to isolated dwellings, such as service stations, restaurants, etc.

The standard does not cover the part of a treatment system used for distribution and disposal of treated effluent.

Capacity: 1,100 L/day – 10,000 L/day.

For information on systems outside the scope of this document, such as commercial, light industrial, communal or clustered applications, please contact us at 1-866-366-4329.

Introduction

The Waterloo Biofilter system uses a patented, highly engineered foam filtration medium to passively treat wastewater. The absorbent foam filter medium creates an ideal environment for microbial attachment. Beneficial bacteria colonize the interior surfaces of the filter medium where they are protected from predators, desiccation, and freezing. These microbes degrade and oxidize organic pollutants, coliform bacteria, ammonium, and other contaminants as the wastewater is retained in the filter medium by capillarity. Air passively circulates throughout the filter media providing an aerobic treatment environment without the need for forced aeration. The Waterloo Biofilter provides both physical filtration and biological treatment of wastewater in one step – ensuring high levels of pollutants are removed before water is returned to the natural environment.

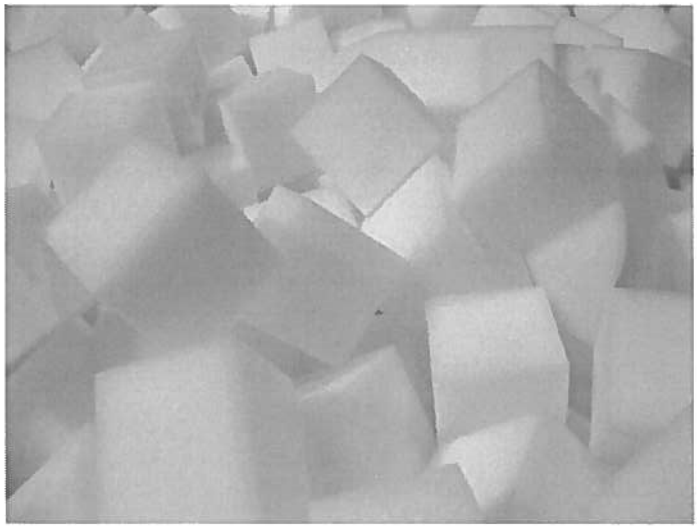


Figure 1. Waterloo Biofilter foam filter media

This attached growth process outperforms activated sludge or suspended growth (suspended sludge) processes with lower energy requirements, fewer moving parts, simpler operation, less maintenance, and a better ability to handle shock loads of chemical addition or hydraulic overloads. Of high importance in Canada, the Waterloo Biofilter is proven in frigid -50°C temperatures, treating cold sewage with influent temperatures as low as 3°C. Compared to other media-based trickling filters, the Waterloo Biofilter does not slough off microbes in the form of aerobic sludge, does not need media replacement, has longer retention times, and can accept much higher organic and hydraulic loads without plugging.

Advantages

Waterloo Biofilters have been treating residential wastewaters onsite since 1993. Besides an excellent and thoroughly tested product, when working with Waterloo Biofilter Systems Inc. you also receive system design assistance, troubleshooting, and technical support from our knowledgeable staff. We pride ourselves on the high level of customer support we offer to our designers, installers, and end users.

A few of the distinct advantages of the Waterloo Biofilter technology include:

- Permanent filter media warranted for 20 years
- Very low energy consumption, up to 85% less than aeration technology
- Simple maintenance with few moving parts
- Compact footprint
- No noisy, high maintenance air compressors
- Variety of system configurations designed for different site constraints
- Simple system components that are lightweight and easy to install
- Accepts high hydraulic loading rates without plugging
- Easily handles seasonal use and surge flows
- Robust technology easily recovers from misuse such as chemical use

Figure 2. Waterloo Biofilter foam media warranty

The Waterloo Biofilter foam filter medium retains moisture even during prolonged periods of no use. Microbial colonies do not desiccate (dry out) and die, instead they go dormant until the system starts up again resulting in a short start-up period. This makes the Waterloo Biofilter system ideal for seasonal use applications such as cottages and cabins.

All Waterloo Biofilter treatment units are designed with installation and maintenance in mind. Units are lightweight and can be easily manoeuvred onsite or even barged to islands. All system components come pre-assembled wherever possible to speed up the installation process and minimize potential errors. Float switch controls come colour coordinated at both ends simplifying installation and reducing errors. Care is taken in the design of our systems to ensure that components are easily accessible and replaceable for maintenance – from providing lifting ropes on all pumps to external electrical splice boxes to prevent pulling wires, we strive to ensure simple and easy installation and ongoing maintenance for high quality septic infrastructure.

The Waterloo Biofilter treatment process itself requires no energy; pumps are used simply to move the water around within the system. If the homeowners are not using water at the residence, either for short or extended periods of time, the system will not use any energy beyond powering the control panel. This results in very low electricity usage – even less than recirculating sand filters.

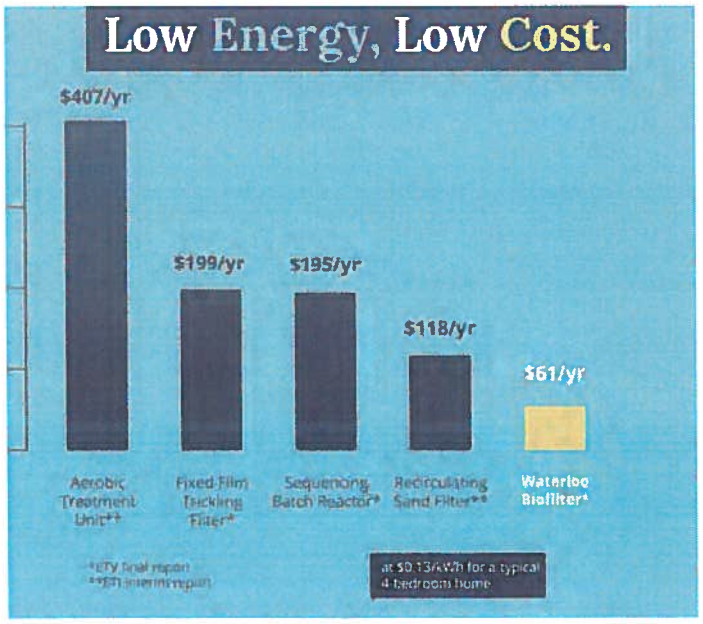


Figure 3. Energy use comparison

Treatment Train

The Waterloo Biofilter process consists of three elements:

1. Anaerobic digester
2. Biofilter dosing pump chamber (internal to anaerobic digester) OR pump tank (external to anaerobic digester)
3. Waterloo Biofilter treatment unit

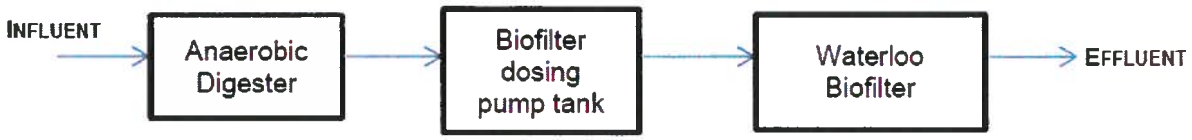


Figure 4. Waterloo Biofilter flow diagram

1. Wastewater from the dwelling first enters the anaerobic digester for pre-treatment. The anaerobic digester function is similar to that of a standard septic tank, however the anaerobic digester is designed to improve treatment by minimizing short circuiting, reducing scum formation, and improving solids digestion by the addition of an InnerTube™ pipe.

2. Effluent from the anaerobic digester is then dosed to the Waterloo Biofilter treatment unit in short frequent doses. This can be accomplished using either an internal pump chamber housed inside the anaerobic digester, or an external pump tank following the anaerobic digester. Dosing to the treatment unit is always controlled on a timed frequency-duration basis.

3. The Waterloo Biofilter treatment unit houses the foam filter media in either an above-ground, at-grade, or below-ground container. Anaerobic digester effluent is evenly distributed over the foam media using pressurized spray nozzles or perforated pipes, where it then trickles down through and is absorbed by the foam media. Passive air vents allow atmospheric oxygen to enter the container providing an aerobic environment without energy use. The foam filter media is free-draining and treated effluent exits the bottom of the foam where it is directed to final dispersal via either gravity or a second pump. Recirculation of treated effluent back to the anaerobic digester is often used to increase nitrogen removal, moderate flow rates, and generally improve treatment performance.

Sizing Chart

Anaerobic digesters are sized using the design flow of the dwelling based on a minimum retention time of 1.89 days (**1.89 x Q**), of which a minimum 0.2 days (**0.2 x Q**) retention is inside the InnerTube pipe, or the minimum listed in the chart below. The volume of the InnerTube pipe is included in the minimum capacity of the anaerobic digester.

The following Table outlines the minimum sizing requirements for the anaerobic digester and the InnerTube pipe:

Table 3. Anaerobic Digester Sizing

Flow Rate (L/day)	Minimum Anaerobic Digester Volume (L)	Minimum InnerTube Volume (L)
1,100	3,021	320
1,600	3,021	320
2,000	3,776	400
2,500	4,720	500
3,000	5,664	600
3,500	6,608	700
4,000	7,552	800
4,500	8,496	900
5,000	9,440	1,000
5,500	10,384	1,100
6,000	11,328	1,200
6,500	12,272	1,300
7,000	13,216	1,400
7,500	14,160	1,500
8,000	15,104	1,600
8,500	16,048	1,700
9,000	16,992	1,800
9,500	17,936	1,900
10,000	18,880	2,000

2020 Waterloo Biofilter Tank Sizing

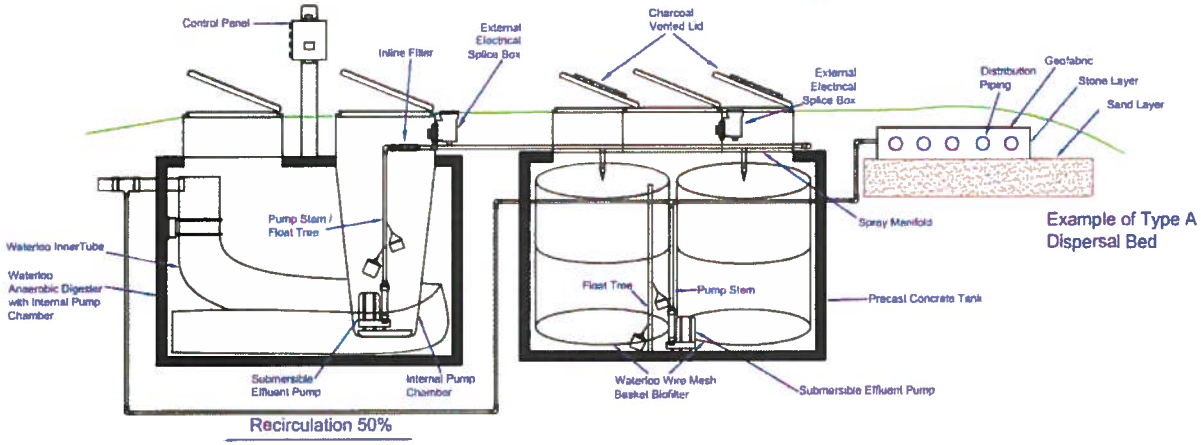
Orangeville Precast Concrete Ltd.

Phone: 519-941-4050 | Email: info@orangevilleprecast.com | Orangeville ON

Select Option 1 to dose Biofilter from the internal pump chamber
Select Option 2 to dose Biofilter from the external pump tank
Please contact concrete provider to order the necessary tankage

Design Capacity (L/day)	Option 1 Anaerobic Digester with Internal Pump Chamber (ADIPC)	Option 2 Anaerobic Digester (AD) with External Pump Tank (PT)		Basket Biofilter Tank (BT)
1,600	ADIPC-4750	AD-3600	PT-1800	BT-5400
2,000	ADIPC-5400	AD-4750	PT-1800	BT-5400
2,500	ADIPC-7480	AD-5400	PT-1800	BT-7480
3,000	ADIPC-9000	AD-7480	PT-3000	BT-7480
3,500	ADIPC-9000	AD-9000	PT-3600	BT-9000
4,000	ADIPC-11250	AD-9000	PT-3600	BT-9000
4,500	ADIPC-11250	AD-9000	PT-4750	BT-11250
5,000	ADIPC-13500	AD-11250	PT-4750	BT-11250
5,500	N/A	AD-11250	PT-5400	BT-11250
6,000		AD-11250	PT-5400	BT-13500
6,500		AD-13500	PT-5400	BT-13500
7,000		AD-13500	PT-7480	BT-18000
7,500		AD-18000	PT-7480	BT-18000
8,000		AD-18000	PT-7480	BT-18000
8,500		AD-18000	PT-9000	BT-18000
9,000		AD-18000	PT-9000	BT-18000
9,500		AD-22980	PT-9000	BT-22980
10,000		AD-22980	PT-9000	BT-22980

System Diagram – Baskets in Concrete Tank



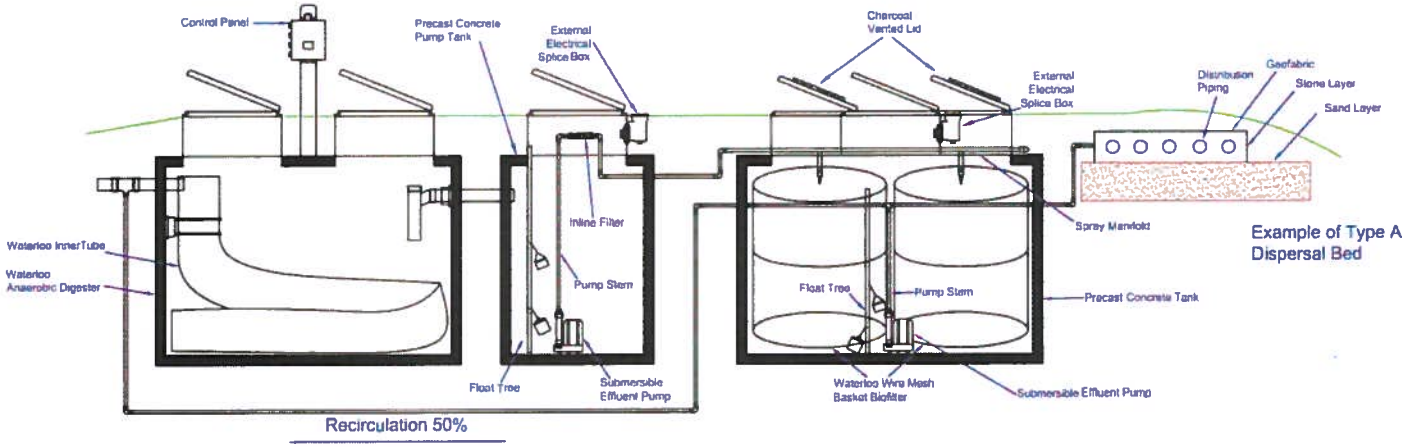
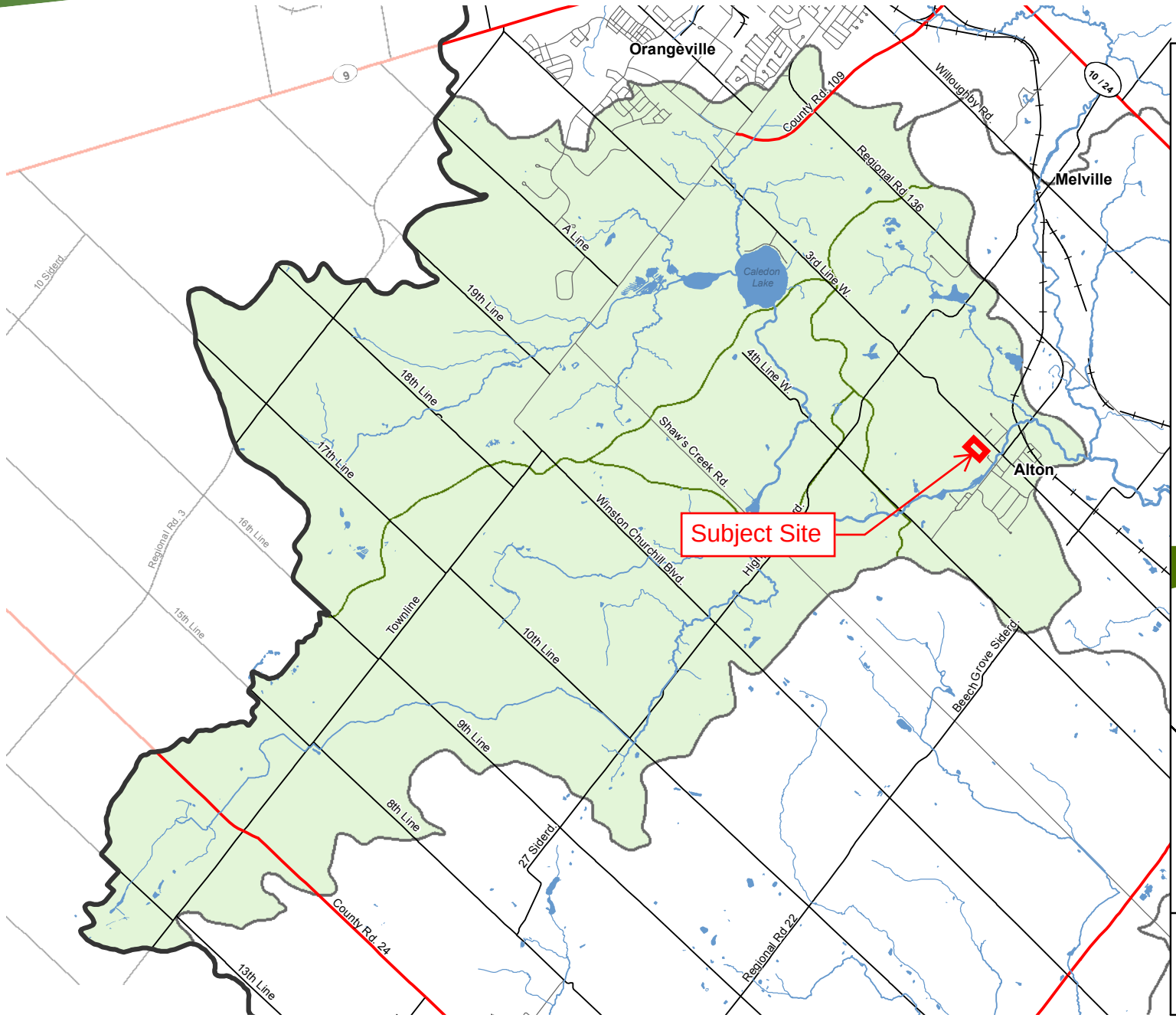


Figure 42. Baskets in concrete tank system diagram

APPENDIX "D"

Watershed Map, Regulation Mapping & IDF Data

17-Shaw's Creek Subwatershed



CVC Conservation Areas

Trail Network

Rivers and Streams

Lakes and Ponds

CVC Watershed Boundary

Subwatershed Boundary

Subcatchment Boundary

Transportation Network

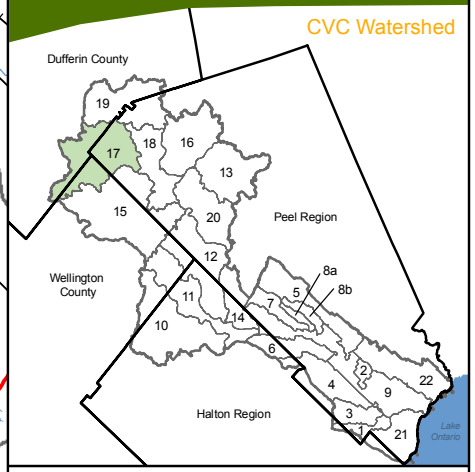
Highway

Railroad

Major Road

Street

0 0.5 1 2 3 km



Credit Valley Conservation assumes no responsibility for any errors and is not liable for any damages of any kind resulting from the use of, or reliance on the information contained herein.

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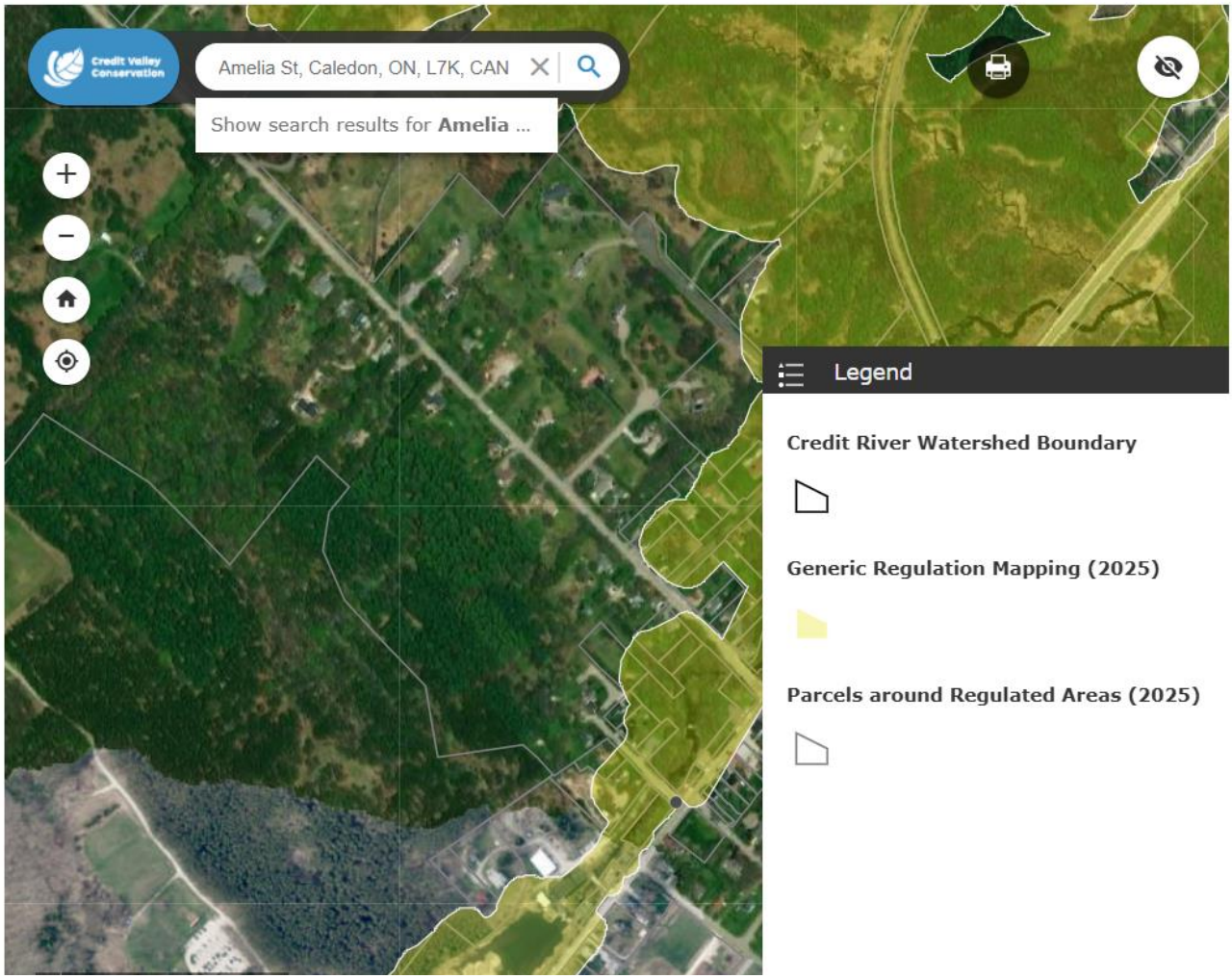
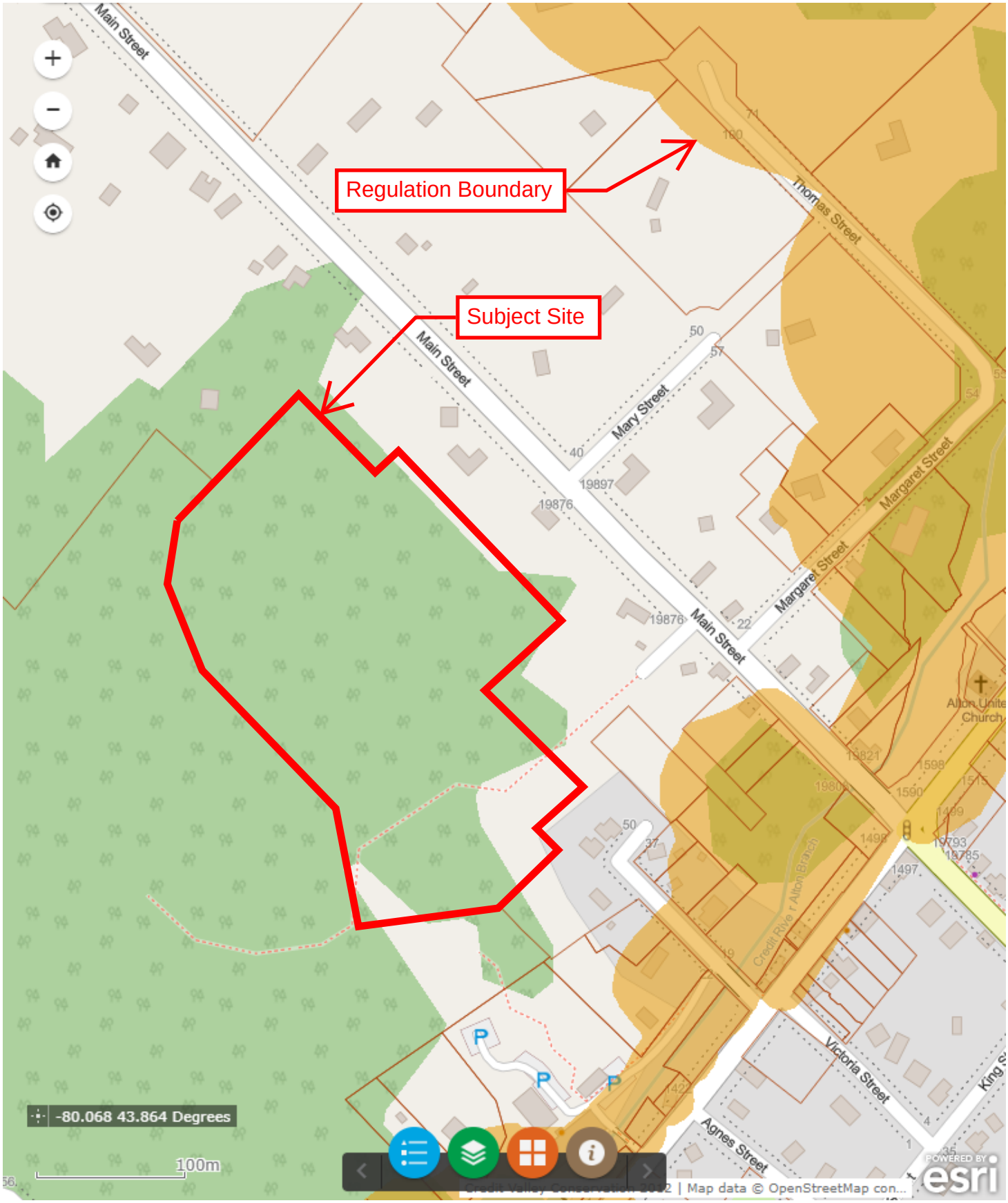
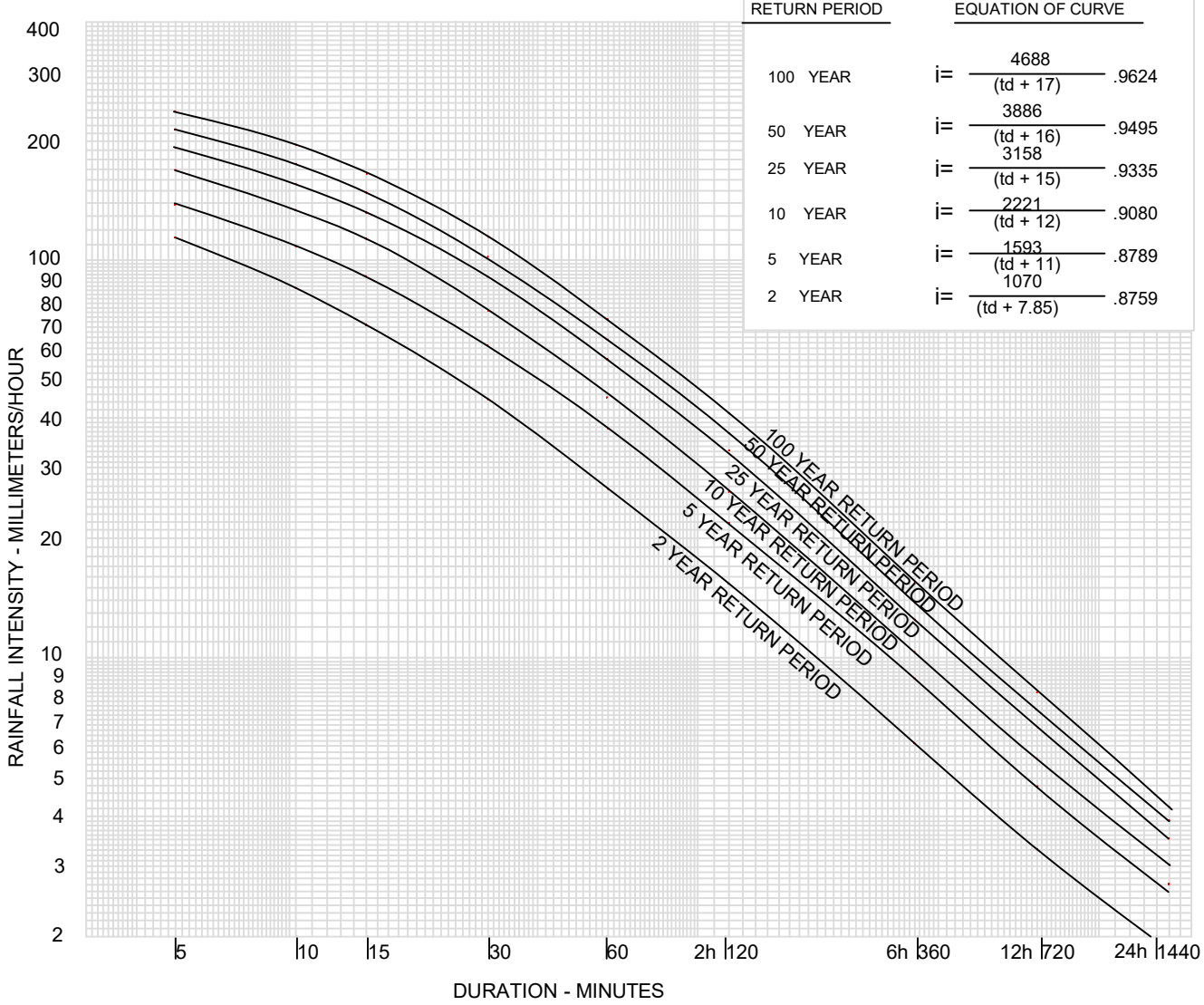
THIS IS NOT A PLAN OF SURVEY

CVC REGULATION MAPPING





CVC Regulation Mapping for
Shaw's Creek



RETURN PERIOD	EQUATION OF CURVE	
100 YEAR	$i = \frac{4688}{(td + 17)}^{.9624}$	
50 YEAR	$i = \frac{3886}{(td + 16)}^{.9495}$	
25 YEAR	$i = \frac{3158}{(td + 15)}^{.9335}$	
10 YEAR	$i = \frac{2221}{(td + 12)}^{.9080}$	
5 YEAR	$i = \frac{1593}{(td + 11)}^{.8789}$	
2 YEAR	$i = \frac{1070}{(td + 7.85)}^{.8759}$	

INLET TIMES

SUBURBAN RESIDENTIAL	
(ROOF DRAINS UNCONNECTED)	15 min
(ROOF DRAINS CONNECTED)	10 min
SUBURBAN, COMMERCIAL, INDUSTRIAL MULTIPLE FAMILY	10 min
DOWNTOWN COMMERCIAL, HIGH DENSITY APARTMENTS, EXPRESSWAYS	5 min

RUNOFF COEFFICIENT

COMMERCIAL	
- DOWNTOWN & SUBURBAN SHOPPING	0.90
INDUSTRIAL	
- DOWNTOWN	0.90
- SUBURBAN INDUSTRIAL PARKS	0.75
RESIDENTIAL	
- APARTMENTS	0.75
- ROW DWELLINGS	0.70
- DUPLEX DWELLINGS	0.70
- SEMIDETACHED - DOWNTOWN	0.60
- SINGLE FAMILY - DOWNTOWN	0.60
- SEMIDETACHED - SUBURBAN	0.50
- SINGLE FAMILY - SUBURBAN	0.40
SCHOOLS, CHURCHES, HOSPITALS	0.75
PARKS, CEMETERIES, RAIL YARDS	
(OVER 4 Ha)	0.20
(UNDER 4 Ha)	0.25
PARKING LOTS ASPHALT & GRAVEL	0.90

TOWN OF CALEDON

RAINFALL INTENSITY CURVES

NO.	REVISION	APR'D	DATE
3	ADDITION OF TEXT		APR 19
2	STANDARD 104 NOW 103		JAN 08
1	STANDARD 112.01 NOW 104		JUNE 08

APR'D: C.C. DATE: FEB 2000

DRAWN: BJM SCALE: N.T.S.

STANDARD No. 103

APPENDIX "E"

Stormwater Management Calculations

VALDOR ENGINEERING INC.
File: 17121
April 2026

Project: **Amelia Street Subdivision, Town of Caledon**

TABLE E1: PEAK FLOW SUMMARY

Condition	Catchment	Area (ha)	Runoff Coefficent	Peak Flow (L/s)					
				2 Year	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**100 year**
Pre-Development	101	2.650	0.15	97.6	124.9	152.7	178.1	200.6	223.7
	Total	2.650	0.15	**97.6**	**124.9**	**152.7**	**178.1**	**200.6**	**223.7**
Post-Development (Un-Mitigated)	201	2.020	0.50	240.4	307.6	376.3	438.9	494.2	551.3
	202	0.630	0.43	64.2	82.2	100.5	117.2	132.0	147.2
	Total	2.650	0.48	**304.7**	**389.8**	**476.8**	**556.1**	**626.2**	**698.5**
Post-Development (Mitigated)	201	2.020	0.50	32.7	40.2	46.2	52.0	55.9	59.9
	202	0.630	0.43	64.2	82.2	100.5	117.2	132.0	147.2
	Total	2.650	0.48	**96.9**	**122.3**	**146.7**	**169.2**	**187.9**	**207.2**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E2

PRE-DEVELOPMENT PEAK FLOW CALCULATION
Catchment 101

Surface Type	Area	R
Landscape/Forest	0.12	0.25
Forest	2.53	0.15
Impervious	0.00	0.95
Total	2.65	0.15

2 Year Pre-Development Flow

$I = 1070 / (t_c+7.85)$ ^ 0.8759 where I = Rainfall Rate (mm/hr)

t_c = 10 minutes
I = 85.72 mm/hr
R = 0.15 (composite)
N = 2.78

Q = R x A x I x N 2 year Q = 97.58 L/s

5 Year Pre-Development Flow

$I = 1593 / (t_c+11)$ ^ 0.8789 where I = Rainfall Rate (mm/hr)

t_c = 10 minutes
I = 109.68 mm/hr
R = 0.15 (composite)
N = 2.78

Q = R x A x I x N 5 year Q = 124.86 L/s

10 Year Pre-Development Flow

$I = 2221 / (t_c+12)$ ^ 0.9080 where I = Rainfall Rate (mm/hr)

t_c = 10 minutes
I = 134.16 mm/hr
R = 0.15 (composite)
N = 2.78

Q = R x A x I x N 10 year Q = 152.73 L/s

25 Year Pre-Development Flow

$I = 3158 / (t_c+15)$ ^ 09335 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	156.47	mm/hr
R =	0.15	(composite)
N =	2.78	

Q = R x A x I x N 25 year Q = 178.13 L/s

50 Year Pre-Development Flow

$I = 3886 / (t_c+16)$ ^ 0.9495 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	176.19	mm/hr
R =	0.15	(composite)
N =	2.78	

Q = R x A x I x N 50 year Q = 200.58 L/s

100 Year Pre-Development Flow

$I = 4688 / (t_c+17)$ ^ 0.9624 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	196.54	mm/hr
R =	0.15	(composite)

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E3-A

POST-DEVELOPMENT PEAK FLOW CALCULATION: UNMITIGATED
Catchment 201

Surface Type	Area	R
Landscape	1.30	0.25
Forest	0.00	0.15
Impervious	0.72	0.95
Total	2.02	0.50

2 Year Post-Development Flow

$I = 1070 / (t_c+7.85)$ ^ 0.8759 where I = Rainfall Rate (mm/hr)

t_c = 10 minutes
I = 85.72 mm/hr
R= 0.50 (composite)
N = 2.78

Q = R x A x I x N 2 year Q = 240.44 L/s

5 Year Post-Development Flow

$I = 1593 / (t_c+11)$ ^ 0.8789 where I = Rainfall Rate (mm/hr)

t_c = 10 minutes
I = 109.68 mm/hr
R= 0.50 (composite)
N = 2.78

Q = R x A x I x N 5 year Q = 307.65 L/s

10 Year Post-Development Flow

$I = 2221 / (t_c+12)$ ^ 0.9080 where I = Rainfall Rate (mm/hr)

t_c = 10 minutes
I = 134.16 mm/hr
R= 0.50 (composite)
N = 2.78

Q = R x A x I x N 10 year Q = 376.33 L/s

25 Year Post-Development Flow

$I = 3158 / (t_c+15)$ ^ 09335 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	156.47	mm/hr
R=	0.50	(composite)
N =	2.78	

Q = R x A x I x N 25 year Q = 438.91 L/s

50 Year Post-Development Flow

$I = 3886 / (t_c+16)$ ^ 0.9495 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	176.19	mm/hr
R=	0.50	(composite)
N =	2.78	

Q = R x A x I x N 50 year Q = 494.22 L/s

100 Year Post-Development Flow

$I = 4688 / (t_c+17)$ ^ 0.9624 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	196.5	mm/hr
R=	0.50	(composite)
N =	2.78	

Q = R x A x I x N 100 year Q = 551.29 L/s

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E3-B

POST-DEVELOPMENT PEAK FLOW CALCULATION: UNMITIGATED
Catchment 201

Surface Type	Area	R
Landscape	0.47	0.25
Forest	0.00	0.15
Impervious	0.16	0.95
Total	0.63	0.43

2 Year Post-Development Flow

$I = 1070 / (t_c+7.85)$ ^ 0.8759 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	85.72	mm/hr
R=	0.43	(composite)
N =	2.78	

Q = R x A x I x N 2 year Q = 64.22 L/s

5 Year Post-Development Flow

$I = 1593 / (t_c+11)$ ^ 0.8789 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	109.68	mm/hr
R=	0.43	(composite)
N =	2.78	

Q = R x A x I x N 5 year Q = 82.17 L/s

10 Year Post-Development Flow

$I = 2221 / (t_c+12)$ ^ 0.9080 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	134.16	mm/hr
R=	0.43	(composite)
N =	2.78	

Q = R x A x I x N 10 year Q = 100.52 L/s

25 Year Post-Development Flow

$I = 3158 / (t_c+15)$ ^ 09335 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	156.47	mm/hr
R=	0.43	(composite)
N =	2.78	

Q = R x A x I x N 25 year Q = 117.23 L/s

50 Year Post-Development Flow

$I = 3886 / (t_c+16)$ ^ 0.9495 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	176.19	mm/hr
R=	0.43	(composite)
N =	2.78	

Q = R x A x I x N 50 year Q = 132.00 L/s

100 Year Post-Development Flow

$I = 4688 / (t_c+17)$ ^ 0.9624 where I = Rainfall Rate (mm/hr)

t_c =	10	minutes
I =	196.54	mm/hr
R=	0.43	(composite)
N =	2.78	

Q = R x A x I x N 100 year Q = 147.25 L/s

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E4-A

CONTROL ORIFICE DESIGN
2 Year Storm

Orifice Pipe Location	=	Detention Tank		
Orifice Coefficient (C)	=	0.63	(Plate)	
Acceleration due to gravity (g)	=	9.81	(m/s/s)	
Orifice Invert	=	0.00	(m)	
High Water Level	=	0.78	(m)	2 Year
Orifice diameter	=	133	(mm)	
Cross section area of orifice (A)	=	0.0139	(sq.m.)	
Head (H)	=	0.71	(m)	
Actual Discharge (Q) (C x A x (2 x g x H)^0.5)	=	32.7	(L/s)	

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E4-B

CONTROL ORIFICE DESIGN
5 Year Storm

Orifice Pipe Location	=	Detention Tank		
Orifice Coefficient (C)	=	0.63	(Plate)	
Acceleration due to gravity (g)	=	9.81	(m/s/s)	
Orifice Invert	=	0.00	(m)	
High Water Level	=	1.14	(m)	5 Year
Orifice diameter	=	133	(mm)	
Cross section area of orifice (A)	=	0.0139	(sq.m.)	
Head (H)	=	1.07	(m)	
Actual Discharge (Q) (C x A x (2 x g x H)^0.5)	=	40.2	(L/s)	

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E4-C

CONTROL ORIFICE DESIGN
10 Year Storm

Orifice Pipe Location	=	Detention Tank		
Orifice Coefficient (C)	=	0.63	(Plate)	
Acceleration due to gravity (g)	=	9.81	(m/s/s)	
Orifice Invert	=	0.00	(m)	
High Water Level	=	1.42	(m)	10 Year
Orifice diameter	=	133	(mm)	
Cross section area of orifice (A)	=	0.0139	(sq.m.)	
Head (H)	=	1.42	(m)	
Actual Discharge (Q)	**=**	**46.2**	**(L/s)**	
$(C \times A \times (2 \times g \times H)^{0.5})$				

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E4-D

CONTROL ORIFICE DESIGN
25 Year Storm

Orifice Pipe Location	=	Detention Tank		
Orifice Coefficient (C)	=	0.63	(Plate)	
Acceleration due to gravity (g)	=	9.81	(m/s/s)	
Orifice Invert	=	0.00	(m)	
High Water Level	=	1.80	(m)	10 Year
Orifice diameter	=	133	(mm)	
Cross section area of orifice (A)	=	0.0139	(sq.m.)	
Head (H)	=	1.80	(m)	
Actual Discharge (Q)	**=**	**52.0**	**(L/s)**	
$(C \times A \times (2 \times g \times H)^{0.5})$				

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E4-E

CONTROL ORIFICE DESIGN
50 Year Storm

Orifice Pipe Location	=	Detention Tank		
Orifice Coefficient (C)	=	0.63	(Plate)	
Acceleration due to gravity (g)	=	9.81	(m/s/s)	
Orifice Invert	=	0.00	(m)	
High Water Level	=	2.08	(m)	100 Year
Orifice diameter	=	133	(mm)	
Cross section area of orifice (A)	=	0.0139	(sq.m.)	
Head (H)	=	2.08	(m)	
Actual Discharge (Q)	**=**	**55.9**	**(L/s)**	
$(C \times A \times (2 \times g \times H)^{0.5})$				

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E4-F

CONTROL ORIFICE DESIGN
100 Year Storm

Orifice Pipe Location	=	Detention Tank		
Orifice Coefficient (C)	=	0.63	(Plate)	
Acceleration due to gravity (g)	=	9.81	(m/s/s)	
Orifice Invert	=	0.00	(m)	
High Water Level	=	2.39	(m)	100 Year
Orifice diameter	=	133	(mm)	
Cross section area of orifice (A)	=	0.0139	(sq.m.)	
Head (H)	=	2.39	(m)	
Actual Discharge (Q)	**=**	**59.9**	**(L/s)**	
$(C \times A \times (2 \times g \times H)^{0.5})$				

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E5-A

Storage Volume Calculations - Rational Method

5-year - Town of Caledon

Project: Amelia Street Subdivision, Town of Caledon

Surface Type	Area	Coefficient
201	2.0200	0.50
202	0.0000	0.43
Total Area (ha)	2.020	
5 yr Runoff Coefficient	0.50	
Maximum Discharge Through Orifice (L/s)	32.7	

Discharged Volume per 5 min Interval (cu.m) 9.8

Time (min)	Intensity (mm/hr)	Runoff Volume (cu.m)	Discharged Volume (cu.m	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
5	2.0	1.646	1.646	0.000
10	2.2	1.827	1.827	0.000
15	2.4	2.054	2.054	0.000
20	2.8	2.345	2.345	0.000
25	3.2	2.733	2.733	0.000
30	3.9	3.272	3.272	0.000
35	4.8	4.068	4.068	0.000
40	6.4	5.347	5.347	0.000
45	9.1	7.684	7.684	0.000
50	15.5	13.020	9.804	3.216
55	39.3	33.076	9.804	23.273
60	114.3	96.195	9.804	86.391
65	51.2	43.051	9.804	33.247
70	27.5	23.102	9.804	13.298
75	17.9	15.088	9.804	5.285
80	13.0	10.952	9.804	1.148
85	10.1	8.489	8.489	0.000
90	8.2	6.880	6.880	0.000
95	6.8	5.758	5.758	0.000
100	5.9	4.937	4.937	0.000
105	5.1	4.313	4.313	0.000
110	4.5	3.824	3.824	0.000
115	4.1	3.432	3.432	0.000
120	3.7	3.112	3.112	0.000
125	3.4	2.845	2.845	0.000
130	3.1	2.620	2.620	0.000
135	2.9	2.428	2.428	0.000
140	2.7	2.262	2.262	0.000
145	2.5	2.117	2.117	0.000
150	2.4	1.990	1.990	0.000
155	2.2	1.877	1.877	0.000
160	2.1	1.777	1.777	0.000
165	2.0	1.687	1.687	0.000
170	1.9	1.605	1.605	0.000
175	1.8	1.532	1.532	0.000
180	1.7	1.465	1.465	0.000

Total Storage Volume Required (cu.m) **165.9**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E5-B

Storage Volume Calculations - Rational Method

5-year - Town of Caledon

Project: Amelia Street Subdivision, Town of Caledon

Surface Type	Area	Coefficient
201	2.0200	0.50
202	0.0000	0.43
Total Area (ha)	2.020	
5 yr Runoff Coefficient	0.50	
Maximum Discharge Through Orifice (L/s)	40.2	

Discharged Volume per 5 min Interval (cu.m) 12.1

Time (min)	Intensity (mm/hr)	Runoff Volume (cu.m)	Discharged Volume (cu.m)	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
5	3.0	2.550	2.550	0.000
10	3.4	2.844	2.844	0.000
15	3.8	3.214	3.214	0.000
20	4.4	3.690	3.690	0.000
25	5.1	4.326	4.326	0.000
30	6.2	5.213	5.213	0.000
35	7.8	6.522	6.522	0.000
40	10.2	8.616	8.616	0.000
45	14.7	12.391	12.051	0.341
50	24.6	20.700	12.051	8.649
55	57.4	48.298	12.051	36.247
60	139.3	117.212	12.051	105.161
65	72.7	61.212	12.051	49.161
70	42.1	35.444	12.051	23.393
75	28.4	23.881	12.051	11.830
80	20.9	17.574	12.051	5.523
85	16.3	13.700	12.051	1.649
90	13.2	11.121	11.121	0.000
95	11.1	9.301	9.301	0.000
100	9.5	7.959	7.959	0.000
105	8.2	6.935	6.935	0.000
110	7.3	6.131	6.131	0.000
115	6.5	5.486	5.486	0.000
120	5.9	4.958	4.958	0.000
125	5.4	4.519	4.519	0.000
130	4.9	4.148	4.148	0.000
135	4.6	3.832	3.832	0.000
140	4.2	3.560	3.560	0.000
145	3.9	3.323	3.323	0.000
150	3.7	3.115	3.115	0.000
155	3.5	2.931	2.931	0.000
160	3.3	2.767	2.767	0.000
165	3.1	2.620	2.620	0.000
170	3.0	2.489	2.489	0.000
175	2.8	2.369	2.369	0.000
180	2.7	2.261	2.261	0.000

Total Storage Volume Required (cu.m) **242.0**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E5-C

Storage Volume Calculations - Rational Method

10-year - Town of Caledon

Project: Amelia Street Subdivision, Town of Caledon

Catchment	Area	Coefficient
201	2.020	0.50
202	0.000	0.43
Total Area (ha)	2.020	
Runoff Coefficient	0.50	
Maximum Discharge Through Orifice (L/s)	46.2	

Discharged Volume per 5 min Interval (cu.m) 13.9

Time (min)	Intensity (mm/hr)	Runoff Volume (cu.m)	Discharged Volume (cu.m)	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
5	3.2	2.662	2.662	0.000
10	3.6	2.999	2.999	0.000
15	4.1	3.427	3.427	0.000
20	4.7	3.985	3.985	0.000
25	5.6	4.738	4.738	0.000
30	6.9	5.800	5.800	0.000
35	8.8	7.389	7.389	0.000
40	11.8	9.963	9.963	0.000
45	17.4	14.664	13.860	0.804
50	29.8	25.107	13.860	11.247
55	70.8	59.560	13.860	45.700
60	169.6	142.678	13.860	128.819
65	89.8	75.525	13.860	61.666
70	51.8	43.595	13.860	29.735
75	34.6	29.075	13.860	15.216
80	25.1	21.139	13.860	7.279
85	19.3	16.275	13.860	2.416
90	15.5	13.053	13.053	0.000
95	12.8	10.792	10.792	0.000
100	10.9	9.135	9.135	0.000
105	9.4	7.879	7.879	0.000
110	8.2	6.899	6.899	0.000
115	7.3	6.118	6.118	0.000
120	6.5	5.483	5.483	0.000
125	5.9	4.958	4.958	0.000
130	5.4	4.518	4.518	0.000
135	4.9	4.145	4.145	0.000
140	4.5	3.825	3.825	0.000
145	4.2	3.548	3.548	0.000
150	3.9	3.306	3.306	0.000
155	3.7	3.093	3.093	0.000
160	3.5	2.905	2.905	0.000
165	3.3	2.737	2.737	0.000
170	3.1	2.587	2.587	0.000
175	2.9	2.452	2.452	0.000
180	2.8	2.330	2.330	0.000

Total Storage Volume Required (cu.m) **302.9**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E5-D

Storage Volume Calculations - Rational Method

25-year - Town of Caledon

Project: Amelia Street Subdivision, Town of Caledon

Catchment	Area	Coefficient
201	2.020	0.50
202	0.000	0.43
Total Area (ha)	2.020	
Runoff Coefficient	0.50	
Maximum Discharge Through Orifice (L/s)	52.0	

Discharged Volume per 5 min Interval (cu.m) 15.6

Time (min)	Intensity (mm/hr)	Runoff Volume (cu.m)	Discharged Volume (cu.m)	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
5	3.7	3.075	3.075	0.000
10	4.2	3.508	3.508	0.000
15	4.8	4.061	4.061	0.000
20	5.7	4.790	4.790	0.000
25	6.9	5.782	5.782	0.000
30	8.5	7.193	7.193	0.000
35	11.1	9.314	9.314	0.000
40	15.2	12.760	12.760	0.000
45	22.6	19.021	15.604	3.417
50	38.8	32.624	15.604	17.020
55	88.2	74.184	15.604	58.580
60	192.7	162.165	15.604	146.561
65	110.1	92.661	15.604	77.057
70	66.2	55.707	15.604	40.103
75	44.8	37.720	15.604	22.116
80	32.7	27.529	15.604	11.925
85	25.1	21.157	15.604	5.553
90	20.1	16.885	15.604	1.281
95	16.5	13.869	13.869	0.000
100	13.8	11.652	11.652	0.000
105	11.8	9.970	9.970	0.000
110	10.3	8.659	8.659	0.000
115	9.1	7.616	7.616	0.000
120	8.0	6.770	6.770	0.000
125	7.2	6.073	6.073	0.000
130	6.5	5.491	5.491	0.000
135	5.9	5.000	5.000	0.000
140	5.4	4.580	4.580	0.000
145	5.0	4.218	4.218	0.000
150	4.6	3.904	3.904	0.000
155	4.3	3.629	3.629	0.000
160	4.0	3.386	3.386	0.000
165	3.8	3.171	3.171	0.000
170	3.5	2.979	2.979	0.000
175	3.3	2.807	2.807	0.000
180	3.2	2.652	2.652	0.000

Total Storage Volume Required (cu.m) **383.6**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E5-E

Storage Volume Calculations - Rational Method

50-year - Town of Caledon

Project: Amelia Street Subdivision, Town of Caledon

Catchment	Area	Coefficient
201	2.020	0.50
202	0.000	0.43
Total Area (ha)	2.020	
Runoff Coefficient	0.50	
Maximum Discharge Through Orifice (L/s)	55.9	

Discharged Volume per 5 min Interval (cu.m) 16.8

Time (min)	Intensity (mm/hr)	Runoff Volume (cu.m)	Discharged Volume (cu.m)	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
5	3.8	3.232	3.232	0.000
10	4.4	3.716	3.716	0.000
15	5.2	4.340	4.340	0.000
20	6.1	5.165	5.165	0.000
25	7.5	6.296	6.296	0.000
30	9.4	7.912	7.912	0.000
35	12.3	10.355	10.355	0.000
40	17.0	14.340	14.340	0.000
45	25.7	21.598	16.782	4.816
50	44.4	37.332	16.782	20.550
55	100.5	84.608	16.782	67.826
60	215.8	181.599	16.782	164.817
65	125.3	105.416	16.782	88.634
70	75.8	63.800	16.782	47.018
75	51.3	43.211	16.782	26.429
80	37.4	31.454	16.782	14.672
85	28.6	24.075	16.782	7.293
90	22.7	19.121	16.782	2.339
95	18.6	15.624	15.624	0.000
100	15.5	13.056	13.056	0.000
105	13.2	11.111	11.111	0.000
110	11.4	9.599	9.599	0.000
115	10.0	8.398	8.398	0.000
120	8.8	7.426	7.426	0.000
125	7.9	6.628	6.628	0.000
130	7.1	5.963	5.963	0.000
135	6.4	5.404	5.404	0.000
140	5.9	4.927	4.927	0.000
145	5.4	4.517	4.517	0.000
150	4.9	4.162	4.162	0.000
155	4.6	3.852	3.852	0.000
160	4.3	3.580	3.580	0.000
165	4.0	3.339	3.339	0.000
170	3.7	3.125	3.125	0.000
175	3.5	2.933	2.933	0.000
180	3.3	2.761	2.761	0.000

Total Storage Volume Required (cu.m) **444.4**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E5-F

Storage Volume Calculations - Rational Method

100-year - Town of Caledon

Project: Amelia Street Subdivision, Town of Caledon

Catchment	Area	Coefficient
201	2.020	0.50
202	0.000	0.43
Total Area (ha)	2.020	
Runoff Coefficient	0.50	
Maximum Discharge Through Orifice (L/s)	59.9	

Discharged Volume per 5 min Interval (cu.m) 18.0

Time (min)	Intensity (mm/hr)	Runoff Volume (cu.m)	Discharged Volume (cu.m)	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
5	4.1	3.439	3.439	0.000
10	4.7	3.983	3.983	0.000
15	5.6	4.686	4.686	0.000
20	6.7	5.622	5.622	0.000
25	8.2	6.909	6.909	0.000
30	10.4	8.756	8.756	0.000
35	13.7	11.557	11.557	0.000
40	19.2	16.139	16.139	0.000
45	29.1	24.485	17.973	6.512
50	50.5	42.498	17.973	24.525
55	113.7	95.658	17.973	77.685
60	239.4	201.417	17.973	183.444
65	141.2	118.862	17.973	100.889
70	86.2	72.565	17.973	54.592
75	58.5	49.269	17.973	31.296
80	42.6	35.844	17.973	17.871
85	32.5	27.377	17.973	9.404
90	25.8	21.677	17.973	3.704
95	21.0	17.649	17.649	0.000
100	17.5	14.691	14.691	0.000
105	14.8	12.451	12.451	0.000
110	12.7	10.711	10.711	0.000
115	11.1	9.331	9.331	0.000
120	9.8	8.217	8.217	0.000
125	8.7	7.303	7.303	0.000
130	7.8	6.544	6.544	0.000
135	7.0	5.905	5.905	0.000
140	6.4	5.363	5.363	0.000
145	5.8	4.898	4.898	0.000
150	5.3	4.496	4.496	0.000
155	4.9	4.145	4.145	0.000
160	4.6	3.838	3.838	0.000
165	4.2	3.567	3.567	0.000
170	4.0	3.326	3.326	0.000
175	3.7	3.112	3.112	0.000
180	3.5	2.919	2.919	0.000

Total Storage Volume Required (cu.m) **509.9**

VALDOR ENGINEERING INC.
File: 17121
April 2026

TABLE: E6

Table E.6: Detention Tank Stage-Storage Table

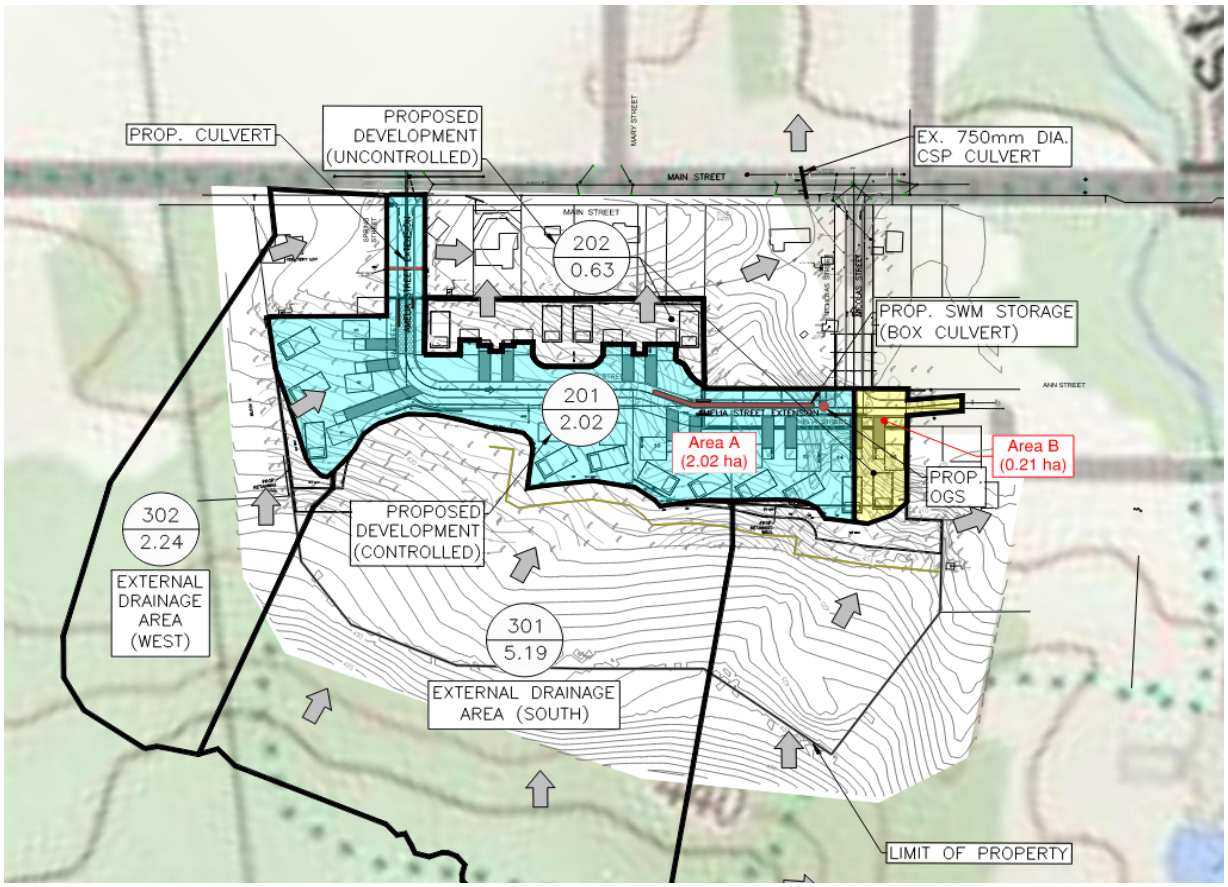
Stage Storage Curve					
Elevation	Sec Area	Avg Area	Sec Volume	Cumulative	
Width	2.4			Volume	
Length	89				
(m)	(m^2)	(m^2)	(m^3)	(m^3)	
Detention Tank					
0.00	**214**	**-**	**-**	**0**	Bottom of Tank
0.10	**214**	**214**	**21**	**21**	
0.78	**214**	214	145	**166**	2 YR HWL
1.14	**214**	214	78	**244**	5 YR HWL
1.42	**214**	**214**	**60**	**303**	10 YR HWL
1.80	**214**	**214**	**81**	**384**	25 YR HWL
2.08	**214**	**214**	**60**	**445**	50 YR HWL
2.39	**214**	**214**	**65**	**510**	100 YR HWL
2.40	**214**	**214**	**3**	**513**	TOP OF TANK

VALDOR ENGINEERING INC.
File: 17121
April 2026

Table E.7: Treatment Train TSS Removal Calculation

Treatment	Removal Efficiency
OGS	0.45
Infiltration Tank	0.7

	Area (ha)	Treatment	Removal rate	TSS removed (ha)	TSS remaining (ha)
Quality	2.23	Infiltration	0.7	**1.561**	0.669
		OGS	0.45	**0.301**	**0.368**
Total	**2.23**			**1.862**	**0.368**
				83.5%	



Project Summary Report: 17121_Amelia Street Subdivision Stormceptor Sizing

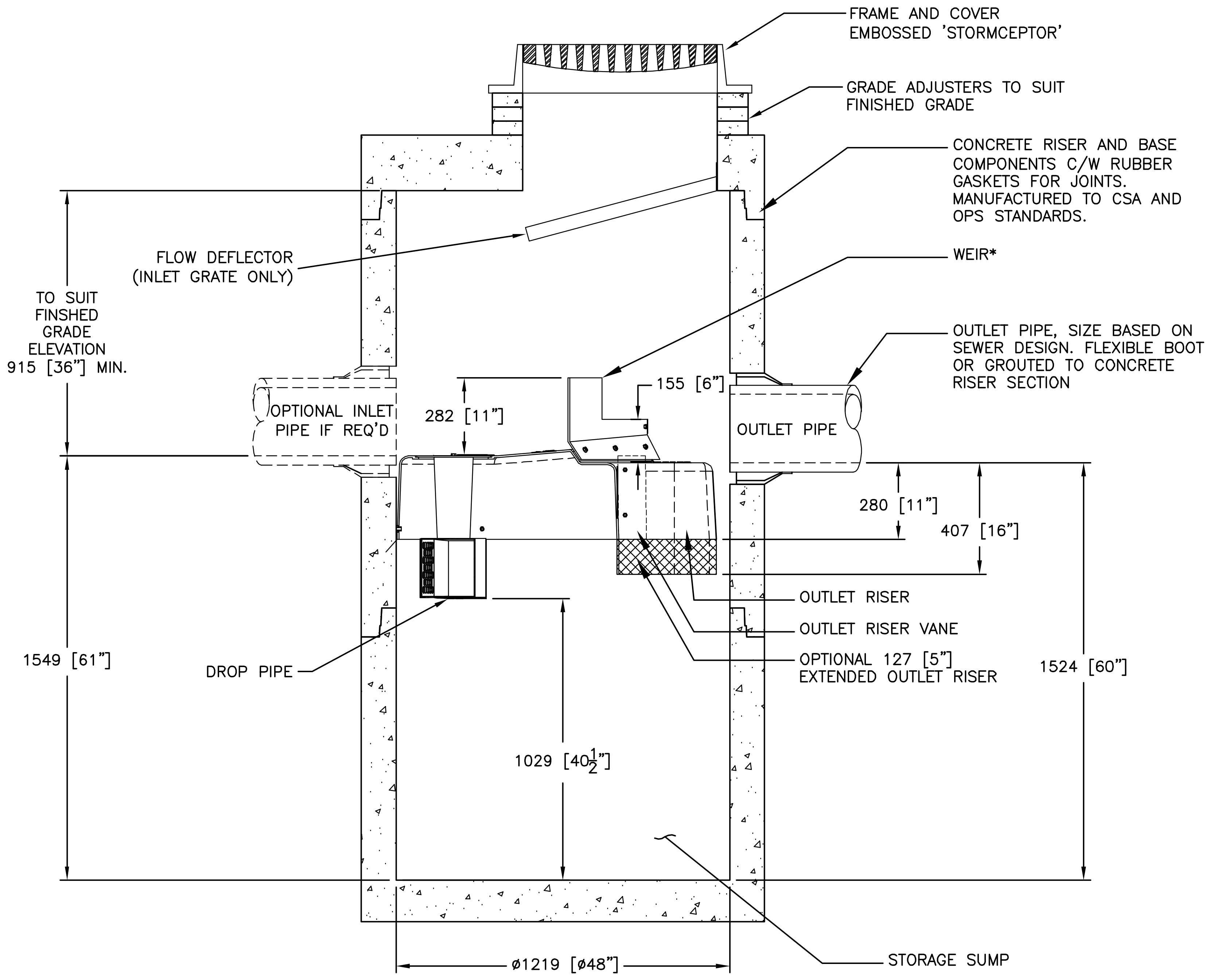
Project Information & Location			
Project Name	17121_Amelia Street Subdivision	**Project Number**	17121
City	Caledon	**State/ Province**	Ontario
Country	Canada	**Date**	4/14/2026
Designer Information		**EOR Information (optional)**	
Name	Phoebe Yung	**Name**	
Company	Valdor	**Company**	
Phone #	905-264-0054	**Phone #**	
Email	pyung@valdor-engineering.com	**Email**	

Stormwater Treatment Recommendation
The recommended Stormceptor Model(s) which achieve or exceed the user defined water quality objective for each site within the project are listed in the below Sizing Summary table.

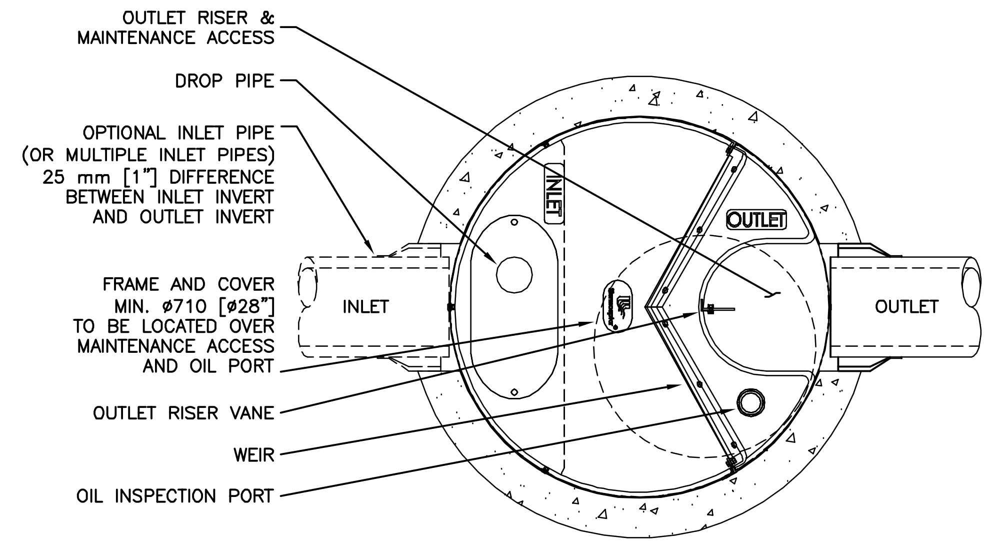
Project Summary						
Site Name	**Drainage Area (ha)**	**Imperviousness %**	**PSD**	**Target TSS Removal (%)**	**TSS Removal (%) Provided**	**Recommended Model**
	2.22	40.00		71	Parallel Models	Parallel Models

Notes

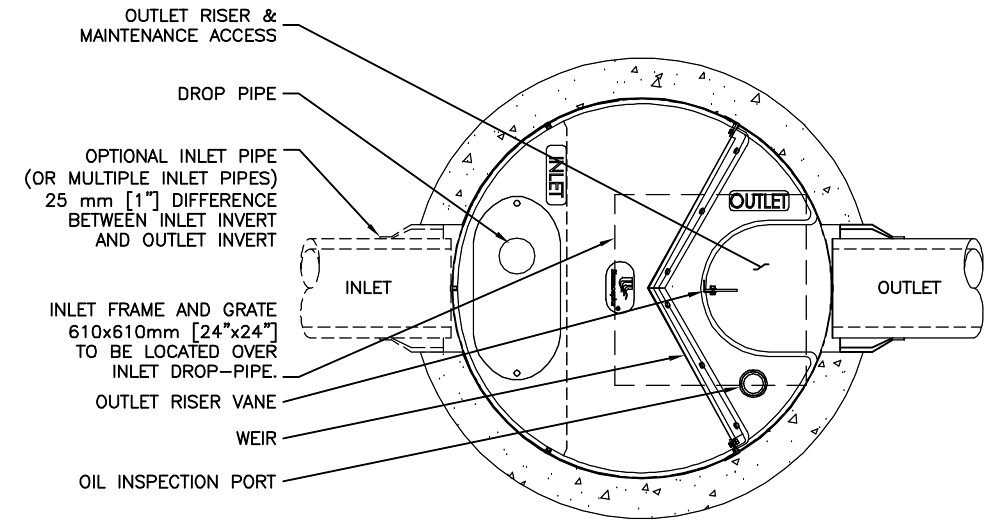
• Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor, which uses the EPA Rainfall and Runoff modules.
• Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal defined by the selected PSD, and based on stable site conditions only, after construction is completed.
• For submerged applications or sites specific to spill control, please contact your local Stormceptor representative for further design assistance.



SECTION VIEW



PLAN VIEW (STANDARD)



PLAN VIEW (INLET TOP)

FOR SITE SPECIFIC DRAWINGS PLEASE CONTACT YOUR LOCAL STORMCEPTOR REPRESENTATIVE. SITE SPECIFIC DRAWINGS ARE BASED ON THE BEST AVAILABLE INFORMATION AT THE TIME. SOME FIELD REVISIONS TO THE SYSTEM LOCATION OR CONNECTION PIPING MAY BE NECESSARY BASED ON AVAILABLE SPACE OR SITE CONFIGURATION REVISIONS. ELEVATIONS SHOULD BE MAINTAINED EXCEPT WHERE NOTED ON BYPASS STRUCTURE (IF REQUIRED).

GENERAL NOTES:

* MAXIMUM SURFACE LOADING RATE (SLR) INTO LOWER CHAMBER THROUGH DROP PIPE IS 1135 L/min/m^2 (27.9 gpm/ft^2) FOR STORMCEPTOR EF4 AND 535 L/min/m^2 (13.1 gpm/ft^2) FOR STORMCEPTOR EFO4 (OIL CAPTURE CONFIGURATION). WEIR HEIGHT IS 150 mm (6 INCH) FOR EF04.

1. ALL DIMENSIONS INDICATED ARE IN MILLIMETERS (INCHES) UNLESS OTHERWISE SPECIFIED.
2. STORMCEPTOR STRUCTURE INLET AND OUTLET PIPE SIZE AND ORIENTATION SHOWN FOR INFORMATIONAL PURPOSES ONLY.
3. UNLESS OTHERWISE NOTED, BYPASS INFRASTRUCTURE, SUCH AS ALL UPSTREAM DIVERSION STRUCTURES, CONNECTING STRUCTURES, OR PIPE CONDUITS CONNECTING TO COMPLETE THE STORMCEPTOR SYSTEM SHALL BE PROVIDED AND ADDRESSED SEPARATELY.
4. DRAWING FOR INFORMATION PURPOSES ONLY. REFER TO ENGINEER'S SITE/UTILITY PLAN FOR STRUCTURE ORIENTATION.
5. NO PRODUCT SUBSTITUTIONS SHALL BE ACCEPTED UNLESS SUBMITTED 10 DAYS PRIOR TO PROJECT BID DATE, OR AS DIRECTED BY THE ENGINEER OF RECORD.

INSTALLATION NOTES

A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE STRUCTURE (LIFTING CLUTCHES PROVIDED)
C. CONTRACTOR WILL INSTALL AND LEVEL THE STRUCTURE, SEALING THE JOINTS, LINE ENTRY AND EXIT POINTS (NON-SHRINK GROUT WITH APPROVED WATERSTOP OR FLEXIBLE BOOT)
D. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT THE DEVICE FROM CONSTRUCTION-RELATED EROSION RUNOFF.
E. DEVICE ACTIVATION, BY CONTRACTOR, SHALL OCCUR ONLY AFTER SITE HAS BEEN STABILIZED AND THE STORMCEPTOR UNIT IS CLEAN AND FREE OF DEBRIS.

STANDARD DETAIL
NOT FOR CONSTRUCTION

SITE SPECIFIC DATA REQUIREMENTS

STORMCEPTOR MODEL	EFO4				
STRUCTURE ID					*
HYDROCARBON STORAGE REQ'D (L)					*
WATER QUALITY FLOW RATE (L/s)					*
PEAK FLOW RATE (L/s)					*
RETURN PERIOD OF PEAK FLOW (yrs)					*
DRAINAGE AREA (HA)					*
DRAINAGE AREA IMPERVIOUSNESS (%)					*
PIPE DATA:	I.E.	MAT'L	DIA	SLOPE %	HGL
INLET #1	*	*	*	*	*
INLET #2	*	*	*	*	*
OUTLET	*	*	*	*	*

* PER ENGINEER OF RECORD

The design and information shown on this drawing is provided as a service to the project owner, engineer and contractor by Imbrium Systems ("Imbrium"). Neither this drawing, nor any part thereof, may be used, reproduced or modified in any manner without the prior written consent of Imbrium. Failure to comply is done at the user's own risk and Imbrium expressly disclaims any liability or responsibility for such use.

If discrepancies between the supplied information upon which the drawing is based and actual field conditions are encountered as site work progresses, these discrepancies must be reported to Imbrium immediately for re-evaluation of the design. Imbrium accepts no liability for designs based on missing, incomplete or inaccurate information supplied by others.

MARK	DATE	REVISION DESCRIPTION	BY
####	####	####	
####	####	####	
####	####	####	
1	6/8/18	UPDATES	JSK
0	5/26/17	INITIAL RELEASE	JSK

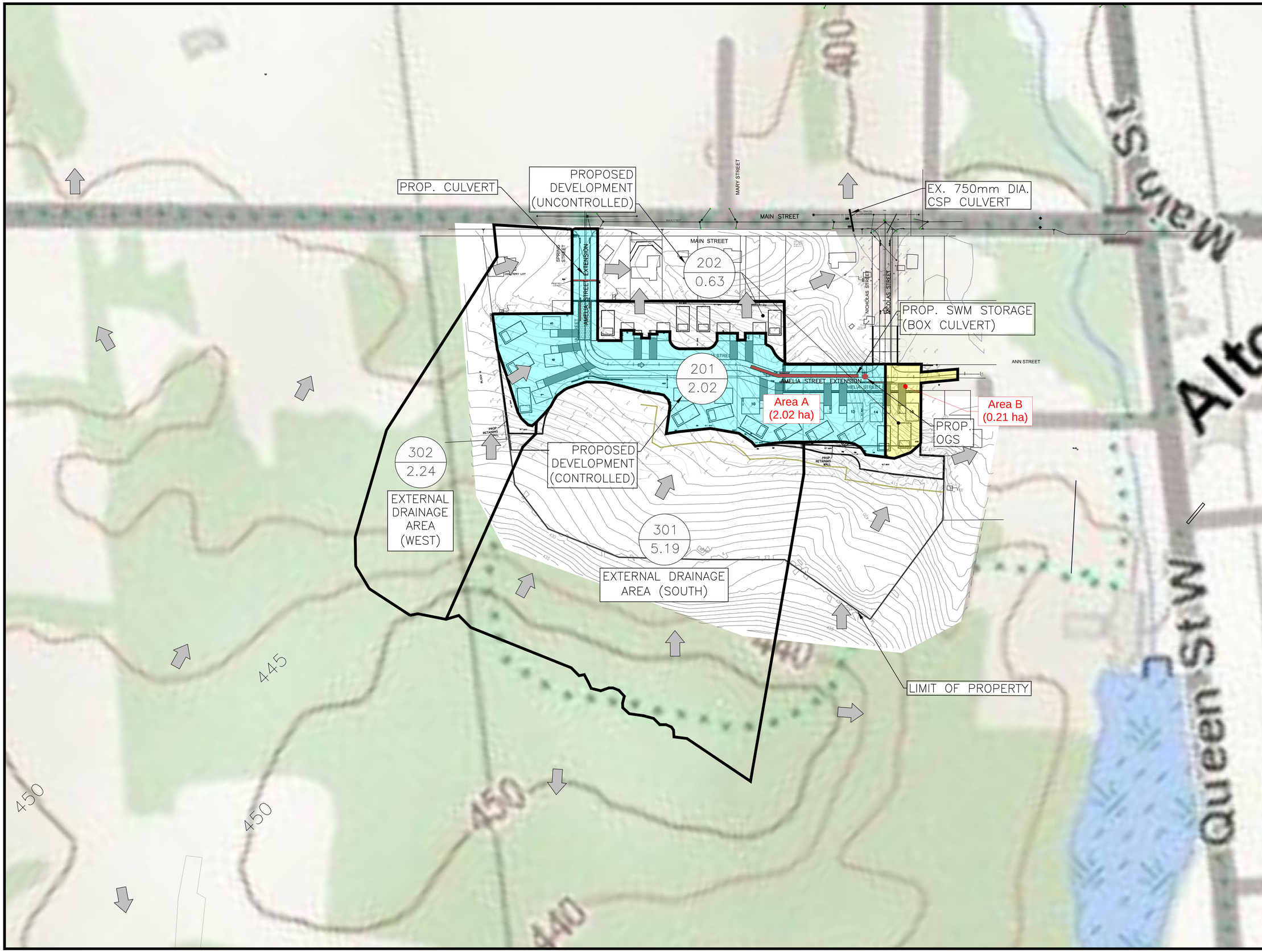
Stormceptor® EF

SCALE = NTS

imbrium®

407 FAIRVIEW DRIVE, WHITBY, ON L1N 3A9
TF 800-565-4801 CA 416-960-9900 INTL +1-416-960-9900

DATE: 10/13/2017	
DESIGNED: JSK	DRAWN: JSK
CHECKED: BSF	APPROVED: SP
PROJECT No.: EFO4	SEQUENCE No.: *
SHEET: 1 OF 1	



KEY PLAN
N.T.S

LEGEND:

301.08	EXISTING ELEVATION
188.5	EXISTING CONTOUR
2%	DRAINAGE FLOW DIRECTION AND SLOPE
× 126.50	PROPOSED ELEVATION
	STORM MANHOLE
	CATCHBASIN
	ENVIRONMENTAL PROTECTION
	REFORESTED / PROTECTION AREA
	DRAINAGE BOUNDARY
201	CATCHMENT ID
1.10	AREA (ha)
	OVERLAND FLOW

NOTE:
THIS PLAN HAS BEEN PREPARED IN CONJUNCTION WITH THE FUNCTIONAL SERVICING REPORT TO DEMONSTRATE FEASIBILITY OF THE PROPOSED DEVELOPMENT IN CONJUNCTION WITH THE DRAFT PLAN OF SUBDIVISION APPLICATION. DETAILED DESIGNS WILL BE PREPARED AT THE DETAILED SUBDIVISION ENGINEERING DESIGN STAGE.

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

POST DEVELOPMENT AREA SUMMARY		
CATCHMENT 201		
SURFACE	AREA (Ha.)	RUNOFF COEFFICIENT
LANDSCAPE	1.30	0.25
IMPERVIOUS	0.72	0.95
TOTAL	2.02	0.50
CATCHMENT 202		
SURFACE	AREA (Ha.)	RUNOFF COEFFICIENT
LANDSCAPE	0.47	0.25
IMPERVIOUS	0.15	0.95
TOTAL	0.62	0.42

NO.	DATE	REVISIONS	BY

PRELIMINARY

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
571 Chrislea Road, Unit 4, 2nd Floor, Woodbridge, Ontario, L4L 8A2
TEL (905)264-0054, FAX (905)264-0069
E-MAIL: info@valdor-engineering.com
www.valdor-engineering.com

AMELIA STREET SUBDIVISION

COMMUNITY OF ALTON
TOWN OF CALEDON
REGION OF PEEL

STORM DRAINAGE PLAN POST-DEVELOPMENT

SCALE:	DATE OF DWG.	PROJECT NO.
1:1250	APR. 06, 2022	17121
DRAWN BY: P.Y.	DRAWING NO.	
CHKD BY D.G.	FIGURE 4	

Alton Village Drainage Study - Phase 2

Part 2 Drainage Study Report - 90% Draft

Town of Caledon

DRAFT

R.J. Burnside & Associates Limited
3 Ronell Crescent
Collingwood ON L9Y4J6

December 2019
300043323.0000

Following the public open house, a memorandum detailing interim maintenance measures was prepared by Burnside and submitted to the Town on July 19, 2019. This memorandum was used by the Town as a guide to address some residents' concerns ahead of proposed construction measures recommended in this report. The interim measures have been provided in Appendix H.

This report includes an overview of the hydrological modelling completed for the study area, an evaluation of existing conditions of the site, an overview of the preliminary design for the preferred drainage improvement alternatives and a cost estimate of the recommended alternatives.

2.0 Existing Conditions Overview of Study Area

Three areas within the Village have been identified as locations that frequently experience drainage issues during storm events. The Town has recognized the need to improve the existing drainage infrastructure within the public right-of-way (ROW) to reduce the impact of the drainage issues in these areas. The locations are shown in Figure 1.

In the Part 1 Drainage Characterization Report recommendations to property owners were provided to help protect their property against flooding that originates from private properties. These recommendations will not be explored further in this report.

2.1 Area 1 – Main Street (#19871, #19876 and #19877)

Area 1 encompasses three properties on Main Street, #19871, #19876 and #19877 Main Street. These properties are located at the bottom of the hill on Main Street. Uphill on the west side of the street, in front of #19904 Main Street, there is a low spot in the right-of-way which allows runoff to enter private properties and contribute to ponding behind #19876 Main Street. The properties on Main Street are designed to outlet through a channel on the east side of the street between #19871 and #19877 Main Street. The channel is eroding onto the private properties. During a rain event in February 2018 runoff exceeded the capacity of the ditches on Main Street and flooded the basement of #19877 Main Street. Council has committed to reduce runoff from the right-of-way away from #19871, #19876 and #19877 Main Street.

During the public open house, the property owner of #19877 Main Street reported that the ditch inlet catchbasin (DICB) in front of his property is not properly sealed, causing runoff to seep from the catchbasin, undermining the house foundation. Field observations made by Burnside indicate that the catchbasin requires maintenance to function properly; however, Burnside has not substantiated the cause of the damages to private property.

DRAFT

Another DICB located in front of #19883 Main Street is intended to capture the east Main Street ditch runoff and drain it into the channel between #19871 and #19877 Main Street. The inlet to the DICB is higher than the ditch invert, allowing runoff to bypass the DICB. Burnside has recommended to the Town as a maintenance measure that the inlet of the DICB should be lowered to match the invert of the ditch.

DRAFT

3.0 Proposed Drainage Mitigation Measures

The proposed mitigation measures for each area have been evaluated based on criteria set out in a decision matrix developed by Burnside with input from the Town. The decision matrix has been provided in Appendix A. If the proposed measures had a high score in the decision matrix, they were modelled in PCSWMM to determine if they would be feasible from a hydraulic and hydrologic perspective. The modeling is discussed in the Section 5.0 of this report.

3.1 Area 1 – Main Street (#19871, #19876 and #19877)

The objective for any of the proposed mitigation measures on the Main Street area is to contain runoff generated on Main Street within the right-of-way. The proposed mitigation measures will not address drainage problems originating from private property, as this is outside of the scope of work.

Three options have been proposed for this area. The evaluation for each option using the decision matrix is provided in Appendix A.

Option 1 is to regrade the low spot in front of #19904 Main Street and install a culvert connecting the low point to the ditch on the west side of Main Street further south, to provide positive drainage relief along the Main Street right-of-way.

Option 2 proposed is to install a storm sewer along Main Street, starting at the low spot in front of #19904 Main Street. However, the objective of this study is to provide interim drainage solutions and construction of a storm sewer is a long-term solution. This option is being investigated in the class EA for Queen Street West and Main Street North currently being undertaken by the Town. As such, Option 2 is not carried forward as alternative in this study.

Option 3 is to regrade the ditches along the west side of Main Street. The ditch regrading is proposed to begin following the driveway in front of #19930 Main Street and continue along the west half of the right-of-way to the cross-road culvert in front of #19876 Main Street. The ditch is proposed to have a depth of 0.45 m and a maximum side slope of 2:1. The ditch will capture runoff generated on Main Street, and bypass the low spot, reducing the amount of runoff contributing to the rear yard of #19876 Main Street. Where necessary a small berm will be provided to create the back slope of the ditch.

To address the drainage problems in front of #19871 and #19877 Main Street, it is proposed to direct the runoff on the east side of Main Street towards the Margaret Street ditch starting at #18971 Main Street. This will reduce the amount of runoff that is directed towards the swale between #19871 and #19877 Main Street. It will also help to contain runoff on the Main Street right-of-way and off private property. To redirect the

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drainage, it is proposed to regrade the ditch on the east side of Main Street to slope towards the south, instead of the north. Due to existing grading restraints it is not feasible to grade the ditch with a continuous positive slope towards Margaret Street. It is proposed to put a catchbasin within the ditch and pipe the runoff towards the Margaret Street ditch. It has been reported by the property owner of #19871 Main Street that runoff within the east Main Street ditch seeps through the soil and impacts the house foundation. It is recommended that any ditch improvements include the addition of an impervious liner to help mitigate this issue. This mitigation measure is included in the evaluation of both Options 1 and 3.

The decision matrix was used to evaluate Options 1 and 3. As discussed, Option 2 is not considered as an alternative because it is a long-term solution. Option 3 scored higher than Option 1 on the decision matrix. From a project cost perspective, Option 3 is anticipated to have a lower capital cost than Option 1. The watermain on Main Street runs under the boulevard and will likely need relocation to install a culvert. Construction of the ditch may result in reduced cover over the watermain; however, the watermain can be insulated to prevent freezing. It is anticipated that the construction of a ditch along Main Street could be implemented sooner than the installation of the culvert and therefore the technical effectiveness of Option 3 is higher than Option 1. In terms of the natural environment, conveying the drainage through a ditch will help maintain the existing sites water balance and water quality, as the ditch will naturally filter and promote infiltration of runoff. Both Options 1 and 3 scored the same in the climate change and social/cultural services categories. Option 3 was carried forward and modelled in PCSWMM.

3.2 Area 2 – James Street and Credit Street (15 Credit Street and 1242 Queen Street)

The objective for the James Street mitigation measures is to contain the road runoff within the right-of-way.

One of the drainage issues facing the James Street area is that runoff drains off of the right-of-way to the east and onto 15 Credit Street during storm events. To contain the runoff within the right-of-way it is proposed to construct a shallow swale on the east side of James Street. To fit within the right-of-way the swale is proposed to be 0.15 m deep with the flattest side slopes that will fit within the right-of-way.

The James Street area currently outlets to the Alton Mill Pond. An 800 mm CSP culvert extends from the downstream end of the west James Street ditch in behind 1302 Queen Street and into the Alton Mill Pond. To Burnside's knowledge the culvert does not have an easement. To provide an outlet for the proposed ditch on the east side on James Street, it was proposed to connect the ditch to the culvert through a

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Figure 2 - Subcatchment Schematic

APPENDIX “F”

Water Balance Calculations

VALDOR ENGINEERING INC. **TABLE: F1**
File: 17121
April 2026

WATER BALANCE CALCULATIONS: 5 mm RETENTION VOLUME REQUIRED

1. INITIAL ABSTRACTION

Surface Type	Area (Ha)	Init. Abstract. (mm)
Catchment 201		
Impervious Area	0.72	0.0
Landscape Area	1.30	5.0
Catchment 202		
Impervious Area	0.16	0.0
Landscape Area	0.47	5.0
Total	**2.65**	**3.3**

2. STORAGE VOLUME REQUIRED

Total Area of Site (A) = 2.65 ha
Target Retention Depth (D) = 5.0 mm
Overall Initial Abstractions (I) = 3.3 mm

Storage Volume Required = V = A x (D - I) = **44.0** cu.m.

3. INFILTRATION TRENCH MAXIMUM DEPTH

In-Situ Percolation Rate = 51.9 mm/hr
Safety Factor = 2.5
Design Percolation Rate (P) = 20.7

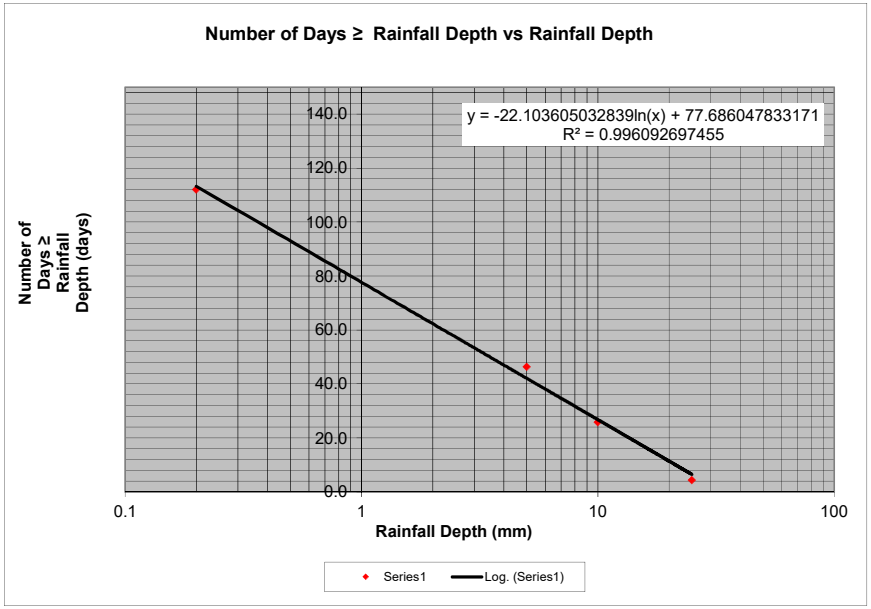
Maximum Retention Time (T) = 48 hours

Maximum Chamber Height Allowable (D) = (PT/(1000*S)) = 1.00 m
*MOE recommends max depth of 1.5m

Clear Stone Trench	Length (m)	Width (m)	Height (m)	Void Ratio	Storage Volume Provided (m³)
Infiltration Trench #1	20.0	1.0	1.00	0.40	8
Infiltration Trench #2	10.0	1.0	1.00	0.40	4
Infiltration Trench #3	10.0	1.0	1.00	0.40	4
Infiltration Trench #4	10.0	1.0	1.00	0.40	4
Infiltration Trench #5	15.0	1.0	1.00	0.40	6
Infiltration Trench #6	15.0	1.0	1.00	0.40	6
Infiltration Trench #7	25.0	1.0	1.00	0.40	10
Infiltration Trench #8	5.0	1.0	0.60	0.40	1.2
Infiltration Trench #9	5.0	1.0	0.60	0.40	1.2
				Total:	**44.40**

Table F.2: Rainfall Analysis Initial Abstraction = 3.0 mm

VALDOR ENGINEERING INC.
File: 17121
Date: April 2022



Normal Rainfall Depth (mm)	Normal Days ≥ Rainfall Depth (days)
0.2	112.0
5	46.4
10	25.9
25	4.4

Orangeville MOE Climate Normals (1981 - 2010)
750.1 Normal Annual Rainfall Depth (mm)
112 Normal Annual Days with Rainfall (≥ 0.2 mm)
901.5 Normal Annual Precipitation Depth (mm)

Simulated Depth (mm)	Simulated Days ≥ Sim Depth (days)	Average Event Depth (mm)	Simulated Days Equal to Avg Depth (days)	Assumed IA (mm)	Runoff (Rain - IA) (mm)	INF Design Storm (mm)	Event Based Maximum Design INF Depth (mm)	Event Based Design INF Depth (mm)	Annual Incremental Design INF Depth (mm)	Annual Cumulative Design INF Depth (mm)	Annual Incremental Total Rain Depth (mm)	Annual Percent of Total Rain (%)	Annual Cumulative Total Rain Depth (mm)	Annual Cumulative Percent of Total Depth (%)
0.2	113.26													
0.5	93.01	0.2 - 0.5	20.25	3.00	0.00	15.00	12.00	0.00	0.00	**0.00**		0.000	0.0	0.000
1.5	68.72	1	24.28	3.00	0.00	15.00	12.00	0.00	0.00	**0.00**	24.28	0.032	24.3	0.032
2.5	57.43	2	11.29	3.00	0.00	15.00	12.00	0.00	0.00	**0.00**	22.58	0.030	46.9	0.062
3.5	50.00	3	7.44	3.00	0.00	15.00	12.00	0.00	0.00	**0.00**	22.31	0.030	69.2	0.092
4.5	44.44	4	5.55	3.00	1.00	15.00	12.00	1.00	5.55	**5.55**	22.22	0.030	91.4	0.122
5.5	40.00	5	4.44	3.00	2.00	15.00	12.00	2.00	8.87	**14.43**	22.18	0.030	113.6	0.151
6.5	36.31	6	3.69	3.00	3.00	15.00	12.00	3.00	11.08	**25.50**	22.15	0.030	135.7	0.181
7.5	33.15	7	3.16	3.00	4.00	15.00	12.00	4.00	12.65	**38.16**	22.14	0.030	157.9	0.210
8.5	30.38	8	2.77	3.00	5.00	15.00	12.00	5.00	13.83	**51.99**	22.13	0.030	180.0	0.240
9.5	27.92	9	2.46	3.00	6.00	15.00	12.00	6.00	14.75	**66.74**	22.13	0.029	202.1	0.269
10.5	25.71	10	2.21	3.00	7.00	15.00	12.00	7.00	15.49	**82.22**	22.12	0.029	224.3	0.299
11.5	23.70	11	2.01	3.00	8.00	15.00	12.00	8.00	16.09	**98.31**	22.12	0.029	246.4	0.328
12.5	21.86	12	1.84	3.00	9.00	15.00	12.00	9.00	16.59	**114.90**	22.12	0.029	268.5	0.358
13.5	20.16	13	1.70	3.00	10.00	15.00	12.00	10.00	17.01	**131.91**	22.11	0.029	290.6	0.387
14.5	18.58	14	1.58	3.00	11.00	15.00	12.00	11.00	17.37	**149.28**	22.11	0.029	312.7	0.417
15.5	17.10	15	1.47	3.00	12.00	15.00	12.00	12.00	17.69	**166.97**	22.11	0.029	334.8	0.446
16.5	15.72	16	1.38	3.00	13.00	15.00	12.00	12.00	16.58	**183.56**	22.11	0.029	356.9	0.476
17.5	14.42	17	1.30	3.00	14.00	15.00	12.00	12.00	15.61	**199.16**	22.11	0.029	379.0	0.505
18.5	13.19	18	1.23	3.00	15.00	15.00	12.00	12.00	14.74	**213.90**	22.11	0.029	401.2	0.535
19.5	12.03	19	1.16	3.00	16.00	15.00	12.00	12.00	13.96	**227.87**	22.11	0.029	423.3	0.564
20.5	10.92	20	1.11	3.00	17.00	15.00	12.00	12.00	13.26	**241.13**	22.11	0.029	445.4	0.594
21.5	9.87	21	1.05	3.00	18.00	15.00	12.00	12.00	12.63	**253.76**	22.11	0.029	467.5	0.623
22.5	8.87	22	1.00	3.00	19.00	15.00	12.00	12.00	12.06	**265.82**	22.11	0.029	489.6	0.653
23.5	7.90	23	0.96	3.00	20.00	15.00	12.00	12.00	11.53	**277.36**	22.11	0.029	511.7	0.682
24.5	6.98	24	0.92	3.00	21.00	15.00	12.00	12.00	11.05	**288.41**	22.11	0.029	533.8	0.712
25.5	6.10	25	0.88	3.00	22.00	15.00	12.00	12.00	10.61	**299.02**	22.11	0.029	555.9	0.741
26.5	5.25	26	0.85	3.00	23.00	15.00	12.00	12.00	10.20	**309.22**	22.11	0.029	578.0	0.771
27.5	4.43	27	0.82	3.00	24.00	15.00	12.00	12.00	9.82	**319.05**	22.11	0.029	600.1	0.800
28.5	3.64	28	0.79	3.00	25.00	15.00	12.00	12.00	9.47	**328.52**	22.11	0.029	622.2	0.830
29	3.26	≥ 29	3.26	3.00	26.00	15.00	12.00	12.00	39.08	**367.60**	127.87	0.170	750.1	1.000

Table F.3: Infiltration Trench Calculation

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Project No: 17121
Date: April 2026

Total Req'd Annual Infiltration Volume to Achieve Target (m^3)	*Total Actual Annual Infiltration Volume per Design (m^3)*	*Soil Infiltration Rate (mm/h)*	*Equivalent Drainage Area (ha)*	*Maximum Trench Length per Site Plan (m)*	*Initial Abstraction (Trench Drainage Area) (mm)*	*Retention Time (hr)*	*Total Annual Rainfall Depth (Per 1981-2010 Climate Normals for Peterborough Airport) (mm)*	*Total Rainfall Depth Available for Infiltration Per Rainfall Analysis Assuming Ia=3.0mm (Refer to Table F.5) (mm)*	*Annual Rainfall Depth Needed to Achieve Target Infiltration (mm)*	*[1] Req'd Design Storm Depth to Achieve Annual Infiltration Requirements*	*Req'd Event-Based Runoff Volume to be Infiltrated (Based on Req'd Design Storm Depth) (m^3)*
866	1,360	**30**	**0.37**	**-**	**3.0**	**48**	**750.1**	**367.60**	234.1	**15.0**	44.4

Total annual infiltration volume provided (m^3/yr): **1,360**

Maximum Allowable Depth	
P, Soil Infiltration Rate (mm/h):	**30**
T, Drawdown Time (hr):	**48**
d, Maximum Allowable Depth (m):	**1.44**

Minimum Bottom Area	
V, Runoff Volume to Infiltrated (m^3):	**44**
P, Soil Infiltration Rate (mm/h):	**30.0**
n, Void Ratio (clear stone):	**0.40**
Δt, Drawdown Time (hr):	**48**
A, Minimum Bottom Area (m^2):	**77**

$$d = \frac{P \cdot T}{1000} \quad \textit{Equation 4.2, Stormwater Management Planning and Design Manual, MOE, 2003}$$

$$A = \frac{1000 \cdot V}{P \cdot n \cdot \Delta t} \quad \textit{Equation 4.3, Stormwater Management Planning and Design Manual, MOE, 2003}$$

Infiltration Trench Design

Infiltration Trench Location	**Drainage Area (ha)**	**Available Infiltration Volume (m^3)**	**Length (m)**	**Width (m)**	**[3]Design Depth (m)**	**Bottom Area (m^2)**	**Void Ratio**	**Storage Volume Provided (m^3)**	**Lesser of Available Infiltration Volume or Storage Volume Provided (m^3)**
Infiltration Trench #1	0.35	42.0	20.0	1.0	1.00	20	0.40	8.0	8.0
Infiltration Trench #2	0.11	13.2	10.0	1.0	1.00	10	0.40	4.0	4.0
Infiltration Trench #3	0.08	9.0	13.0	1.0	1.00	13	0.40	5.2	5.2
Infiltration Trench #4	0.08	9.0	13.0	1.0	1.00	13	0.40	5.2	5.2
Infiltration Trench #5	0.07	7.8	15.0	1.0	1.00	15	0.40	6.0	6.0
Infiltration Trench #6	0.07	7.8	15.0	1.0	1.00	15	0.40	6.0	6.0
Infiltration Trench #7	0.11	13.2	25.0	1.0	1.00	25	0.40	10.0	10.0
								Total:	44.40

Total Drainage Area (ha): **0.85**
Total Bottom Area Provided (m^2): **111**
Total Infiltration Volume Used (m^3): **44.4**

Notes:

Infiltration facilities are sized based on the following criteria (SWMPDM, MOE, 2003) and/or assumptions:

(1) Infiltration trench volume should be sized based on the runoff generated by a 4-hr 15-mm event or smaller.
(2) The drainage area to each infiltration trench should be sufficient to provide required runoff quantity.
(3) The maximum allowable depth of the infiltration facility is based on the soil infiltrate rate and the retention time.
(4) It is feasible to convey the runoff to the infiltration facility.
(5) The seasonal high water table should be at least 1 m below the infiltration trench.

74 Berkeley Street, Toronto, ON M5A 2W7
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Hydrogeological Investigation

0 Nicholas Street, Alton Village, Town of Caledon, ON

Palmer Project #

1904308

Prepared For

Goodman, Solomon & Gold

January 24, 2022

LEGEND:

- Borehole Location
- Monitoring Well Location
- Watercourse
- Subject Site

Imagery (2018) provided by Esri basemap service. Contains information licensed under the Open Government Licence – Ontario

0 50 100
metres

N

PROJECT NO.	1904308	REVISION:	1-1
DATE:	Jun 22, 2021	SCALE:	1:3000
DRAWN:	CV	DATUM:	NAD 1983
CHECKED:	SH	PROJECTION:	UTM zone 17

CLIENT: The Biglieri Group

PREPARED BY: Palmer™

PROJECT: Alton Village

TITLE:

Site Location

FIGURE 1

Document Path: G:\Shared drives\Projects 2019\19043 - The Biglieri Group\1904308 - Alton Village\Mapping\Figures\5_ArcGIS\HydroG\1904308_1-1_Site Location.mxd

Topsoil and Fill Material.

A 220 mm to 330 mm thick surficial layer of topsoil was found at all borehole locations. Fill material was encountered at all borehole locations except for BH6. The fill ranges from 0.8 m to 2.3 m and consists of mainly sand and silty sand with trace clay and topsoil.

Cohesionless Silty and Sandy Deposits

The native soil underlying the fill material in all boreholes consists of cohesionless material of sand, sandy silt to silty sand, silt and some sand and gravel. This layer was not fully penetrated in BH1, BH5 and BH8.

Sandy Silt to Silty Sand Till

The silty and sandy deposits in BH6 was underlain by glacial till deposits comprising of sandy silt to silt sand soils. The layer was observed to extend from 6.1 m to the bottom of the borehole. The sand was found to be in a loose to dense state and was not fully penetrated. This layer was also observed at depth in BH4.

Clayey Silt to Silt Clay Till

A cohesive layer of clayey silt to silty clay was observed in BH7 underlying the sand to silty sand layer. This layer extended from 10.7 m to the bottom of the borehole. The later was found to be in a stiff and hard state and was not fully penetrated.

3.3 Water Level Monitoring and Groundwater Flow

Groundwater levels were measured manually at installed MWs at the site by SPCL on July 13, 2017 and by Palmer on May 05, May 18, and June 23, 2021. Measurements were made using a water level tape and recorded to the nearest centimeters. In addition, a datalogger, programmed to record data at intervals of one hour, was installed in BH6 to continuously record water levels on-site before, during and following site grading and construction. Manual groundwater level results are summarized in **Table 2**, below. During Palmer's monitoring program to date, the depth to water table ranged from 1.73 to 3.73 mbgs, corresponding to an elevation range of 401.97 to 415.72 metres above sea level (mASL). As indicated on Figure 6, shallow horizontal groundwater flow is directed east-northeastwards reflecting the influences of local surface water features and topography on the hydraulic regime. It is important to consider that groundwater levels are subject to fluctuations due to seasonal trends and precipitation input

Table 2. Groundwater Level Monitoring Results

MW ID	Grade Elevation (mASL)	Depth (mbgs)	Unit	Water Level			
				July 13, 2017	May 05, 2021	May 18, 2021	June 23, 2021
BH1	418.91	6.02	mbgs	2.9	3.29	3.19	3.47
			mASL	416.0	415.62	415.72	415.44
BH5	Not available	6.80	mbgs	2.7	3.05	3.06	3.39
			mASL	-	-	-	-
BH6	415.85	10.50	mbgs	2.9	3.36	-	3.63
			mASL	414.95	412.49	-	412.22
BH7	403.93	12.56	mbgs	1.3	1.73	1.75	1.97
			mASL	403.6	402.20	402.18	401.97

3.3.1 Hydraulic Conductivity Testing and Infiltration Rate Estimates

Single Well Response Tests (SWRT) were performed by Palmer at all monitoring wells to estimate the hydraulic conductivity of the soil materials surrounding the well screen at each location. Tests were completed by lowering a 1 L slug into each well and then rapidly removing it to create a change in hydraulic head. Hydraulic conductivity values were estimated by measuring the rate of change in recovery of the water level (also known as a Rising Head Test). Water levels in each well were recorded using a datalogger set to record either every one or two seconds. The tests were terminated once either 80% recovery had been attained, or 30 minutes had elapsed.

Hydraulic conductivity values were calculated from the displacement-time data using the Bouwer & Rice (1976, 1989) Method for an unconfined aquifer as implemented in a spreadsheet distributed by the United States Geological Survey. The results are presented in **Appendix D.1** and included in **Table 3,** below. The calculated hydraulic conductivities range from 2.0×10^{-7} m/s to 6.4×10^{-6} m/s with a geometric mean of 2.0×10^{-6} m/s. The results assume fully penetrating conditions. Hydraulic conductivity decreases slightly if a thicker aquifer/partial penetration is assumed.

In addition, the hydraulic conductivity of the soil samples collected during SPCL's 2017 geotechnical investigation were estimated from grain size distribution data using the spreadsheet HydrogeoSieveXL ver 2.2. (Devlin, 2016). This spreadsheet contains 15 different methods of estimating hydraulic conductivity; however, only the results of those methods for which the sample met acceptance criteria are used. The HydrogeoSieveXL output sheets are provided in **Appendix D.2** and the results are included in **Table 3**, below. The calculated hydraulic conductivities range from 3.8×10^{-7} m/s to 2.3×10^{-6} m/s with a geometric mean of 1.2×10^{-6} m/s.

An estimation of the percolation rate of the soils within the site was produced empirically using the method developed by Toronto and Region Conservation Authority (TRCA) (2012) and Credit Valley Conservation (CVC) (2010). The results of the calculations are also provided in **Table 3,** below. The estimated percolation rate of the tested soils ranged from 30.1 to 76.0 mm/hr (T-time of 19.9 to 7.9 min/cm) with a geometric mean of 51.9 mm/hr (T-time of 11.6 min/cm). When applying a default safety factor of 2.5, the geometric mean infiltration rate is 20.7 mm/hr (T-time of 29.9 min/cm). It is recommended that this rate be used for preliminary design purposes until future site-specific percolation rate testing is completed.

3.3.2 Groundwater Chemistry

Groundwater samples were collected on May 04, 2021 from BH6 and BH7. The samples were submitted to ALS Canada Ltd. (ALS) for analysis for a suite of general potability parameters including turbidity, TSS, TDS, pH, metals, and cations and anions. ALS is accredited by the Canadian Association for Laboratory Accreditation (CALA). A copy of the Certificate of Analysis is provided in **Appendix E**.

The results were compared against the 2006 Ontario Drinking Water Standards, Objectives and Guidelines (ODWSOG), 2006 and the Ontario Drinking Water Quality Standards (ODWQS), 2020.

A summary of the groundwater analytical results is presented in **Table 4**, below, with the Certificate of Analysis provided in **Appendix E**. Both samples exceeded the ODWSOG Aesthetic Objectives (AOs) for colour and turbidity and the ODWSOG Operational Guideline (OG) for hardness. Sample BH6 exceeded the ODWQS for sodium but not its ODWSOG AO. Sample BH7 exceeded the ODWSOG AO for manganese.

Table 3. Summary of Hydraulic Conductivity Values and Infiltration Rates

Location	Test Used	Depth (m)	Soil Description	K (m/s)	Infiltration Rate (mm/hour)	Infiltration Rate (min/cm)
BH1	Slug Test	3.3 - 6.4	Sandy silt to sand	$2.0x10^{-07}$	30.1	19.9
BH5	Slug Test	4.5 - 7.6	Sandy silt/sand and gravel	$6.4x10^{-06}$	76.0	7.9
BH6	Slug Test	7.3 - 10.4	Silty sand	$3.2x10^{-06}$	63.2	9.5
BH7	Slug Test	9.6 - 12.7	Silt to silty clay	$3.7x10^{-06}$	65.7	9.1
BH1	Grain size – SS4	2.3 - 2.9	Sandy silt/silty sand, trace clay	$3.8x10^{-07}$	35.7	16.8
BH2	Grain size distribution – SS3	1.5 - 2.1	Sand and gravel ,trace silt	$2.1x10^{-06}$	56.4	10.6
BH3	Grain size distribution – SS6	4.4 - 5.0	Sand, trace silt	$1.1x10^{-06}$	47.5	12.6
BH7	Grain size distribution – SS3	1.5 - 2.1	Silty sand, trace clay	$2.3x10^{-06}$	57.8	10.4

4. Water Balance

4.1 Water Budget Methodology

4.1.1 Water Surplus

The water surplus described the difference between precipitation and evapotranspiration (ET) to estimate the amount of water or surplus that is available to contribute to infiltration and runoff. The surplus was calculated using the monthly soil-moisture balance approach as described in Thornthwaite and Mather (1957). Surplus on impermeable surfaces is assumed to be 90% of the precipitation. A soil moisture storage value of 200 mm was used, which is appropriate for silt loam and moderately – deep rooted crops. Data for average monthly precipitation and temperature was derived from the 1981 – 2010 climate normals from the Orangeville climate station (ID 6155790). Actual evapotranspiration is calculated based on a potential ET (or PET) and soil-moisture storage withdrawal. Monthly PET is estimated using monthly temperature data and is defined as a water loss from a homogeneous vegetation-covered area that never lacks water (Thornthwaite, An Approach Toward a Rational Classification of Climate, 1948).

4.1.2 Infiltration Factors

The partitioning of the water surplus between runoff and infiltration depends on soil type, topography and vegetation cover. Water will infiltrate more easily through sands compared to clays, on flat slopes compared to steep sloped, and through natural vegetated soils compared to agricultural crops or urban areas. The method developed by Bernard (1932) and described by the Ministry of the Environment and Energy (MOEE) (1995) was used to estimate infiltration for the site. The infiltration factor is calculated by adding the individual sub-factors at the site. The water surplus is then multiplied by the total infiltration factor to determine the partitioning between the amount of runoff and the amount of infiltration that occurs annually. The runoff is the total amount of surplus remaining after taking into account infiltration (1 – infiltration factor = runoff factor).

4.2 Water Budget Results

4.2.1 Pre-Development Conditions

For reference, the CVC Water Balance Tool (CVC, 2021) reports precipitation of 897 mm/yr, infiltration of 370 to 381 mm/yr (lower values are on the plateau to the west), evapotranspiration of approximately 402 mm/yr and runoff of approximately 114 mm/yr. However, it should be noted that the eastern flank of the most prominent hill in the area comprises the western portion of the site.

The total yearly precipitation recorded by the Orangeville Meteorological Station was 901.5 mm/year (**Table 5.**). The calculated actual ET (or AET) based on the Thornthwaite and Mather monthly water balance model is approximately 559 mm/year (**Table 5**). The estimated water surplus for the site area is approximately 343 mm/year (38% of the total precipitation). The surplus values are given below in **Table 5.**

Based on OGS surficial geology mapping and borehole data, the site is underlain by sand to silty sand, treated as “open sandy loam” under MOE, 1995 (infiltration factor of 0.4), the site has hilly topography (infiltration factor 0.1), and the land use on-site is woodland (infiltration factor of 0.2). **Table 6.** presents the soil infiltration factors used for the site-wide water budget.

Using the method in the MOE stormwater management (SWM) manual and MOEE (1995) for guidance, it is estimated that approximately 70% (240 mm/year) of the surplus infiltrates, and the remaining 30% (102.8

mm/year) runs off. Based on a site area of 5.91 ha, it is estimated that 14,181.4 m^3/year infiltrates and 6,077.8 m^3/year runs off under existing conditions. The results are summarized in **Table 7.**.

Table 5. Water Surplus

Water Balance	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Precipitation (mm)	64.3	54.5	60.9	70.1	86.6	81.3	80.8	88.2	87	76.6	87.1	64.2	901.6
Temperature (°C)	-7.5	-6.5	-2.1	5.3	11.7	16.9	19.4	18.4	14.3	7.8	2.0	-4.1	6.3
Potential Evapotranspiration (PET) (mm)	0.0	0.0	0.0	29.5	75.1	110.3	129.1	113.1	75.4	36.8	7.7	0.0	577.0
P – PET	64.3	54.5	60.9	40.6	11.5	-29.0	-48.3	-24.9	11.6	39.8	79.4	64.2	325.0
Change in Soil Moisture Storage	0.0	0.0	0.0	0.0	0.0	-29.0	-48.0	-25.0	102.0	0.0	0.0	0.0	0.0
Soil Moisture Storage	200.0	200.0	200.0	200.0	200.0	171.0	123.0	98.0	200.0	200.0	200.0	200.0	200.0
Actual Evapotranspiration (AET)	0.0	0.0	0.0	29.5	75.1	108.3	118.8	107.2	75.4	36.8	7.7	0.0	558.8
Soil Moisture Deficit (mm)	0.0	0.0	0.0	0.0	0.0	-29.0	-48.0	-25.0	0.0	0.0	0.0	0.0	0.0
Surplus (P – AET)	64.3	54.5	60.9	40.6	11.5	-27.0	-38.0	-19.0	11.6	39.8	79.4	64.2	342.8

Table 6. MOEE (1995) Infiltration Factors

Description of the Area	Value of the Infiltration Factor
TOPOGRAPHY	
- Hilly topography	0.10
SOIL	
- Sand to silty sand	0.4 (treated as “Open sandy loam”)
COVER	
- Agricultural	0.10
- Woodland	0.20

4.2.2 Post-Development Conditions

It is intended to develop the site with 15 residences and a roadway (refer to Site Plan by The Biglieri Group Ltd., (**Appendix A**). The portion of the site that is not reserved as a Conservation Easement is approximately 2.61 ha. Details of the planned residences are not available. As a temporary measure, it is assumed that the homes will have a first floor footprint of 1,800 ft^2 (167.3 m^2) each and that the standard driveway will be, on average, 4.5 x 10 m. The roadway is approximately 188 m long with an assumed paved

width of 25 ft/7.6 m. This results in 4,615 m^2 of impervious surfaces (rooftops and paved areas). It is also assumed that the remaining portion of the developed area will be divided evenly between new lawns/open space and existing woodlands. An infiltration factor of 0.15 (average of agricultural and woodland) is applied to this 2.15 ha.

It is estimated that, post-development, 12,705.7 m^3/year (215 mm/year) of precipitation will infiltrate (a decrease of 10.4%), and 9,534.6 m^3/year (161 mm/year) will run off (an increase of 56.8%). Through the use of Low Impact Development (LID) measures which are designed to re-infiltrate 1,475.7 m^3 of precipitation per year across the entire site area, the post-development infiltration will be maintained at pre-development conditions.

Based on the soil and groundwater conditions at the site, infiltration based LIDs such as infiltration trenches, galleries, or swales would be effective in promoting groundwater recharge to balance the pre-to-post development water balance. LIDs should be installed 1 m above the high water table and designed based a percolation rate of 20.7 mm/hr, which includes a 2.5x factor of safety from the percolation rates provided in **Table 3**.

Table 7. Pre- and Post Development Water Balances

Pre-Development

Land Use	Area (ha)	Impervious Factor	Impervious Area (ha)	Water Surplus on Impervious Area (m/yr)	Runoff from Impervious Area (m^3/yr)	Pervious Area (ha)	Water Surplus on Pervious Area (m/yr)	Runoff Coefficient	Runoff Volume from Pervious Area (m^3/yr)	Infiltration Coefficient	Infiltration Volume from Pervious Area (m^3/yr)	Total Runoff Volume (m^3/yr)	Total Infiltration Volume (m^3/yr)
Woodland	5.91	0.00	0.00	0.772	0	5.91	0.3428	0.30	6,077.8	0.70	14,181.4	6,077.8	14,181.4
Development – Building/Paved Areas	0.00	1.00	0.00	0.772	0	0.00	0.3428	1.00	0	0.00	0	0	0
Total	**5.91**		**0.00**		**0**	**5.91**			**6,077.8**		**14,181.4**	**6,077.8**	**14,181.4**

Post-Development (No Mitigation)

Land Use	Area (ha)	Impervious Factor	Impervious Area (ha)	Water Surplus on Impervious Area (m/yr)	Runoff from Impervious Area (m^3/yr)	Pervious Area (ha)	Water Surplus on Pervious Area (m/yr)	Runoff Coefficient	Runoff Volume from Pervious Area (m^3/yr)	Infiltration Coefficient	Infiltration Volume from Pervious Area (m^3/yr)	Total Runoff Volume (m^3/yr)	Total Infiltration Volume (m^3/yr)
Woodland (Conservation Reserve)	3.2985	0.00	0.00	0.772	0.00	3.2985	0.3428	0.3	3,392.2	0.70	7,915.1	3,392.2	7,915.1
Lawn/Woodland	2.15	0.00	0.00	0.772	0.00	2.15	0.3428	0.35	2,579.6	0.65	4,790.6	2,579.6	4,790.6
Development – Building/Paved Areas	0.4615	1.00	0.4615	0.772	3,562.8	0.00	0.3428	1.00	0	0.00	0	3,562.8	0
Total	**5.91**		**0.4615**		**3,562.8**	**5.4485**			**5,971.8**		**12,705.7**	**9,534.6**	**12,705.7**

Post-Development (With Mitigation)

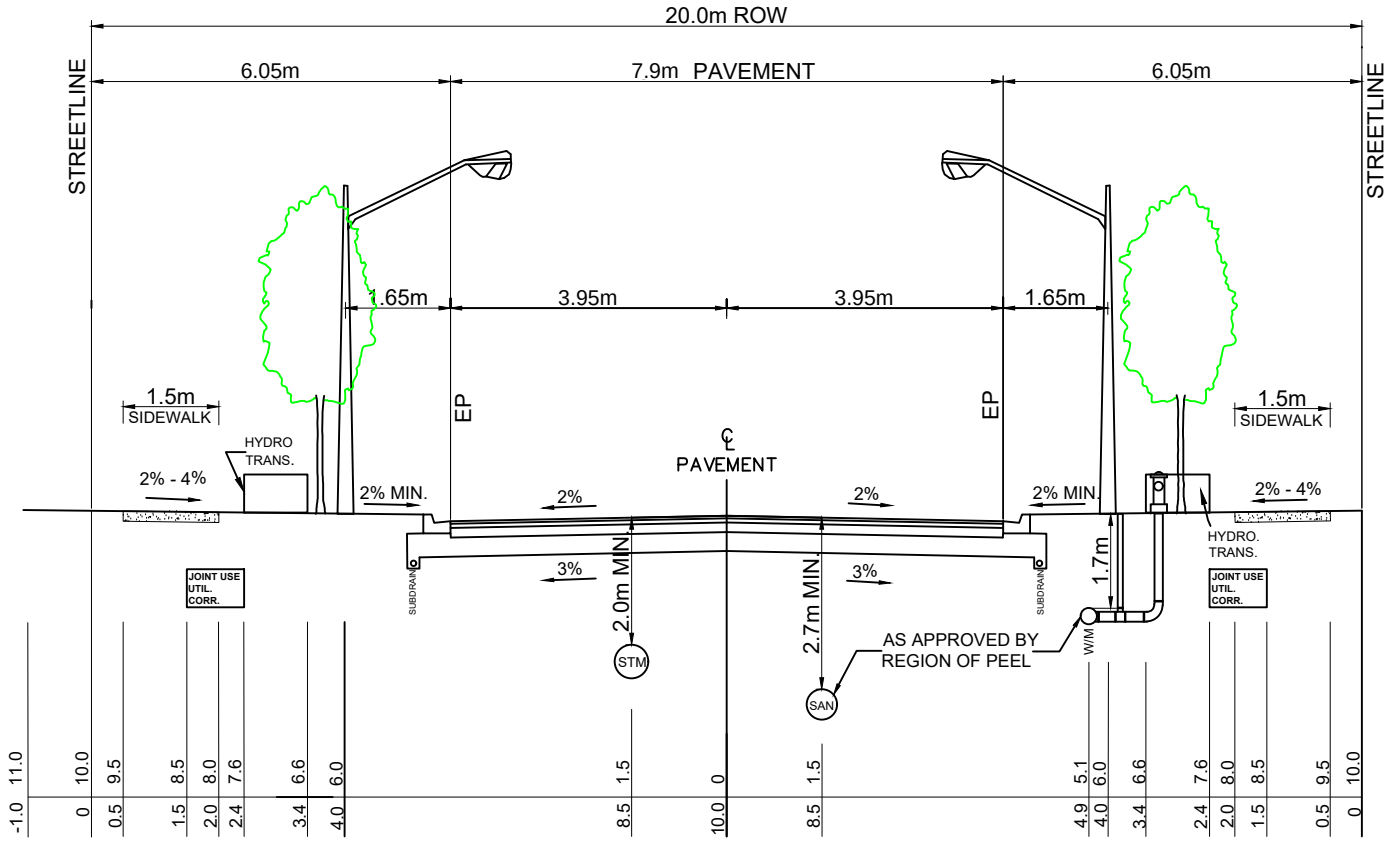
Land Use	Area (ha)	Impervious Factor	Impervious Area (ha)	Water Surplus on Impervious Area (m/yr)	Runoff from Impervious Area (m^3/yr)	Pervious Area (ha)	Water Surplus on Pervious Area (m/yr)	Runoff Coefficient	Runoff Volume from Pervious Area (m^3/yr)	Infiltration Coefficient	Infiltration Volume from Pervious Area (m^3/yr)	Total Runoff Volume (m^3/yr)	Total Infiltration Volume (m^3/yr)
Woodland (Conservation Reserve)	3.2985	0.00	0.00	0.772	0.00	3.2985	0.3428	0.3	3,392.2	0.70	7,915.1	3,392.2	7,915.1
Lawn/Woodland	2.15	0.00	0.00	0.772	0.00	2.15	0.3428	0.35	2,579.6	0.65	4,790.6	2,579.6	4,790.6

Land Use	Area (ha)	Impervious Factor	Impervious Area (ha)	Water Surplus on Impervious Area (m/yr)	Runoff from Impervious Area (m^3/yr)	Pervious Area (ha)	Water Surplus on Pervious Area (m/yr)	Runoff Coefficient	Runoff Volume from Pervious Area (m^3/yr)	Infiltration Coefficient	Infiltration Volume from Pervious Area (m^3/yr)	Total Runoff Volume (m^3/yr)	Total Infiltration Volume (m^3/yr)
Development – Building/Paved Areas	0.4615	1.00	0.4615	0.772	3,562.8	0.00	0.3428	1.00	0	0.00	0	3,562.8	0
Infiltration Based LID (assumed)												-1,475.7	+1,475.7
Total	**5.91**		**0.4615**		**3,562.8**	**5.4485**			**5,971.8**		**12,705.7**	**8,058.9**	**14,181.4**

No Mitigation	**Pre-to-Post Difference (%)**	**+56.8**	**-10.4%**
	Pre-to-Post Difference (m^3)	**+3,456.8**	**-1,475.7**
With Mitigation	**Pre-to-Post Difference (%)**	**+32.6%**	**0%**
	Pre-to-Post Difference (m^3)	**+1,981.1**	**0%**

APPENDIX “G”

Road Details



NOTES:

1. UTILITY CORRIDOR TO HAVE A MINIMUM COVER OF 0.9m.
2. WATERMAIN TO HAVE MINIMUM COVER OF 1.7m.
3. THE FOLLOWING IS A MINIMUM ROAD BASE AND WILL REQUIRE A SOILS REPORT VERIFICATION
   40mm HL3
   65mm HL8
   150mm GRANULAR "A"
   300mm GRANULAR 'B'
4. THE BOULEVARDS REQUIRE A MINIMUM OF 300mm OF TOPSOIL AND NURSERY SOD
5. TREES TO BE PLACED IN LOCATIONS PER APPROVED LANDSCAPE PLAN
6. STREETLIGHT FIXTURE PER APPROVED TOWN STANDARD.
7. ON A CRESCENT THE WATERMAIN SHALL BE PLACED ON THE OUTSIDE.
8. FULL LENGTH MINIMUM 100MM DIA.SUB-DRAINS C/W FILTERCLOTH SHALL BE INSTALLED TO TOWN OF CALEDON STANDARD NO. 218.
9. SUB-GRADE SHALL BE COMPACTED TO A MINIMUM 95% OF S.P.D. AT OPTIMUM MOISTURE CONTENT.
10. WHERE POSSIBLE MANHOLE LIDS TO BE LOCATED OUT OF TIRE LANE OF TRAFFIC.
11. ESTATE RESIDENTIAL 4.0 METERS LANE WIDTH.

TOWN OF CALEDON

20.0m LOCAL URBAN/RURAL
8.5m ROADWAY (7.9m PAVEMENT)

NO.	REVISION	APR'D	DATE
5	TEXT AND SLOPE REVISIONS		JULY 19
4	DIMENSION AND TEXT REVISION		JAN. 09
3	DIMENSION, LAYOUT REVISION		JULY 08
2	DIMENSION EDIT, STD No. 201 NOW 203		JUNE 08
1	OFFSETS		APRIL 08

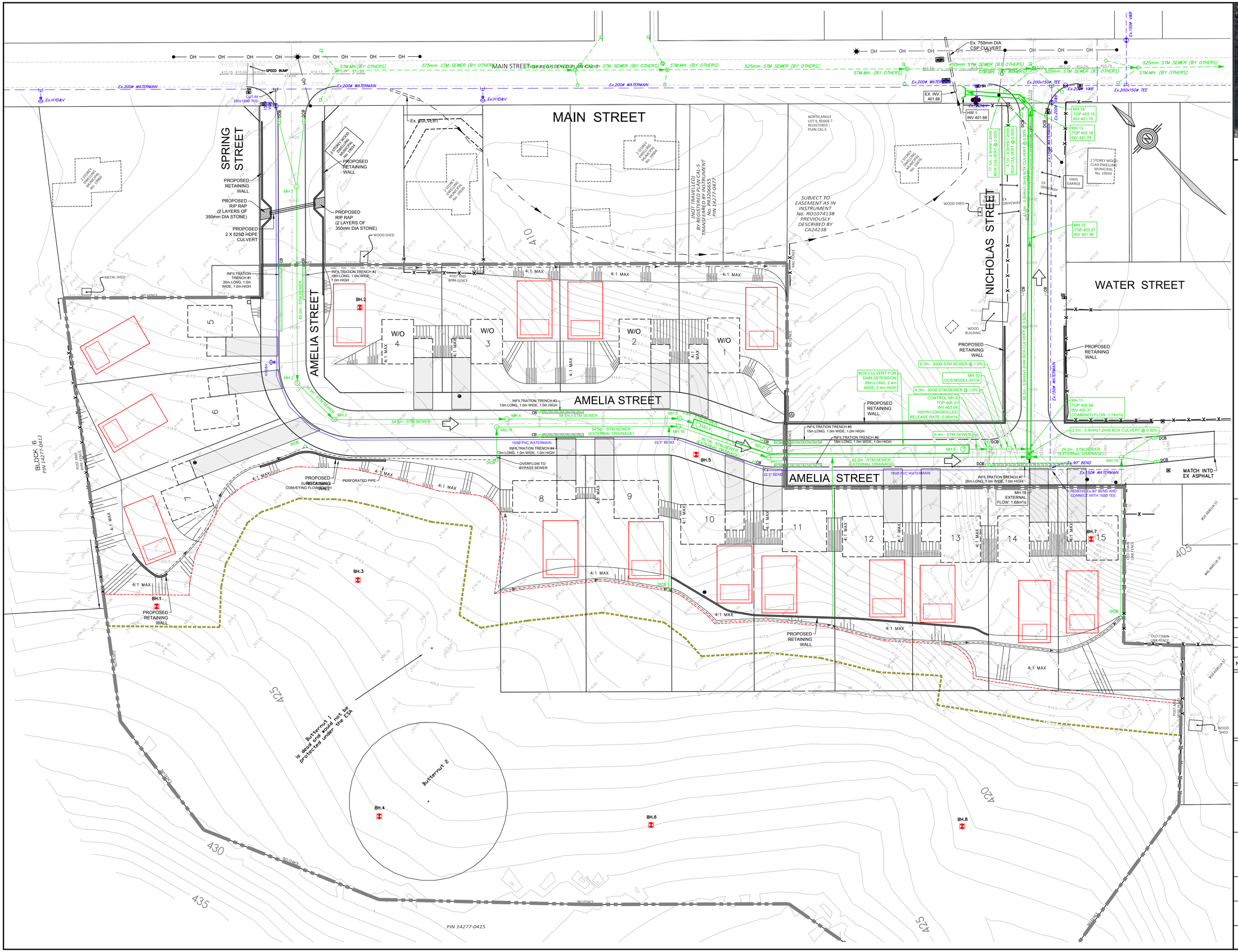
APR'D: C.C. DATE: 19/01/00
DRAWN: BJM SCALE: N.T.S.

STANDARD No. 203

MODIFIED PAVEMENT WIDTH FOR 21T-24001C

APPENDIX “H”

Functional Servicing, Grading & ESC Plan



KEY PLAN
N.T.S

LEGEND:

- PROPOSED STORM SEWER
- STORM MANHOLE
- CATCHBASIN
- VALVE AND BOX
- EX WATERMAIN
- PROPOSED WATERMAIN
- HYD&V
- LIMIT OF SUBDIVISION
- LIMIT OF WOODLAND
- 10m WOODLAND MVPZ
- BH.1 BOREHOLE LOCATION (SPCL, 2017)
- SEPTIC SYSTEM

NOTE:
THIS PLAN HAS BEEN PREPARED IN CONJUNCTION WITH THE FUNCTIONAL SERVICING REPORT TO DEMONSTRATE FEASIBILITY OF THE PROPOSED DEVELOPMENT IN CONJUNCTION WITH THE DRAFT PLAN OF SUBDIVISION APPLICATION. DETAILED DESIGNS WILL BE PREPARED AT THE DETAILED SUBDIVISION ENGINEERING DESIGN STAGE.

BENCHMARK NOTE:
ELEVATIONS ARE BASED ON GPS OBSERVATIONS TO PERMANENT REFERENCE STATIONS IN THE NAD83 (CSRS-2010) COORDINATE SYSTEM WITH HEIGHTS CONVERTED TO ORTHOMETRIC ELEVATIONS ON THE CGVD28 DATUM (1978 ADJUSTMENT) WITH GEOID MODEL HTv2.0, AS SUPPLIED BY NATURAL RESOURCES CANADA.

SURVEYOR: VAN HARTEN LAND SURVEYORS DATE: MARCH 13, 2026

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

NO.	DATE	REVISIONS	BY
2	APR.17, 2026	AS PER TOWN AND REGION COMMENTS	D.G.
1	FEB.14, 2024	UPDATED BOUNDARY SURVEY	D.G.

PRELIMINARY

VALDOR ENGINEERING INC.
Consulting Engineers - Project Managers
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AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
TOWN OF CALEDON
REGION OF PEEL
FILE NUMBER: 21T-24001C

FUNCTIONAL SERVICING PLAN

SCALE:	DATE OF DWG.	PROJECT NO.
1:500	APR. 11, 2022	17121
DRAWN BY: D.N.Z.	DRAWING NO.	
CHKD BY D.A.G.	FSP-1	

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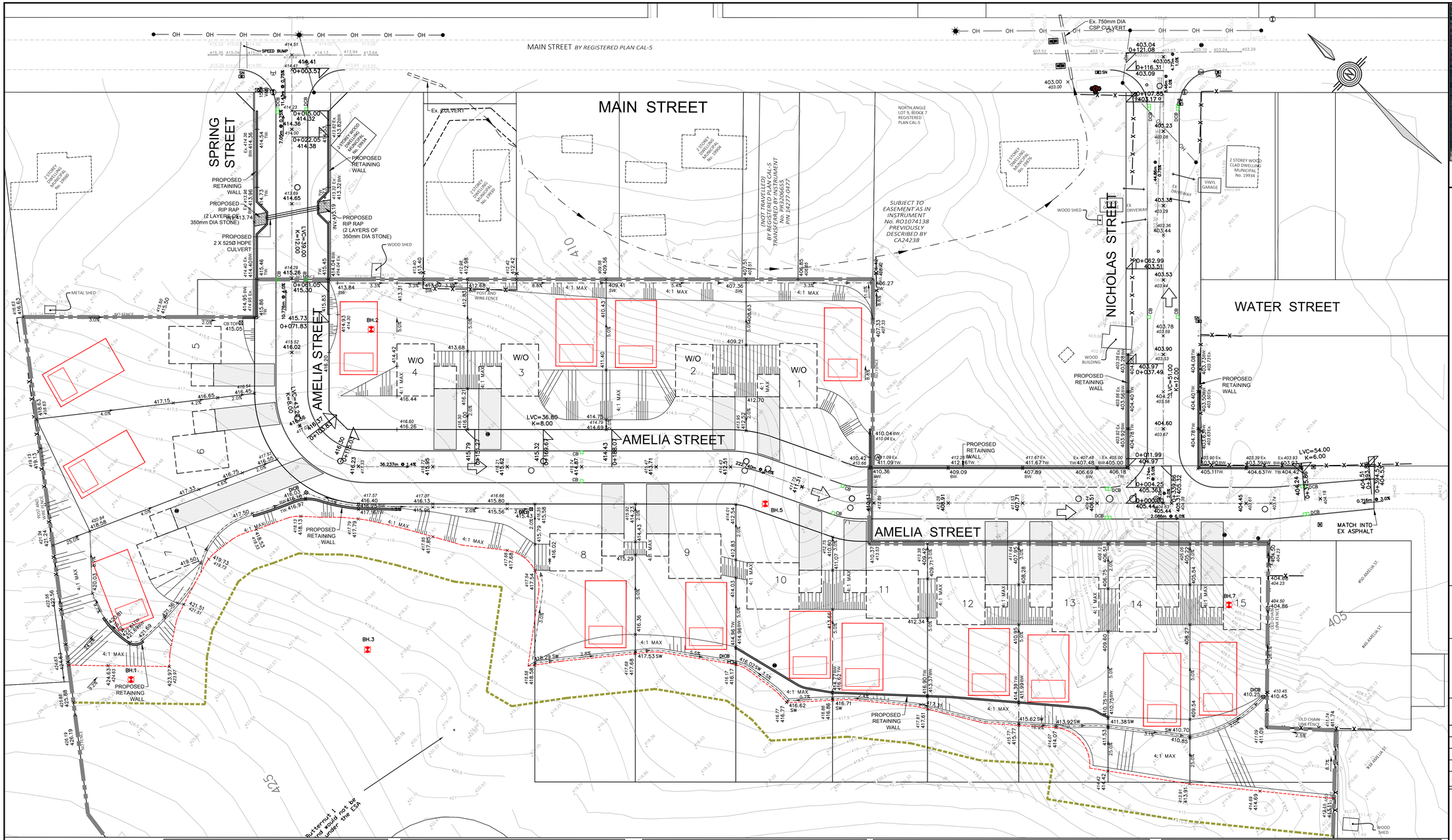

KEY PLAN
N.T.S

LEGEND:

- EXISTING ELEVATION
- EXISTING CONTOUR
- DRAINAGE FLOW DIRECTION AND SLOPE
- × 126.50 PROPOSED ELEVATION
- FFF FINISHED FIRST FLOOR ELEVATION
- BSMT BASEMENT ELEVATION
- STORM MANHOLE
- CATCHBASIN
- VALVE AND BOX
- HYD&V
- LIMIT OF SUBDIVISION
- LIMIT OF WOODLAND
- 10m WOODLAND MVPZ
- BH.1 BOREHOLE LOCATION (SPCL, 2017)
- SEPTIC SYSTEM
- OVERLAND FLOW ARROW

NOTE:
THIS PLAN HAS BEEN PREPARED IN CONJUNCTION WITH THE FUNCTIONAL SERVICING REPORT TO DEMONSTRATE FEASIBILITY OF THE PROPOSED DEVELOPMENT IN CONJUNCTION WITH THE DRAFT PLAN OF SUBDIVISION APPLICATION. DETAILED DESIGNS WILL BE PREPARED AT THE DETAILED SUBDIVISION ENGINEERING DESIGN STAGE.

BENCHMARK NOTE:
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SURVEYOR: VAN HARTEN LAND SURVEYORS DATE: MARCH 13, 2026

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

NO.	DATE	REVISIONS	BY
2	APR.17, 2026	AS PER TOWN AND REGION COMMENTS	D.G.
1	FEB.14, 2024	UPDATED BOUNDARY SURVEY	D.G.

PRELIMINARY

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AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
TOWN OF CALEDON
REGION OF PEEL
FILE NUMBER: 21T-24001C

FUNCTIONAL GRADING PLAN

SCALE:	DATE OF DWG.	PROJECT NO.
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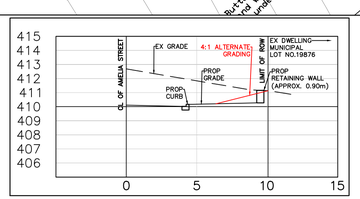
CHKD BY D.A.G.	FGP-1	



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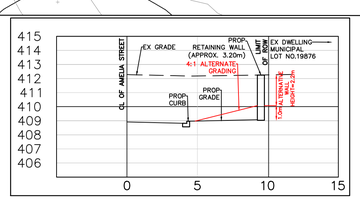

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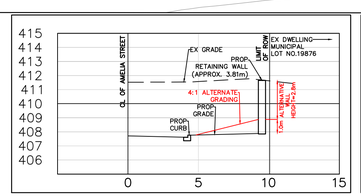

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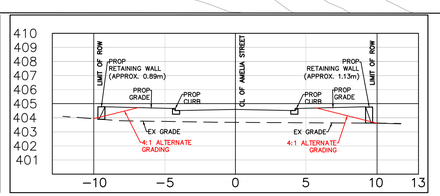

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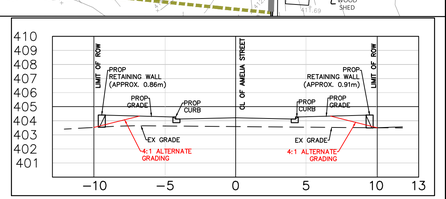

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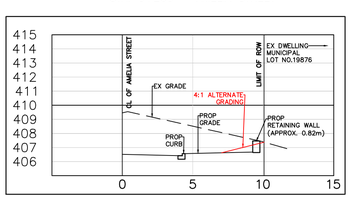

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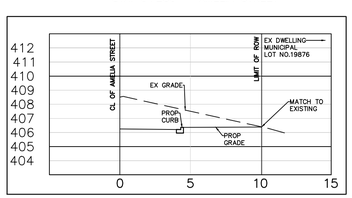

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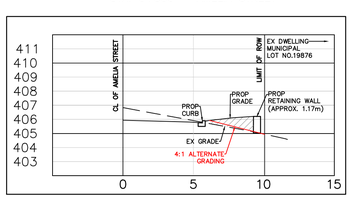

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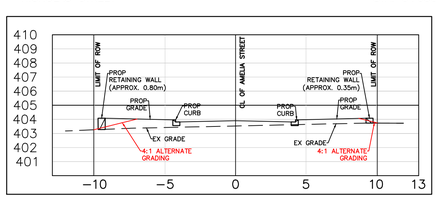

STA. 0+040 – NICHOLAS STREET

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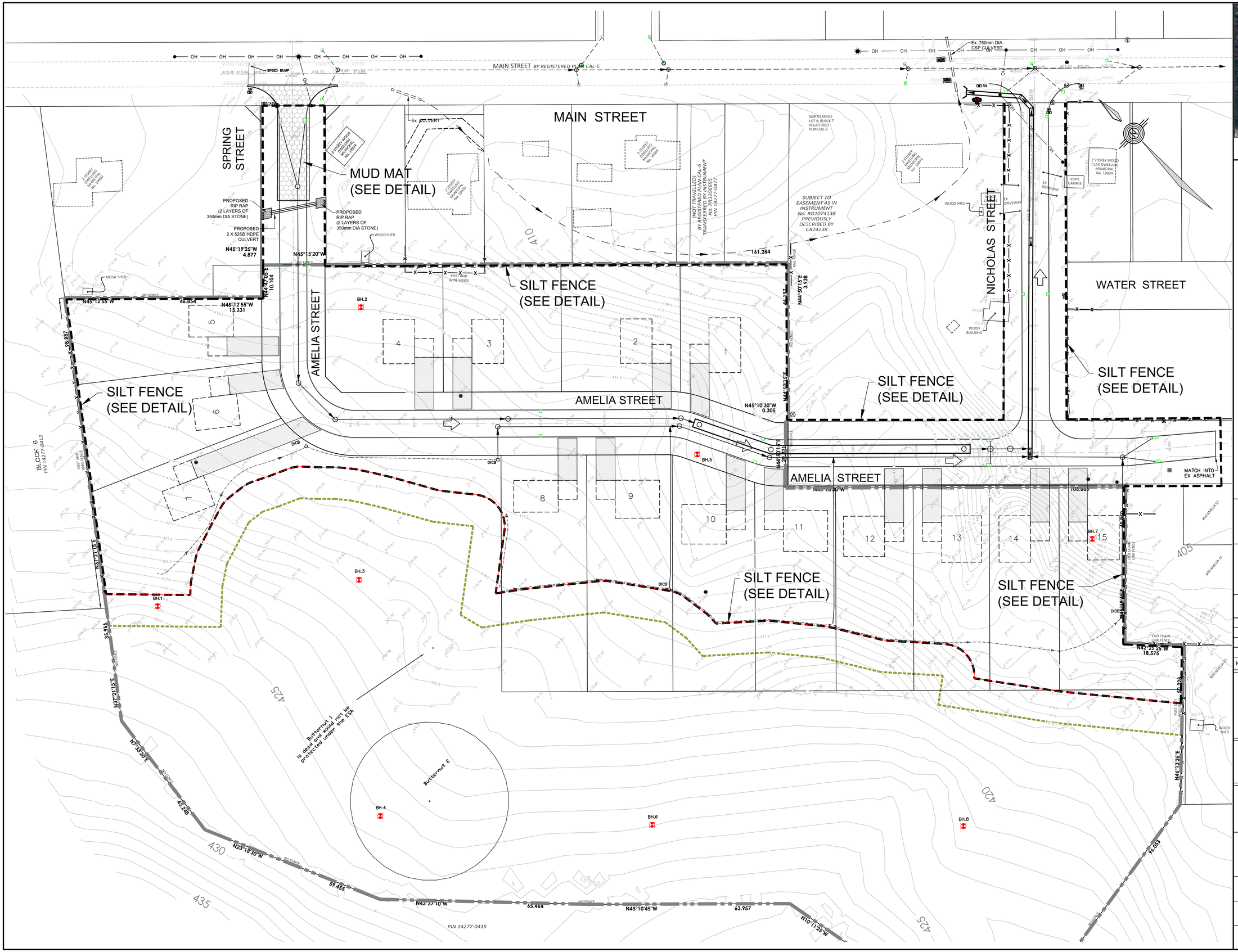

KEY PLAN
N.T.S

LEGEND:

- LIMIT OF SUBDIVISION
- 301.08 EXISTING ELEVATION
- 188.5 EXISTING CONTOUR
- STORM MANHOLE
- CATCHBASIN
- BH7 BOREHOLE LOCATION
- TP-4 TEST PIT LOCATION
- SILT FENCE
- ROCK CHECK DAM
- LIMIT OF WOODLAND
- 10m WOODLAND MVPZ

NOTE:
THIS PLAN HAS BEEN PREPARED IN CONJUNCTION WITH THE FUNCTIONAL SERVICING REPORT TO DEMONSTRATE FEASIBILITY OF THE PROPOSED DEVELOPMENT IN CONJUNCTION WITH THE DRAFT PLAN OF SUBDIVISION APPLICATION. DETAILED DESIGNS WILL BE PREPARED AT THE DETAILED SUBDIVISION ENGINEERING DESIGN STAGE.

BENCHMARK NOTE:
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SURVEYOR: VAN HARTEN LAND SURVEYORS DATE: MARCH 13, 2026

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

NO.	DATE	REVISIONS	BY
2	APR.17, 2026	AS PER TOWN AND REGION COMMENTS	D.G.
1	FEB.14, 2024	UPDATED BOUNDARY SURVEY	D.G.

PRELIMINARY

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AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
TOWN OF CALEDON
REGION OF PEEL
FILE NUMBER: 21T-24001C

FUNCTIONAL EROSION AND SEDIMENT CONTROL PLAN

SCALE:	DATE OF DWG.	PROJECT NO.
1:500	APR. 11, 2022	17121
DRAWN BY: D.N.Z.	DRAWING NO.	
CHKD BY D.A.G.	ESC-1	

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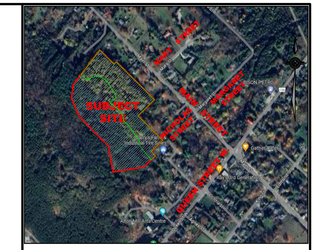
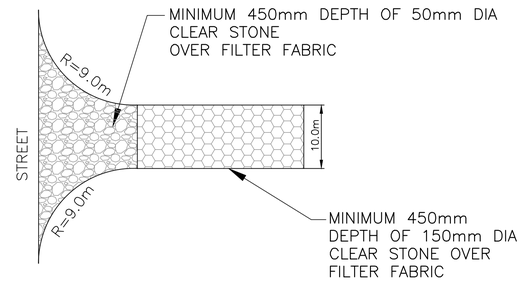


KEY PLAN
N.T.S



NOTE:

1. MUD MAT TO BE INSTALLED AT THE TEMPORARY CONSTRUCTION ACCESS LOCATION.
2. GRANULAR MATERIAL TO BE PERIODICALLY REPLACED AS IT BECOMES CONTAMINATED

MUD MAT DETAIL

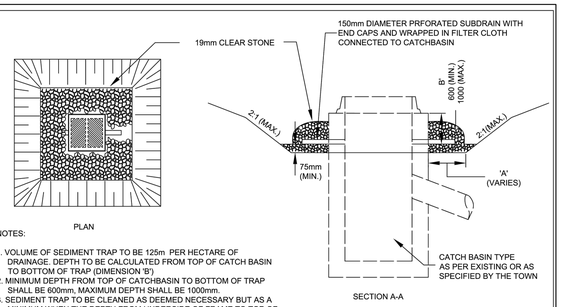
N.T.S.

EROSION & SEDIMENT CONTROL NOTES

1. THE EROSION AND SEDIMENT CONTROL STRATEGIES OUTLINED ON THE PLANS ARE NOT STATIC AND MAY NEED TO BE UPGRADED/AMENDED AS SITE CONDITIONS CHANGE TO PREVENT SEDIMENT RELEASES TO THE NATURAL ENVIRONMENT.
2. ALL SEDIMENT AND EROSION CONTROL MEASURES SHALL BE INSTALLED AND IN PROPER WORKING ORDER PRIOR TO THE COMMENCEMENT OF EARTHWORKS, THE EXACT LOCATION TO BE DETERMINED IN THE FIELD.
3. ALL EROSION AND SEDIMENT CONTROL MEASURES SHALL BE ROUTINELY INSPECTED AND MAINTAINED IN PROPER WORKING ORDER AND CLEANED PERIODICALLY.
4. ALL CONSTRUCTION VEHICLES SHALL EXIT THE SITE VIA THE TEMPORARY CONSTRUCTION ACCESS.
5. HYRDOSEEDING TO BE APPLIED BY AN APPROVED CONTRACTOR. SEED MIX TO BE OBTAINED FROM LOCAL SEED SOURCE.
6. MONITORING OF THE GERMINATION OF THE HYDROSEED IS REQUIRED TO IDENTIFY ANY BARE AREAS THAT MAY RESULT. CONTRACTOR TO RE-SOW ALL BARE AREAS TO ENSURE SURFACES ARE ADEQUATELY STABILIZED.
7. ALL AREAS DISTURBED DUE TO CONSTRUCTION SHALL BE RESTORED TO THEIR ORIGINAL CONDITION OR BETTER TO THE SATISFACTION OF THE MUNICIPALITY AND NVCA. DISTURBED AREAS TO BE STABILIZED AS DIRECTED BY THE CONSULTANT ENGINEER AND TO THE SATISFACTION OF THE MUNICIPALITY IF LEFT UN-WORKED WITHIN 30 DAYS.
8. CONTRACTOR TO CLEAN AND MAINTAIN THE EXISTING ROADS ON A REGULAR BASIS FREE FROM DIRT AND MUD.

LEGEND:

+ 301.08	EXISTING ELEVATION
188.5	EXISTING CONTOUR
	STORM MANHOLE
	CATCHBASIN
BH7	BOREHOLE LOCATION
TP-4	TEST PIT LOCATION
	STRUCTURAL ENVELOPE
	ENVIRONMENTAL PROTECTION
	REFORESTED / PROTECTION AREA
	SILT FENCE
	CONSTRUCTION FENCE
	ROCK CHECK DAM



NOTES:

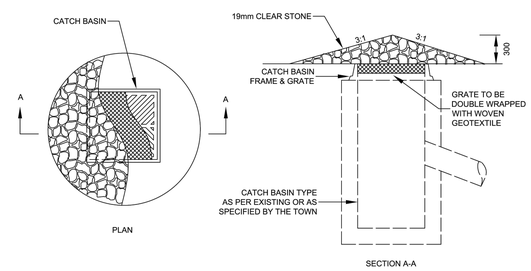
1. VOLUME OF SEDIMENT TRAP TO BE 125m PER HECTARE OF DRAINAGE. DEPTH TO BE CALCULATED FROM TOP OF CATCH BASIN TO BOTTOM OF TRAP (DIMENSION 'B')
2. MINIMUM DEPTH FROM TOP OF CATCHBASIN TO BOTTOM OF TRAP SHALL BE 600mm. MAXIMUM DEPTH SHALL BE 1000mm.
3. SEDIMENT TRAP TO BE CLEANED AS DEEMED NECESSARY BUT AS A MINIMUM WHEN THE DEPTH FROM UNDERSIDE OF FRAME TO TOP OF SILT IS REDUCED TO 300mm.
4. UPON REMOVAL OF TRAP, CATCHBASIN CONNECTION HOLES TO BE PARGED WITH CONCRETE.

TOWN OF CALEDON

CATCH BASIN SEDIMENT TRAP

NO.	REVISION	APR'D	DATE
1	STANDARD No. 320.01 NOW 302		JUNE 08

APR'D: C.C. DATE: APRIL 00
DRAWN: BJM SCALE: NTS
STANDARD No. 302



NOTES:

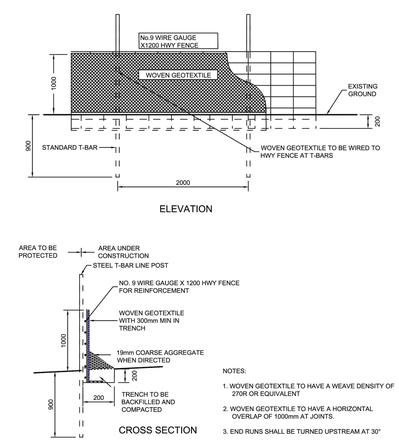
1. CATCH BASIN LIFT HOLES TO BE PARGED WITH CONCRETE.
2. WOVEN GEOTEXTILE TO HAVE A MINMUM EQUIVALENT OPENING SIZE OF 0.15mm AND A MAXIMUM OPENING SIZE OF 0.25mm.

TOWN OF CALEDON

CATCH BASIN SEDIMENT BARRIER

NO.	REVISION	APR'D	DATE
1	STANDARD No. 320.02 NOW 303		JUNE 08

APR'D: C.C. DATE: APRIL 00
DRAWN: BJM SCALE: NTS
STANDARD No. 303



NOTES:

1. WOVEN GEOTEXTILE TO HAVE A WEAVE DENSITY OF 270R OR EQUIVALENT
2. WOVEN GEOTEXTILE TO HAVE A HORIZONTAL OVERLAP OF 1000mm AT JOINTS.
3. END RUNS SHALL BE TURNED UPSTREAM AT 30°

NO.	REVISION	APR'D	DATE
2	NAME CHANGED, STANDARD No. 325.01 NOW 304		JUNE 08
1	SUPPORT FENCING CHANGED TO HWY FENCE		Oct 02

TOWN OF CALEDON

SEDIMENT CONTROL FENCE

APR'D: C.C. DATE: APRIL 2000
DRAWN: BJM SCALE: N.T.S.
STANDARD No. 304

NOTE:
THIS PLAN HAS BEEN PREPARED IN CONJUNCTION WITH THE FUNCTIONAL SERVICING REPORT TO DEMONSTRATE FEASIBILITY OF THE PROPOSED DEVELOPMENT IN CONJUNCTION WITH THE DRAFT PLAN OF SUBDIVISION APPLICATION. DETAILED DESIGNS WILL BE PREPARED AT THE DETAILED SUBDIVISION ENGINEERING DESIGN STAGE.

BENCHMARK NOTE:
ELEVATIONS ARE BASED ON GPS OBSERVATIONS TO PERMANENT REFERENCE STATIONS IN THE NAD83 (CSRS-2010) COORDINATE SYSTEM WITH HEIGHTS CONVERTED TO ORTHOMETRIC ELEVATIONS ON THE CGVD28 DATUM (1978 ADJUSTMENT) WITH GEOID MODEL HTv2.0, AS SUPPLIED BY NATURAL RESOURCES CANADA.

SURVEYOR: VAN HARTEN LAND SURVEYORS DATE: MARCH 13, 2026

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

NO.	DATE	REVISIONS	BY
2	APR.17, 2026	AS PER TOWN AND REGION COMMENTS	D.G.
1	FEB.14, 2024	UPDATED BOUNDARY SURVEY	D.G.

PRELIMINARY

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www.valdor-engineering.com

AMELIA STREET SUBDIVISION
COMMUNITY OF ALTON
TOWN OF CALEDON
REGION OF PEEL
FILE NUMBER: 21T-24001C

EROSION AND SEDIMENT CONTROL NOTES & DETAILS

SCALE:	DATE OF DWG.	PROJECT NO.
1:500	APR. 11, 2022	17121
DRAWN BY: D.N.Z.	DRAWING NO.	
CHKD BY D.A.G.	ESC-2	

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