

**SERVICING & STORMWATER
MANAGEMENT REPORT – PHASE 1**

0 AIRPORT ROAD

**TOWN OF CALEDON
REGION OF PEEL**

**PREPARED FOR:
BROCCOLINI AIRPORT ROAD LP**

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MAY 2026

CFCA FILE NO. 2278-7228

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Revision Number	Date	Comments
Rev.0	October 30, 2025	Issued for First Submission (ZBA & SPA)
Rev.1	May 5, 2026	Issued for Second Submission (SPA)

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1.0 Introduction

C.F. Crozier & Associated Inc. (Crozier) was retained by Broccolini Airport Road LP to prepare a Servicing and Stormwater Management Report in support of Site Plan Application (SPA) for Phase 1 of the property located at 0 Airport Road (the Site) in the Town of Caledon (Town). This report demonstrates how the proposed development's servicing and stormwater management will conform with the Tullamore North West Secondary Plan design prepared by Crozier (March 2026) as well as the requirements of the Town and Region of Peel (Region).

1.1 Site Description

The Site is bound by Airport Road to the east and agricultural lands to the north, west and south. The property consists of a total area of 24.25 ha, of which 5.24 ha is proposed to be redeveloped as part of Phase 1. The portion of the Site that is proposed to be redeveloped currently consists of agricultural land. There is an environmental zone within the west portion of the entire Site. Salt Creek flows through the environmental zone within the Site.

1.2 Proposed Development

As shown in the Site Plan prepared by Powers Brown Architecture, the developable area is proposed to be phased, Phase 1 and future Phase 2. Each phase will have its own servicing and stormwater management. Only Phase 1, which is within the north portion of the property, is proposed to be developed at this time. The proposed development includes one (1) one-storey industrial building with a total area of 23,117 m² with associated parking and two (2) vehicular access points from Airport Road.

1.3 Referenced Documents

The following reports and design standards were referenced during the preparation of this report:

- Master Environmental Servicing Report, Tullamore North Secondary Plan (GEI, March 2026)
- Functional Servicing and Stormwater Management Report for Tullamore North West Secondary Plan (Crozier, March 2026)
- Hydrogeological Investigation Report (Crozier, May 2026)
- Environmental Impact Study, Broccolini Airport Road
- Region of Peel Public Works Design, Specifications & Procedures Manual (Region of Peel, 2010)
- Development Standards Manual (Town of Caledon, 2019)
- Stormwater Management Criteria (TRCA, August 2012)
- Green Development Standard Guidebook (Town of Caledon, June 2024)

2.0 Sanitary Servicing

2.1 Sanitary Design Criteria

The proposed sanitary sewer for the Site has been designed based on the Region of Peel – Linear Wastewater standards. The sanitary design criteria are as follows:

- Population Density – 70 persons/ha (industrial)
- Infiltration – 0.26 L/s/ha
- Non-Residential Sanitary Sewage Demand – 270 L/capita/day
- Harmon Peaking Factor

2.2 Existing Municipal Sanitary System

Review of the Region of Peel As-Constructed Drawings 21983-D, 21984-D, 21985-D and 21986-D shows an existing 600 mm diameter gravity sanitary sewer sloping between 0.25 – 0.35% on Airport Road across the entire frontage of the site. The existing sewer is located on the west side of the Airport Road ROW and currently services Caledon East.

2.3 Proposed Sanitary Sewer System

Sanitary servicing for Building A will be provided by a 200 mm diameter sanitary sewer connection to EX.MH31 on Airport Road with MH101A serving as the property line maintenance hole. Sanitary connections to Building A will be 200 mm in diameter and will be plugged at the face of the building.

A sanitary sewer design sheet has been completed based on the Sanitary Drainage Area Plan (Figure 1) and is included in Appendix A. The proposed sanitary services are shown on Drawings C102 and C102A.

The sanitary sewage design flows were calculated using the Region's standards and the Site Plan prepared by Powers Brown Architecture. Table 1 outlines the sanitary demands for the proposed development. Detailed calculations are included in Appendix A.

Table 1: Proposed Sanitary Demand Flows

Building	Equivalent Population	Peak Flow (L/s)	Infiltration Flow (L/s)	Total Flow (L/s)	Outlet
A	367	4.59	1.36	5.95	Airport Road

Sanitary flows from Building A will discharge to Airport Road and result in an increase of 5.95 L/s.

3.0 Water Servicing

3.1 Sanitary Design Criteria

Water servicing for the Site has been designed in accordance with the Region of Peel standards and specifications to ensure that adequate pressures and flows are achieved. Watermain design flows are based on the following criteria:

- Average Day Demand – 250 L/employee/day
- Population Density – 70 persons/ha (industrial)
- Maximum Day Factor – 1.4
- Peak Hour Factor – 3.0
- Design Flow - greater of maximum daily demand plus fire flow or peak hourly demand

3.2 Existing Municipal Watermain System

Review of the Region of Peel As-Constructed Drawings 21983-D, 21984-D, 21985-D and 21986-D shows an existing 300 mm diameter watermain on Airport Road. The Site is in Pressure Zone 6 within the South Region of Peel Water Supply System.

3.3 Proposed Watermain System

Water servicing for Building A will be provided by a 200 mm diameter water service connection to the existing 300 mm diameter watermain within Airport Road. At the property line, the 200 mm water service will split into a 200 mm fire and a 150 mm domestic water service. The 200mm fire service is set to be complete with a detector check valve in chamber at the property line. The fire service will enter Building A where the mechanical room is located, then it will exit the building and wrap around the parcel to service hydrants and individual fire connections to the building. Note, the fire service design will be detailed through coordination with the mechanical engineer and sprinkler consultant as part of upcoming submissions. The domestic service will enter the building, where a water meter and backflow preventer will be provided per mechanical design and specifications. The proposed water servicing is shown on Drawings C102 and C102A.

The domestic water demand has been calculated based on the Site Plan by Powers Brown Architecture. Table 2 below summarizes the domestic water demand. The detailed calculations are included in Appendix B.

Table 2: Proposed Domestic Demand

Building	Equivalent Population	Average Daily Demand (L/s)	Maximum Daily Demand (L/s)	Peak hourly demand (L/s)
A	367	1.06	1.49	3.19

The fire flow requirements for Building A were calculated based on the Fire Underwriters Survey (FUS) requirements, as shown in Table 3.

Table 3: Fire Flow Demands – Fire Underwriter's Survey

Building	Required Fire Flow (L/s)
A	150.0

Note, a fire suppression consultant may be retained to calculate the building's specific fire flow demands based on the individual fire suppression tactics that are proposed.

A hydrant flow test was completed on October 28, 2025 for the 300 mm diameter watermain on Airport Road. It was determined that at 20 psi, a flow of 191 L/s (3024 USGPM) is available. As such, the municipal watermain has sufficient pressure and flow to service the proposed development. Refer to the hydrant flow test in Appendix B for reference.

4.0 Stormwater Management

The stormwater management strategy for the Site has been designed in accordance with the Secondary Plan for Tullamore North West (Crozier, March 2026). The stormwater management criteria are summarized below:

- **Quantity Control:** Control of the 2-year to 100-year storm events according to the unit flow relationships established by TRCA for Sub-Basin 63. Refer to Section 4.2 for more details on the design flow targets determined with reference to the Secondary Plan report (March 2026).
- **Regional Control:** The Secondary Plan report (March 2026) indicates that Regional storm event control is required. Refer to Section 4.2 for more details on the design flow target.
- **Quality Control:** Enhanced level of water quality protection (80% total suspended solids (TSS) removal). Per the Town's Green Development Standards, treatment is to be provided through a hierarchical approach to the greatest extent possible using retention, filtration and conventional stormwater management.
- **Site Water Balance:** Water balance analysis is required using the average and more frequent precipitation events that comprise of the bulk volume of annual precipitation to ensure maintenance of pre-development water balance following development. Per the analysis completed in the Hydrogeological Investigation Report (May 2026) this results in an infiltration target of 6,702 m³/year, which equates to the 4.8 mm event over the impervious areas on Site resulting in a volume of 195 m³.
- **Feature-Base Water Balance:** Candidate significant wetland has been identified within the lands owned by Broccolini by GEI. The wetland is located along the length of Salt Creek and is 2 m wide at the bottom of the valley. A feature-based water balance assessment was undertaken to estimate the potential impact of the proposed development on the wetland. Refer to Section 4.8 for more details.
- **Erosion Control:** At a minimum, TRCA requires on-site retention of the first 5 mm of runoff generated from the total impervious area. For properties with stormwater management facilities, extended detention of the 25 mm design storm event for a period of 48 hours is required.

4.1 Existing Drainage Conditions

According to aerial imagery, the portion of property proposed for redevelopment currently consists of agricultural land. A review of existing drainage patterns on Site and the surrounding area was completed based on the topographic survey completed by R-PE Surveying Ltd. (July 29, 2025) and the TRCA GTA_2023 data retrieved from the Government of Canada High Resolution Digital Elevation Model (HRDEM) – CanElevation Series on April 11, 2025.

Existing drainage is split within the Site, where approximately 81% drains towards Salt Creek and the other 19% drains towards Airport Road. There is also a 1.93 ha external area north of the Site that drains through the northwest corner for the property. Refer to Figure 3 for the pre-development drainage catchments and overland flow directions.

4.2 Proposed Drainage Conditions

The proposed grading on Site is designed to match the drainage patterns outlined in the Secondary Plan Report (March 2026). As part of the overall stormwater design described in the Secondary Plan Report, drainage from the Site must be directed towards Salt Creek. The grading design ensures that drainage is contained within the Site.

Stormwater for the proposed rooftop will be controlled to a release rate of 42 L/s/ha prior to discharging to the internal storm sewer system. Drainage from the parking, paved surfaces and landscaping surrounding the buildings will be captured through catchbasins. Some minor landscaped areas along the northwest border of the Site will drain uncontrolled towards Salt Creek. The internal storm sewer system will direct runoff towards the proposed stormwater management (SWM) pond at rear of the property (Catchment 203), which will be used to control all storm events. The outlet for the SWM pond is to Salt Creek. A storm sewer design sheet based on the Catchbasin Drainage Area Plan (Figure 2) was used for internal pipe sizing and is included in Appendix C. The storm sewers within the Site have been sized to convey the 5-year storm event. Runoff from storm events that exceed the 5-year storm event will be directed towards the pond overland through the proposed inlet spillway located in the northeast corner of the SWM pond.

The post-development catchments are summarized in Table 4. Refer to Figure 4 for the post-development drainage area plan.

Table 4: Post-Development Catchments

Catchment ID	Area (ha)	Runoff Coefficient	TIMP	XIMP	Curve Number (CN)	Time to Peak (hr)
201	2.31	0.90	0.99	0.99	81	-
202	2.20	0.76	0.79	0.79	81	-
203	0.65	0.25	-	-	50	0.36
UC1	0.08	0.25	-	-	81	0.30
Total	5.24	0.75	-	-	-	-
EXT1	1.93	0.25	-	-	86	0.39

4.2.1 External Drainage

As shown in Figure 4, there is external drainage entering the Site at the northwest corner of the property. The external catchment (EXT1) is 1.93 ha and currently consists of agricultural land. A swale is proposed along the north boundary of the Site to convey external runoff towards Salt Creek, mimicking existing conditions. The swale is sized as a triangular channel, with the low point in line

with the property line. The swale will convey the Regional storm event flow ($0.264 \text{ m}^3/\text{s}$) with 0.3 m of freeboard. Refer to Appendix C for the FlowMaster calculations.

4.2.2 Headwater Drainage Feature H4S2

There is an existing headwater drainage feature (HDF) located near the northwest corner of the Site. Following development, the Site outlet will be directly to Salt Creek, downstream of HDF H4S2 however, the external drainage area (Catchment EXT1) will continue to contribute to this feature. It is noted that given grade restrictions, it is not feasible for the Site outlet to direct flows to the HDF.

4.3 Stormwater Quantity Control

Visual Otthymo (VO), a hydrologic stormwater modeling software, was used to determine peak runoff within the Site. In accordance with Table 3-1 of the *Stormwater Management Criteria* (TRCA, 2012) calculation of return period peak flows within the Humber River Watershed shall be based on the 6-hour and 12- hour AES storm distributions.

Table 4 summarizes the Time of Peak (T_p) values and SCS Curve Numbers (CN), which are based on soil, land use and topographic conditions.

The TIMP and XIMP values for each STANDHYD were determined based on the proposed land cover for each catchment. For catchments consisting only of rooftop, TIMP was assigned as 0.99, representing completely impervious cover. The XIMP values are equal to the TIMP for all catchments. The TIMP and XIMP values are also summarized in Table 4.

An initial abstraction (IA) value of 1 mm has been applied for catchments consisting of rooftops only and an IA of 2 mm was applied for parking lots.

As indicated in the Secondary Plan Report (March 2026), the allowable release rate is dictated by the unit flow relationships for Sub-Basin 36 established by the TRCA for the design storm events (2-year to 100-year storm events). The Secondary Plan also indicates that Regional storm controls are required. Flows from Catchments 201, 202 and 203 are controlled to ensure they do not exceed the allowable release rates, minus the uncontrolled flows from Catchment UC1.

The allowable release rates for the design storm events were determined as part of the Secondary Plan using the pre-development area directed towards Salt Creek for the entire property. Given that only Phase 1 is being developed at this time, the allowable release rate was determined as a percentage of the rate indicated in the Secondary Plan report based on the percentage of the property that is being redeveloped at this time. Refer to Table 5 for the allowable release rates for the Site.

Table 5: Summary of Unit Flow Relationships and Allowable Release Rates for the Site

Storm Event	Unit Flow Equations (Sub-Basin 36)	Allowable Release Rate (m³/s)	
		Total Property¹	Phase 1 Only²
2-year	$Q = 9.506 - 0.719 * \ln(A)$	0.018	0.007
5-year	$Q = 14.652 - 1.136 * \ln(A)$	0.055	0.021
10-year	$Q = 17.957 - 1.373 * \ln(A)$	0.135	0.051
25-year	$Q = 22.639 - 1.741 * \ln(A)$	0.172	0.065
50-year	$Q = 26.566 - 2.082 * \ln(A)$	0.201	0.076
100-year	$Q = 29.912 - 2.316 * \ln(A)$	0.226	0.086

1. Total allowable release rate for the property per the Secondary Plan
 2. Allowable release rate for Phase 1 only, determined to be 38% of the total allowable release rate based on percentage of the total area being redeveloped for this phase
- Additional notes: Q = unit flow (L/s/ha), A = area (ha)
Source: Stormwater Management Criteria prepared by TRCA (August 2012)

The allowable release rate for the Regional storm was also determined as part of the Secondary Plan through detailed modeling and review of downstream impacts. Similarly to the design storm target, the allowable release rate for Phase 1 has determined using the percent area that is being developed at this time (38%) and the rate indicated in the Secondary Plan report (0.861 m³/s). This results in a Regional storm allowable release rate of 0.327 m³/s for Phase 1.

Rooftop control is provided at a rate of 42 L/s/ha. This results in a total release rate of 0.097 m³/s for Catchment 201, requiring rooftop storage of 909 m³. Flows from the rooftop are also directed to the proposed SWM pond for further control. It is noted that the rooftop controls are not considered during the Regional storm modeling.

An outlet control manhole (CONTROL MH100) with three (3) orifice controls is proposed to control the post-development peak stormwater flows from Catchments 201, 202 and 203. The first orifice is 50 mm diameter at an elevation of 246.60 and controls the 2-year event. The second orifice is 235 mm diameter at an elevation of 247.80. The combination of the two orifices controls the 5-year to 100-year storm events. The third orifice is 250 mm diameter at an elevation of 248.32; the combination of all three orifices controls the Regional storm event. Refer to Drawing C106A for the control manhole details. Refer to Table 6 for a summary of the results and Appendix C for the VO model inputs.

Table 6: Summary of Peak Flow Rates

Storm Event	6-hour AES Distribution				12-hour AES Distribution			
	Controlled		Uncontrolled Peak Flow (m³/s)	Total Peak Flow (m³/s)	Controlled		Uncontrolled Peak Flow (m³/s)	Total Peak Flow (m³/s)
	Peak Flow (m³/s)	Storage Required (m³)			Peak Flow (m³/s)	Storage Required (m³)		
2-year	0.005	1,384	0.002	0.006	0.006	1,567	0.002	0.006
5-year	0.006	1,900	0.003	0.007	0.013	2,060	0.003	0.013
10-year	0.028	2,181	0.004	0.028	0.037	2,254	0.003	0.037
25-year	0.059	2,455	0.005	0.059	0.064	2,541	0.004	0.064
50-year	0.068	2,618	0.006	0.068	0.074	2,732	0.005	0.075
100-year	0.075	2,747	0.008	0.075	0.085	2,902	0.006	0.086
Regional	0.307	3,695	0.011	0.315	-			

1. Controlled flow for Catchment 201, 202 and 203 represents the peak flow from the orifice per the VO results. Refer to Appendix C for the orifice control curve used in VO.
2. Addition of individual flows do not exactly equal the total flow indicated. The difference in peak flows occurs because the total flow to the outlet is calculated in VO, which accounts for time to peak values.

As shown in Table 6, the storage requirement for the design storm events is dictated by the 12- hour design storm. The maximum required storage volume in addition to the rooftop storage provided is 2,902 m³ during the 100-year storm event. The storage required for the Regional storm is 3,695 m³, which needs to be provided in addition to the design storm storage. The proposed SWM pond provides 6,920 m³ of active storage between the bottom of the pond (246.60) and the emergency spillway (249.70), which is sufficient to accommodate both the design storm events and the Regional storm event (2,902 m³ + 3,695 m³ = 6,597 m³ required). During the Regional storm event, the maximum ponding level in the SWM pond is 249.61, providing 0.39 m of freeboard from the top of the pond at 250.00. Refer to Drawings C106 and C106A for the SWM pond design. The orifice control calculations and stage storage discharge table for the SWM pond are included in Appendix C.

Emergency overland flow for the Site is provided through a curb cut located in the northwest corner of the trailer parking area and a spillway into the SWM pond. Both the curb cut and inlet spillway were sized using the Regional storm flow for Catchments 201 and 202 (0.662 m³/s). The curb cut was sized as a weir using the full curb height (0.3 m) and a weir coefficient of 1.55. The minimum curb cut width is noted on Drawing C103A, and curb cut sizing calculations are included in Appendix C. The pond inlet spillway has been sized as 5 m wide to convey the Regional flow with a maximum depth of 0.06 m. Refer to Appendix C for the FlowMaster calculations.

An emergency spillway has also been designed to safely convey runoff from the uncontrolled Regulatory event (greater of the 100-year and Regional events) through the SWM pond to Salt Creek. The 100-year uncontrolled flows to the SWM pond are 0.902 m³/s and 0.506 m³/s for the 6-hour and 12-hour AES storm distribution, respectively. The Regional uncontrolled flow directed to the SWM pond is 0.734 m³/s. The maximum anticipated flow was used to size the emergency spillway (0.902 m³/s). The spillway is located on the northwest bank of the SWM pond and has been sized as a 5 m wide weir to convey the Regulatory flow with 0.06 m of freeboard to the top of the pond. Detailed emergency spillway calculations (FlowMaster) are provided in Appendix C.

4.4 Stormwater Quality Control

An "Enhanced Level of Protection" of 80% TSS removal is required to treat runoff leaving the Site. Water quality for each parcel is achieved with a Jellyfish Filtration System. An offline Jellyfish model JF12-24-5 has been sized to treat all stormwater that is directed to the stormwater tank, to provide a total TSS removal of 80% for Catchments 201 and 202. Refer to Drawing C104C for the Jellyfish system details. The Jellyfish Filtration System is certified with ISO 14034/ETV verification to provide the minimum 80% TSS removal requirement, as shown in the Jellyfish system specifications in Appendix C. It is noted that Catchments 203 and UC1 are fully landscaped and are therefore considered clean.

The proposed SWM pond and infiltration gallery also contribute to improving the water quality on site by providing retention and promoting infiltration and evapotranspiration. It is understood that per the hierarchical approach outlined in Town's Green Development Standards the first step is retention of stormwater however, filtration is provided upstream of retention in this treatment train. The reason for this is that the high groundwater elevation within the Site make it challenging to provide infiltration upstream of the pond and treatment of parking lot drainage is required prior to infiltration. Therefore, the treatment train approach as required by the Town's Green Development Standards is provided by the Jellyfish system and infiltration/evapotranspiration within the SWM pond.

4.5 Erosion Control

Extended Detention Drawdown Time

The SWM pond has been designed to detailed runoff from the 25 mm design storm event and the extended detention orifice within the outlet structure has been designed to release this volume over a minimum period of 48 hours.

Per the VO model prepared for the Site, the total runoff depth directed to the SWM pond during the 25 mm event is 18.96 mm. Given the total drainage area to the pond of 5.16 ha, this results in a total extended detention volume requirement of 978 m³.

The extended detention orifice has been designed to slowly release runoff from the 25 mm design storm event over a minimum period of 48 hours. A 50 mm diameter orifice is proposed within the outlet structure at an invert of 246.60. The top of extended detention is at an elevation of 247.40, resulting in a volume of 1,099 m³. Given the maximum head of 0.80 m acting on the orifice, MECP Equation 4.10 (Stormwater Management Planning and Design Manual, March 2003) was used to determine that the total extended detention volume is released over a period of 117 hours. Refer to the calculations in Appendix C for more details.

Onsite Retention

At a minimum, TRCA and the Town's CLI-ECA Stormwater Criteria required on-site retention of the first 5 mm of runoff generated from the total impervious area. The minimum requirement for retention of the first 5 mm of runoff from the impervious area on Site will be provided using an infiltration gallery. Refer to Section 4.7 for more details.

4.6 Groundwater Condition

A Hydrogeological Investigation Report for the subject Site was completed by Crozier (May 2026), which details the groundwater conditions for the Site. The major conclusions of the Hydrogeological Investigation Report are summarized below:

- The seasonally high groundwater elevation varies across the Site. Table 5 of the Hydrogeological Investigation Report was referenced when designing the proposed infiltration gallery. Based on the well locations on Site, MW202-25 is the closest to the proposed infiltration gallery, which has a high groundwater elevation of 244.20.
- An infiltration rate of 12.7 mm/hr was determined based on the grain size curves in the Geotechnical Investigation & Slope Assessment report prepared by MTE (May 15, 2025) for the Site as part of the work completed for the Secondary Plan Application. A safety factor of 1.5 was applied to this, to achieve an effective infiltration rate of 8.5 mm/hr. This infiltration rate was used when designing the infiltration gallery.

4.7 Water Balance

A water balance analysis is required using the average and more frequent precipitation events that comprise of the bulk volume of annual precipitation to ensure maintenance of pre-development water balance following development. Water balance calculations were completed using the Thornthwaite and Mather Method (1957) as part of the Hydrogeological Investigation. To mitigate the infiltration deficit within the entire Site, the Hydrogeological Investigation indicates that the infiltration target is 6,702 m³/year. Refer to the Hydrogeological Investigation Report prepared by Crozier under separate cover (May 2026) for more details.

In addition to the water balance requirement, TRCA has an erosion criterion that applies to the Site, which requires retention of 5 mm of runoff generated from the total impervious area. Therefore, the larger infiltration target between the water balance and erosion control requirements must be applied.

As indicated in the Hydrogeological Investigation Report (Crozier, May 2026), the infiltration target for the Site is 4.8 mm of rainfall over the impervious area to match pre-development infiltration rates, which equates to a total volume of 195 m³ (4.05 ha impervious area x 4.8 mm). The erosion control infiltration target is 203 m³ (4.05 ha impervious area x 5 mm). Therefore, the infiltration requirement for the Site is dictated by the erosion control criteria, resulting in a target of 203 m³.

One (1) infiltration gallery is proposed to satisfy the water balance and erosion control requirements for the Site. The gallery is located at the bottom of the stormwater management pond. The internal storm sewer system is designed with the proposed Jellyfish upstream of the pond to ensure clean runoff is directed to the pond for infiltration. The infiltration gallery has been designed using a design infiltration rate of 8.5 mm/hr as outlined in Section 4.5. A minimum 1.0 m clearance with the high groundwater elevation was provided. Refer to Table 7 for more details on the design of the infiltration gallery and Appendix C for the sizing calculations.

Table 7: Summary of Proposed Infiltration Gallery

Footprint (m²)	Depth (m)	Volume (m³)	Drawdown Time (hours)	Bottom of Gallery Elevation (masl)	Groundwater Elevation¹ (masl)
800	0.65	208	31	245.85	244.20

1. Groundwater elevation in MW202-25 per Table 5 of the Hydrogeological Investigation Report (Crozier, May 2026)

4.8 Feature-Based Water Balance

A feature-based water balance assessment was undertaken by Crozier under separate cover to estimate the potential impact of the proposed development on Salt Creek and its surrounding wetland. A Thornthwaite and Mather assessment was conducted using the catchment areas directed to the wetland in the pre- and post-development scenarios. The results of the analysis show that the area directed to the wetland is increased by roughly 16% post-development. Refer to the Hydrogeological Investigation Report prepared by Crozier (May 2026) for more details.

5.0 Erosion and Sediment Control

The following erosion and sediment control features will be implemented for the project.

5.1 Erosion Control Measures

Interceptor Swales – Interceptor swales have been designed with reduced slope gradients to reduce erosion potential during the construction period. The interceptor swales have been designed to convey the 100-year storm event. Refer to Appendix D for calculations related to the interceptor swale design.

5.2 Sediment Control Measures

Stone Mud Mat – A stone mud mat will be provided to minimize the migration of unwanted material on to the adjacent ROWs. The construction access must be maintained (cleaned, swept and

flushed) to minimize any disruption to the Regional ROW. Construction access for the Site is provided through Airport Road.

Silt Fence – Sediment control fence will be installed in accordance with the drawings. The erosion and sediment control fencing will be monitored on a regular basis and repaired/replaced as required.

Temporary Sediment Basin – A temporary sediment basin has been designed to intercept sediment laden water and allow for settling of suspended soil particles. Refer to Appendix D for additional details and calculations related to the permanent pool, active storage, and outlet structure design.

Sediment Curtain – A sediment curtain will be installed in the temporary sediment basin and will be located between the swale inlets and basin outlet. The curtain keeps sediment contained to the area between the curtain and the pond bank and slows the movement of water in the isolated area, providing additional sediment control within the temporary sediment basins. The sediment curtains will be monitored on a regular basis and repaired/replaced as required.

6.0 Conclusions

Based on the conclusions and recommendations outlined in this report, the proposed development can be serviced for water, sanitary, and stormwater in general conformance with the servicing and stormwater management design outlined in the Secondary Plan prepared by Crozier (March 2026). The design is also in accordance with the Town of Caledon and Region of Peel requirements and standards.

Should you have any questions or require any further information, please do not hesitate to contact us.

Respectfully submitted,

C.F. CROZIER & ASSOCIATES INC.

Isabelle Forsyth, P.Eng.
Project Engineer



C.F. CROZIER & ASSOCIATES INC.

Katrina Weel, P.Eng.
Project Engineer



C.F. CROZIER & ASSOCIATES INC.

Julie Scott, P.Eng.
Manager, Land Development



J:\2200\2278- Broccolini\7228 - Broccolini Secondary Plan\Reports\Civil\Broccolini Site\2026.05.01_Second Submission_Phase 1\7228_S&SWM Report.docx

APPENDIX A

Sanitary Flow Calculations



Project: Broccolini Airport Rd
Project No.: 2278-7228

Design: A.H.
Check: K.W./J.S.

Date: 2026-05-01

Proposed Sanitary Design Flow - Building A			
North Parcel Area	5.24	ha	
Building A GFA	2.31	ha	
Population Density	70	persons/ha	
Population*	367	persons	
Design Criteria			
Total Peak Flow = Peak Daily Flow + Infiltration Allowance			
Peak Factor =	4.0	Harmon Peaking Factor	
Average Industrial Flow =	270.0	L/cap/d	
Infiltration =	0.26	L/s/ha	
Sanitary Design Flow - Unit Sewage Flow Rate			
Average Daily Flow =	1.15	L/s	
Peak Factor =	4.0		
Peak Daily Flow =	4.59	L/s	
Infiltration Flow =	1.36	L/s	
Total Peak Flow =	5.95	L/s	
			Region of Peel - Linear Wastewater Standards (March 2023)
			*Population calculation and is based on the Site Plan prepared by Powers Brown Architecture.

SANITARY SEWER DESIGN SHEET

Broccolini Site - Phase 1, Region of Peel, Town of Caledon



Project #: 2278-7228
Updated Date: 2025.10.29
Designed: V.M.
Checked: J.S.

Min Diameter = 200 mm
Mannings 'n'= 0.013
Min. Velocity = 0.75 m/s
Max. Velocity = 3.50 m/s
Avg. Domestic Flow = 270.0 l/c/d
Infiltration = 0.260 l/s/ha
Max. Peaking Factor = 4.00
Min. Peaking Factor= 1.50

Factor of Safety = 20 %
NOMINAL PIPE SIZE USED

DESCRIPTION	FROM MH	TO MH	EXTERNAL / BLOCK AREA							COMMERCIAL/INDUSTRIAL/INSTITUTIONAL						FLOW CALCULATIONS							PIPE DATA						NOTES	
			AREA (ha)	ACC. AREA (ha)	UNITS (#)	DENISTY (P/ha)	DENSITY (P/unit)	EXT POP	ACCUM. EXT POP.	AREA (ha)	ACC. AREA (ha)	EQUIV. POP. (p/ha)	FLOW RATE (l/s/ha)	EQUIV. POP.	ACCUM. EQUIV. POP.	INFILTRATION (l/s)	TOTAL ACCUM. POP.	PEAKING FACTOR	POP. FLOW (l/s)	CONSTANT COMM. FLOW (l/s)	ACCUM. COMM. FLOW (l/s)	TOTAL FLOW (l/s)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (l/s)	FULL FLOW VELOCITY (m/s)	ACTUAL VELOCITY (m/s)	PERCENT FULL (%)		
BUILDING A																														
3A	PLUG103A	MH105A								0.34	0.34	70		24	24	0.09	23.8	4.00	0.30			0.39	2.00	200	46.38	1.48	0.45	0.8%		
101	MH105A	MH104A								0.23	0.57	70		16	40	0.15	39.9	4.00	0.50			0.65	0.50	200	23.19	0.74	0.32	2.8%		
2A	PLUG102A	MH104A								0.37	0.37	70		26	26	0.10	25.9	4.00	0.32			0.42	2.00	200	46.38	1.48	0.46	0.9%		
102	MH104A	MH103A								0.27	1.21	70		19	85	0.31	84.7	4.00	1.06			1.37	0.50	200	23.19	0.74	0.40	5.9%		
103	MH103A	MH102A								0.08	1.29	70		6	90	0.34	90.3	4.00	1.13			1.46	0.50	200	23.19	0.74	0.41	6.3%		
1A	PLUG101A	MH102A								0.40	0.40	70		28	28	0.10	28	4.00	0.35			0.45	2.00	200	46.38	1.48	0.47	1.0%		
5A	PLUG105A	MH109A								0.65	0.65	70		46	46	0.17	45.5	4.00	0.57			0.74	2.00	200	46.38	1.48	0.55	1.6%		
107	MH109A	MH108A								1.67	2.32	70		117	163	0.60	162.70184	4.00	2.03			2.64	0.50	200	23.19	0.74	0.49	11.4%		
4A	PLUG104A	MH108A								0.55	0.55	70		39	39	0.14	38.5	4.00	0.48			0.62	2.00	200	46.38	1.48	0.52	1.3%		
106	MH108A	MH107A								0.49	3.36	70		34	236	0.87	235.50184	4.00	2.94			3.82	0.50	200	23.19	0.74	0.55	16.5%		
105	MH107A	MH106A								0.10	3.46	70		7	243	0.90	242.50184	4.00	3.03			3.93	0.50	200	23.19	0.74	0.55	17.0%		
104	MH106A	MH102A								0.09	3.55	70		6	249	0.92	248.80184	4.00	3.11			4.03	0.50	200	23.19	0.74	0.55	17.4%		
	MH102A	MH101A									5.24				367	1.36	367.10184	4.00	4.59			5.95	0.45	200	22.00	0.70	0.59	27.1%		
	MH101A	EX.MH31									5.24				367	1.36	367.10184	4.00	4.59			5.95	2.00	200	46.38	1.48	1.02	12.8%		
																									0.00					

APPENDIX B

Water Demand Calculations



Project: Broccolini Airport Rd
Project No.: 2278-7228

Design: A.H.
Check: K.W./J.S.

Date: 2026-05-01

Water Demand - Building A				
North Parcel Area	5.24	ha		
Population Density	70	persons/ha		
Population	367	persons		
<u>Design Criteria:</u>				
	Average Daily Demand:	250	L/employee.day	Water Demand Criteria as per Peel's 2020 Development Charges Background Study
	Maximum Daily Demand Peaking Factor:	1.4	-	
	Peak Hourly Demand Peaking Factor:	3.0	-	
<u>Domestic Water Demand:</u>				
Average Daily Demand:	91775	L/day		*Population calculation and is based on the Site Plan prepared by Powers Brown Architecture.
	1.06	L/s		
Maximum Daily Demand:	128486	L/day		
	1.49	L/s		
Peak Hourly Demand:	275326	L/day		
	3.19	L/s		

Water and Wastewater Modelling Demand Table

Site Plan Applications

Version	Date	Description of Revision
1.0	January 10 2023	Posted to Peel Website
2.0	August 30 2024	Reflects 2023 Linear Wastewater Standards and ICI population estimates as per Peel 2020 DC background study

Introduction

Water and wastewater modelling may be required as a condition of the development approval process or prior to regional site servicing connection approval where intensification is proposed, where a possible increase in water demand or wastewater discharge is identified or where deemed necessary by Regional staff.

A completed table includes the Professional Engineer's signature and stamp as well as a site servicing concept. The table will be deemed complete once all the information below is submitted and/or included. Modelling will commence once the information is deemed complete. All required calculations must be submitted with the completed demand table. The calculations shall be based on the specific development proposal.

Application Information

Application Number:	2025-0067
Address:	0 Airport Road
Consulting Engineer:	Crozier Consulting Engineers
Date Prepared:	May 1, 2026

Population

Existing

		Units	Persons
1	Residential ⁸⁾		0
2	Institutional/Employment ⁸⁾		0
3	Total		0

Proposed

			Units	Persons
4	Residential ¹⁾	singles/semis (4.2 ppu)		
5		Townhomes (3.4 ppu)		
6		Large apartments (>1 bedroom – 3.1 ppu)		
7		Small apartments (<=1 bedroom – 1.7 ppu)		
8		Total proposed residential		
9	Proposed Institutional ²⁾			
10	Proposed employment ³⁾			362
11	Total Proposed			362

Other

12	Existing gross floor area for commercial and/or retail (sqm)	0
13	Proposed gross floor area for commercial and/or retail (sqm)	23116.5
14	Land area (ha)	5.18

Water Connection**Hydrant flow test ⁴⁾**

15	Location 1	Airport Road
16	Location 2	

WATER AND WASTEWATER MODELLING DEMAND TABLE

		Pressure (kPa)	Flow (L/s)	Time
17	Minimum water pressure	31	2600	12:40pm
18	Maximum water pressure	54	919	12:40pm

Water Demands (L/s)

		Use 1 ⁶⁾	Use 2 ⁶⁾	Use 3 ⁶⁾	Total
19	Existing fire flow ^{5) 8)}				0
20	Proposed average day flow				1.05
21	Proposed maximum day flow				1.47
22	Proposed peak hour flow				3.15
23	Proposed fire flow ⁵⁾				150

Water calculations

Please use the following updated typical water demand criteria as per Peel's 2020 Development Charges background study.

Population Type	Unit	Average Consumption Rate	Max Day Factor	Peak Hour Factor
Residential	L/cap/d	270	1.8	3.0
Institutional/Commercial/Industrial	L/emp/d	250	1.4	3.0

Wastewater Connection

Wastewater Effluent (L/s)

		Discharge location ⁷⁾	Flow
24	Existing effluent ⁸⁾		0
25	Proposed effluent	Airport Road	5.88
26	Proposed effluent		
27	Proposed effluent		
28	Proposed additional effluent ⁸⁾		
29	Other proposed effluent*		
30	Total proposed effluent		5.88

*Please specify other proposed effluent (ex. occasional tank purges, off peak discharge, pool drainage)

N/A

Wastewater calculations

Please use the following updated daily per capita as per 2023 Peel Linear Wastewater Standards

Population Type	Unit	Average Day Demand	Min Peaking Factor	Max Peaking Factor	Inflow and Infiltration**
Residential	L/cap/d	290	2	4	0.26L/s/Ha
Non-residential	L/emp/d	270	2	4	0.26L/s/Ha

**For maintenance holes that are flood prone or located in low lying areas, an extra 0.28 L/s per maintenance hole may be added to the I&I calculation.

Notes

- 1) In accordance with Peel Linear Wastewater Standards and Region of Peel 2020 DC background Study
- 2) refer to Peel Linear Wastewater Standards
- 3) For the commercial and industrial design flow calculations, please refer to Schedule 8b on page A-9 of the Region of Peel 2020 DC background Study to determine population.
- 4) Please include the graphs associated with the hydrant flow test data. Hydrant flow tests should be performed within 2 years of submission to the Region. The Region will not permit hydrant flow tests during the winter, please contact Region Water Operations for scheduling. The Region reserves the right to request an updated hydrant flow test as required at any time.
- 5) Please reference the Fire Underwriters Survey Document
- 6) Please identify the flows for each use type, **if applicable**
- 7) Please include drainage plan for multiple discharge locations
- 8) For Intensification, sites with additions to buildings or additional buildings please provide existing flow for existing buildings and the added flows for the new proposal, **if applicable**

Water Supply for Public Fire Protection - 2020
Fire Underwriters Survey
Part II - Guide for Determination of Required Fire Flow

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

$$RFF = 220 * C * \sqrt{A}$$

where

RFF = the required fire flow in litres per minute

C = coefficient related to the type of construction:

=	1.5	for Type V Wood Frame Construction (structure essentially all combustible)
=	0.8	for Type IV-A Mass Timber Construction (encapsulated mass timber)
=	0.9	for Type IV-B Mass Timber Construction (rated mass timber)
=	1.0	for Type IV-C Mass Timber Construction (ordinary mass timber)
=	1.5	for Type IV-D Mass Timber Construction (un-rated mass timber)
=	1.0	for Type III Ordinary Construction (brick or other masonry walls, combustible floor and interior)
=	0.8	for Type II Non-combustible Construction (unprotected metal structural components)
=	0.6	for Type I Fire-resistive Construction (fully protected frame, floors, roof)

Proposed Buildings

Floor Area 23,116.5 sq.m

Area = 23,117 sq.m

C = 0.6 Assumes Type I Fire-resistive Construction (fully protected frame, floors, roof)

Therefore RFF = 20,070 L/min

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

Non-Combustible	-25%	Free Burning	15%
Limited Combustible	-15%	Rapid Burning	25%
Combustible	0% (No Change)		

Limited Combustible	-15% reduction
---------------------	----------------

-3,011 L/min reduction
17,060 L/min

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

3. Sprinklers - The value obtained in No. 2 above may be reduced by up to 50% for complete automatic sprinkler protection. The initial credit for the system will be a maximum of 30% for an adequately designed system conforming to NFPA 13 and other NFPA sprinkler standards.

As part of this analysis, the building will have sprinkler protection:

50%

8,530 L/min reduction

Water Supply for Public Fire Protection - 2020
Fire Underwriters Survey

Part II - Guide for Determination of Required Fire Flow

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

Separation	Charge	Separation	Charge
0 to 3 m	25%	20.1 to 30 m	10%
3.1 to 10 m	20%	> 30 m	0%
10.1 to 20 m	15%		

Exposed buildings

Name	Distance (m)	Charge (%)	Surcharge (L/min)
E	> 30	0%	-
W	> 30	0%	-
N	> 30	0%	-
S	> 30	0%	-

0 L/min Surcharge

Determine Required Fire Flow

No. 1	20,070
No. 2	-3,011 reduction
No. 3	-8,530 reduction
No. 4	0 surcharge

Required Flow: 8,530 L/min
Rounded to nearest 1000 L/min: 9,000 L/min or **150.0 L/s**
 2,378 USGPM



Hydrant Testing Ontario

Tel: 289-354-1942

Info@HTOntario.ca

**REPORT
Nº. 2592**

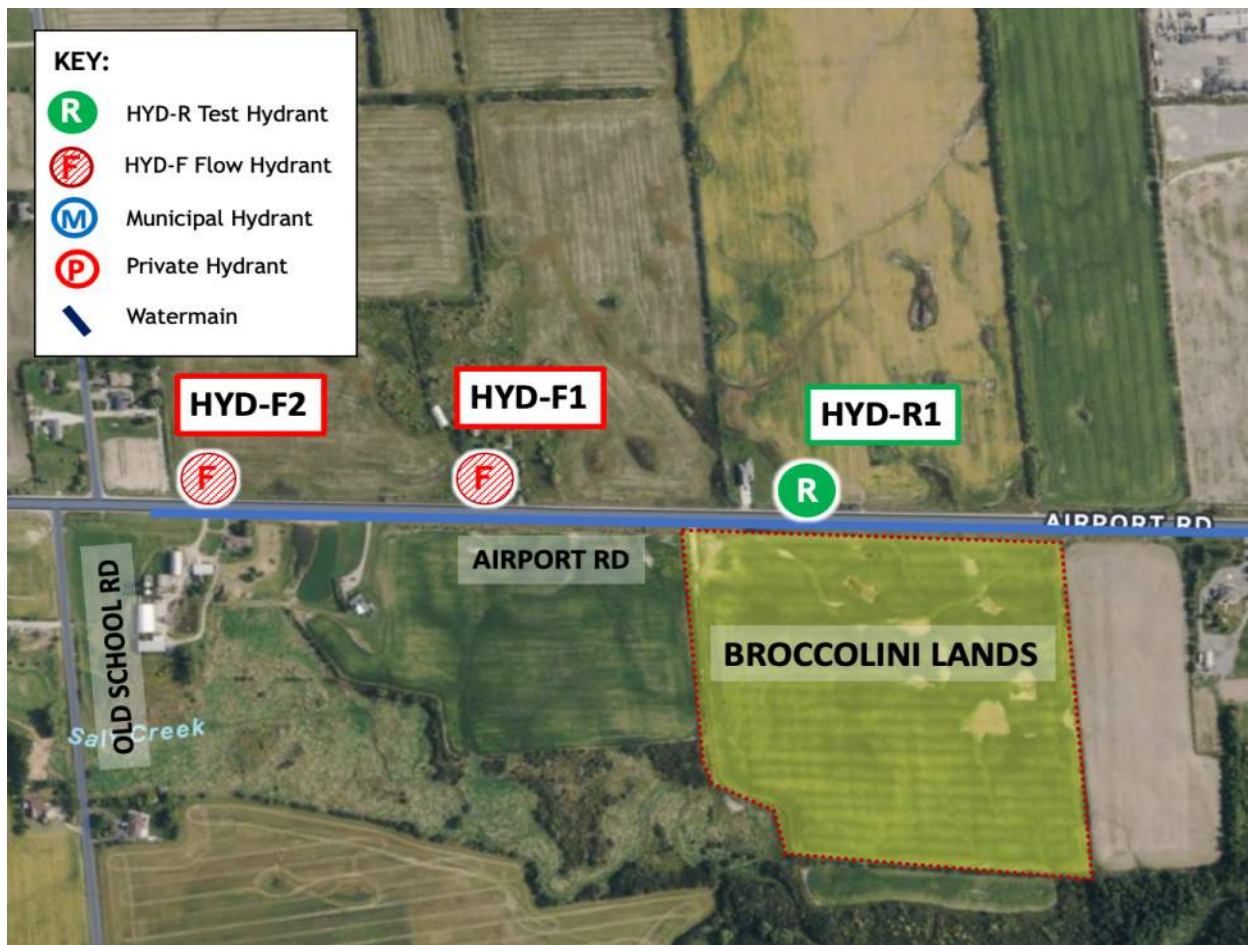
November 12, 2025

To: Alex Di Cesare
Development Manager
Broccolini Airport Road LP
2680 Skymark Avenue, Suite 800
Mississauga, Ontario, L4W 5L6

RE: Hydrant Flow Test - BROCCOLINI LANDS

Please find the Report for the following works

Scope: Conducted Hydrant Flow Test as per NFPA291 Recommended Practices for Water Flow Testing and Marking of Hydrants.





CALEDON

October 28, 2025

TIME: 12:40 PM

R - TEST HYDRANT

AIRPORT ROAD

HYDRANT No. HYD-R1

HYDRANT MODEL:

CENTURY

COLOUR: BLUE

STATIC PRESSURE psi (hr-20^{0.54}):

65

VARIANCE: 52%

Q - FLOW HYDRANT

AIRPORT ROAD

HYDRANT No. HYD-F1/2

HYDRANT MODEL:

CENTURY

COLOUR: BLUE

No. Outlets	Residual Pressure ($h_f - R^{0.54}$)	Orifice Dia Dia. (in.) (d^2)	Coefficient	Nozzle PSI ($\sqrt{psi}$)	Q = Flow (USGPM) $Q = 29.83 (c) (d2) (\sqrt{psi})$
1	54	2.5	0.90	30	919
2	43	2.5	0.90	15	650
3	37	2.5	0.90	25	839
4	31	2.5	0.90	15	650
Q_F = Total Flow (USGPM)					2599

Q_R = flow predicted @ 20 psi

$$Q_R = Q_F * (H_r - 20^{0.54}) / (H_f - R^{0.54})$$

3024

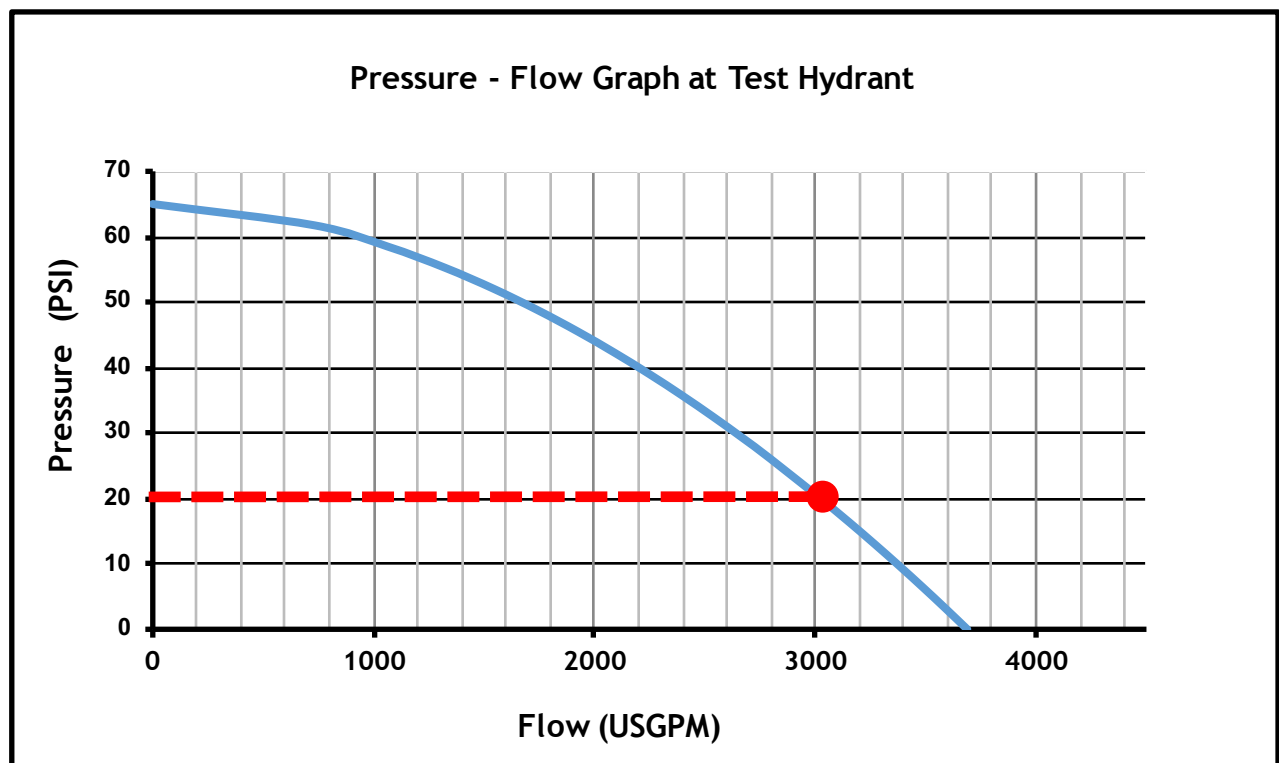
USGPM

191

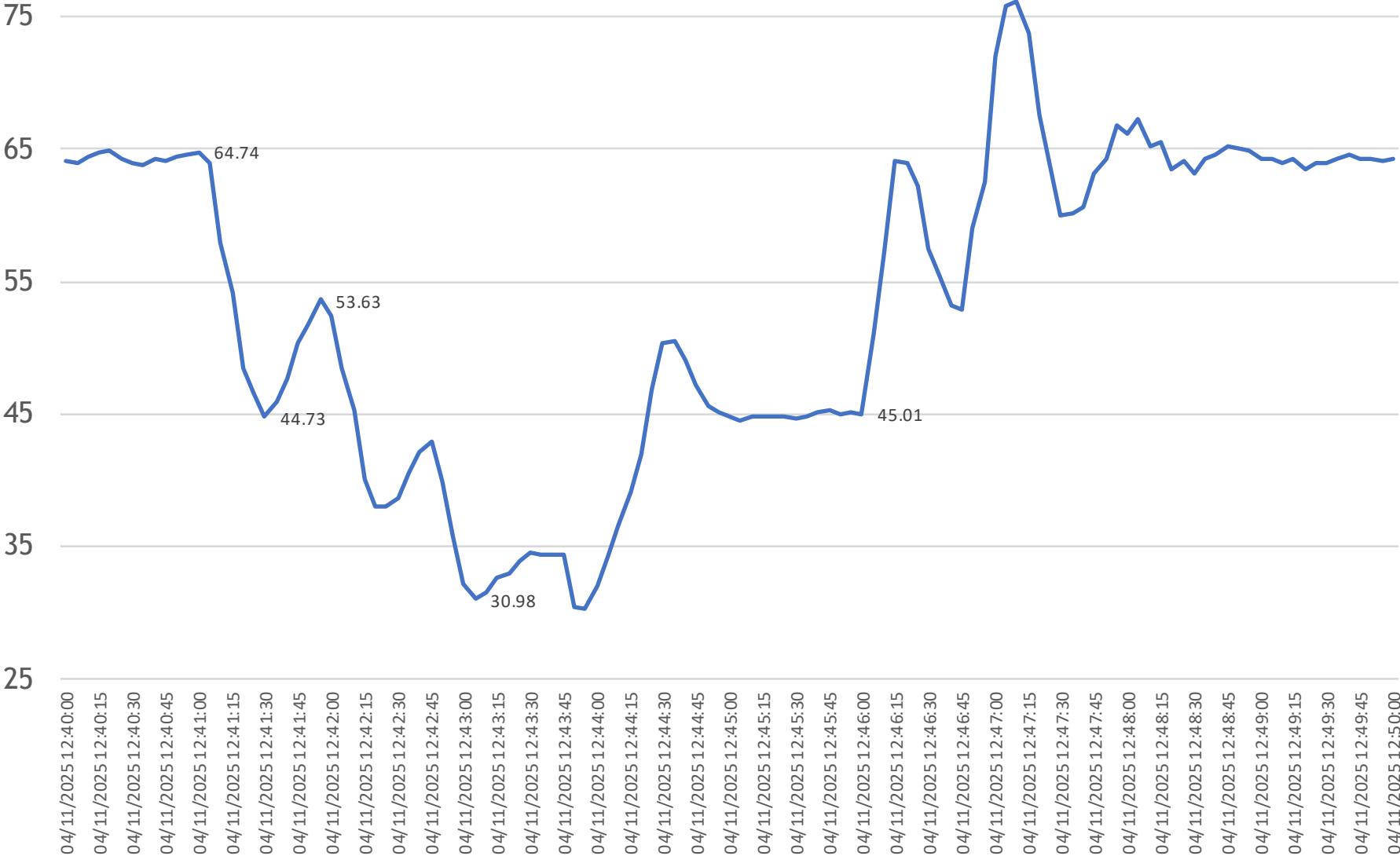
L/s

NFPA Rating:

CLASS AA - BLUE



HYD R1 RESIDUAL PRESSURE psi - BROCCOLINI LANDS



Test Conclusion

The system at the time of testing produced a theoretical projected flow rate of:

LOCATION	Total USGPM	USGPM at 20 psi	lps at 20 psi	Test #
AIRPORT ROAD	2599	3024	191	1

Hydrants are classified in accordance with their rated capacities as per NFPA291.

COLOUR	CLASS	Available Flow @ 20psi
BLUE	AA	1500 GPM or more
GREEN	A	1000 - 1499 GPM
ORANGE	B	500 - 999 GPM
RED	C	Below 500 GPM

We strongly feel that all attempts have been made to ensure that the required data as stipulated was captured, stored and presented in an accurate, efficient and timely manner for the required period.

We look forward to working with you in the future.

Please feel free to contact the undersigned should you require any further information.

Best Regards



Rob Gamache
Manager of O
Hydrant Testing Ontario

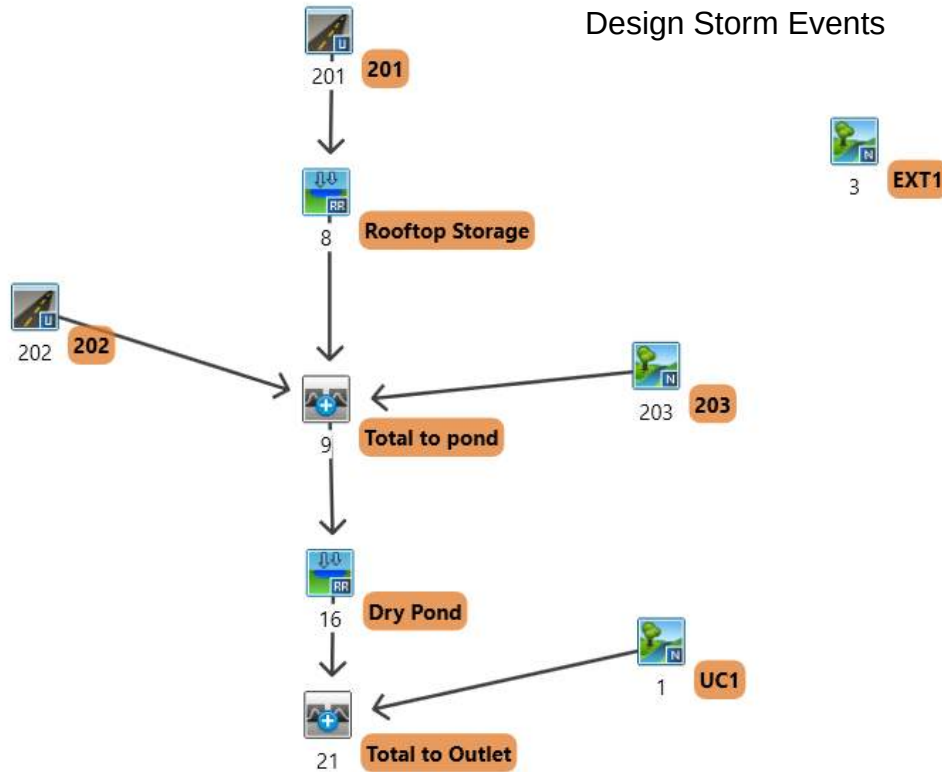
Info@HTOntario.ca

APPENDIX C

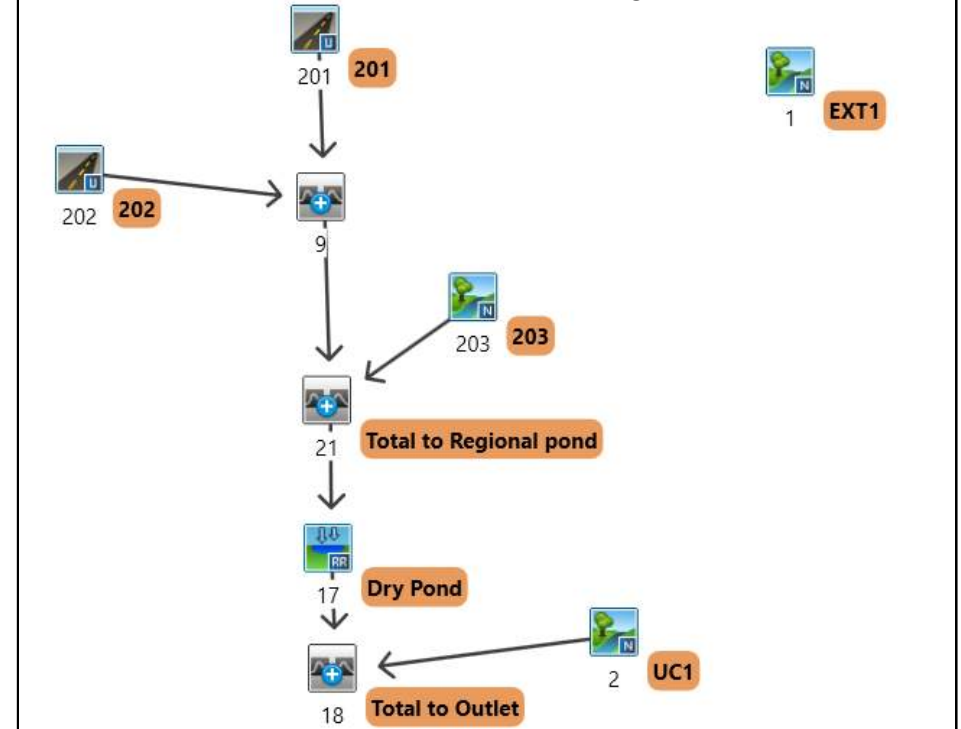
Stormwater Management Calculations

Proposed Conditions VO Model Schematic

Design Storm Events



Regional Storm Event



NASHYD - CN & IA Values

Parameter	Hydrologic Soil Group	Soil Category	Landuse							
			Woodland	SWMF / Wetland	Meadow	Cultivated/ Crops	Lawn / Landscape	Gravel	Paved / Concrete	Building
CN Values	D	Clay Loam/Clay	77	50	78	86	81	98	98	98
	N/A									
IA Values			10	10	8	7	5	2	2	1

*Note these values are taken from the MTO manual, check against municipal standards for CN values to be used

Catchment ID	Hydrologic Soil Group	Soil Category	Area (ha) for each land use and soil group								Subtotal Area (ha)	Weighted CN	Weighted IA
			Woodland	SWMF / Wetland	Meadow	Cultivated / Crops	Lawn / Landscape	Gravel	Paved / Concrete	Building			
203	D	Clay Loam/Clay		0.65							0.65	50	10.0
	N/A										0.00		
	Total		0.00	0.65	0.00	0.00	0.00	0.00	0.00	0.00	0.65		
UC1	D	Clay Loam/Clay					0.08				0.08	81	5.0
	N/A										0.00		
	Total		0.00	0.00	0.00	0.00	0.08	0.00	0.00	0.00	0.08		
EXT1	D	Clay Loam/Clay				1.93					1.93	86	7.0
	N/A										0.00		
	Total		0.00	0.00	0.00	1.93	0.00	0.00	0.00	0.00	1.93		



Project Name: 0 Airport Road
 Project Number: 2278-7228
 Date: 2026-04-16
 By: IF
 Checked by: JS

Time to Peak - Bransby Williams & Airport Methods

Airport Method

$$T_c = \frac{3.26 * (1.1 - C) * L^{0.5}}{S_w^{0.33}}$$

Bransby William

$$T_c = \frac{0.057 * L}{S_w^{0.2} * A^{0.1}}$$

Equation from MTO Drainage Management Manual 1997

Catchment ID	Subtotal Area (ha)	Runoff Coefficient	Method	Flow Path Length (m)	US Elev	DS Elev	Flow Path Drop (m)	Slope (%)	Tc (hr)	Tp (hr)
203	0.65	0.25	AIRPORT	86				0.50	0.538	0.36
UC1	0.08	0.25	AIRPORT	132	251.76	249.65	2.11	1.60	0.455	0.30
EXT1	1.93	0.25	AIRPORT	229	253.50	249.50	4.00	1.75	0.581	0.39

*Note: A minimum Tp of 0.17hrs (10 minutes) has been applied



CROZIER
CONSULTING ENGINEERS

Project Name: 0 Airport Road
Project Number: 2278-7228
Date: 2026-04-16
By: IF
Checked by: JS

NASHYD Summary

Catchment ID	Area (ha)	DT (min)	CN Value	CN*	CN(III)	IA (mm)	N	Tp (hr)
203	0.65	5	50	51	71	10.0	3	0.36
UC1	0.08	5	81	81	91	5.0	3	0.30
EXT1	1.93	5	86	89	95	7.0	3	0.39

STANDHYD - TIMP, XIMP & CN Values

Parameter	Hydrologic Soil Group	Landuse				
		Woodland	SWMF / Welland	Meadow	Cultivated/ Crops	Lawn / Landscape
CN Values	D	77	50	78	86	81
	N/A					
IA Values		10	10	8	7	5

*Note these values are taken from the MTO manual, check against municipal standards for CN values to be used

Catchment ID	Hydrologic Soil Group	Pervious Areas (ha)						Impervious Areas (ha)			Results						
		Woodland	SWMF (Landscaped)	Meadow	Crops	Lawn / Landscape	Pervious Total Area (ha)	Paved / Concrete / Building		SWMF (Permanent pool)	Impervious Total Area (ha)	Subcatchment Total Area (ha)	TIMP (Fraction)	XIMP (Fraction)	Weighted RC	Weighted Pervious CN	Weighted Pervious IA
		N/A	N/A	N/A	N/A	N/A		To Outlet	To Pervious	To Outlet							
201	D						0.00	2.31			2.31	2.31	0.99	0.99	0.90	81	5
	N/A						0.00				0.00	0.00					
	Total	0.00	0.00	0.00	0.00	0.00	0.00	2.31	0.00	0.00	2.31	2.31					
202	D					0.46	0.46	1.74			1.74	2.20	0.79	0.79	0.76	81	5
	N/A						0.00				0.00	0.00					
	Total	0.00	0.00	0.00	0.00	0.46	0.46	1.74	0.00	0.00	1.74	2.20					



Project Name: 0 Airport Road
Project Number: 2278-7228
Date: 2026-04-16
By: IF
Checked by: JS

STANDHYD Summary

Catchment ID	Area (ha)	DT (min)	TIMP	XIMP	Pervious Area							Impervious Area			
					CN Value	CN*	CN(III)	IA (mm)	SLPP (%)	LGP (m)	MNI	DPSI	SLPI (%)	LGI (m)	MNI
201	2.31	5	0.99	0.99	81	81	91	5	2.0	40	0.25	1	1	AUTO	0.013
202	2.20	5	0.79	0.79	81	81	91	5	2.0	40	0.25	2	1	AUTO	0.013

North Property Line Swale

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.030	
Channel Slope	0.00300	m/m
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Discharge	0.264	m³/s

Results

Normal Depth	0.39	m
Flow Area	0.45	m²
Wetted Perimeter	2.44	m
Hydraulic Radius	0.18	m
Top Width	2.32	m
Critical Depth	0.28	m
Critical Slope	0.01834	m/m
Velocity	0.59	m/s
Velocity Head	0.02	m
Specific Energy	0.40	m
Froude Number	0.43	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.39	m
Critical Depth	0.28	m
Channel Slope	0.00300	m/m
Critical Slope	0.01834	m/m

Dry Pond Operation Levels

ED Orifice

ED Orifice Coefficient: 0.62
ED Orifice Diameter: 0.050 m
ED Orifice Invert Elevation: 246.60 m
ED Orifice Centroid Elevation: 246.63 m

Orifice 1

Orifice 1 Coefficient: 0.62
Orifice 1 Diameter: 0.235 m
Orifice 1 Invert Elevation: 247.80 m
Orifice 1 Centroid Elevation: 247.92 m

Orifice 2

Orifice 2 Coefficient: 0.62
Orifice 2 Diameter: 0.250 m
Orifice 2 Invert Elevation: 248.32 m
Orifice 2 Centroid Elevation: 248.44

Operating Level	Elev.	Depth Above Bottom of Tank	Depth Above Orifice	Total Storage Volume	Discharge ED Orifice	Discharge Orifice 1	Discharge Orifice 2	Total Discharge	Storage
	(m)	(m)	(m)	(cu.m)	(cu.m/s)	(cu.m/s)	(cu.m/s)	(cu.m/s)	(ha-m)
Bottom of Pond	246.60	0.00	0.00	0	0.000	0.000	0.000	0.000	0.000
	246.70	0.10	0.10	116	0.001	0.000	0.000	0.001	0.012
	246.80	0.20	0.20	238	0.002	0.000	0.000	0.002	0.024
	246.90	0.30	0.30	366	0.003	0.000	0.000	0.003	0.037
	247.00	0.40	0.40	500	0.003	0.000	0.000	0.003	0.050
	247.10	0.50	0.50	640	0.004	0.000	0.000	0.004	0.064
	247.20	0.60	0.60	786	0.004	0.000	0.000	0.004	0.079
	247.30	0.70	0.70	939	0.004	0.000	0.000	0.004	0.094
Top of Extended Detention	247.40	0.80	0.80	1,099	0.005	0.000	0.000	0.005	0.110
	247.50	0.90	0.90	1,265	0.005	0.000	0.000	0.005	0.127
	247.60	1.00	1.00	1,439	0.005	0.000	0.000	0.005	0.144
2-Year Water Level	247.67	1.07	1.07	1,567	0.006	0.000	0.000	0.006	0.157
	247.70	1.10	1.10	1,620	0.006	0.000	0.000	0.006	0.162
	247.80	1.20	1.20	1,808	0.006	0.000	0.000	0.006	0.181
	247.90	1.30	1.30	2,004	0.006	0.000	0.000	0.006	0.200
5-Year Water Level	247.93	1.33	1.33	2,060	0.006	0.007	0.000	0.013	0.206
	248.00	1.40	1.40	2,207	0.006	0.034	0.000	0.041	0.221
10-Year Water Level	248.02	1.42	1.42	2,254	0.006	0.039	0.000	0.045	0.225
	248.10	1.50	1.50	2,418	0.007	0.051	0.000	0.057	0.242
25 Year Water Level	248.16	1.56	1.56	2,541	0.007	0.057	0.000	0.064	0.254
	248.20	1.60	1.60	2,637	0.007	0.063	0.000	0.070	0.264
50-Year Water Level	248.24	1.64	1.64	2,732	0.007	0.067	0.000	0.074	0.273
	248.30	1.70	1.70	2,864	0.007	0.074	0.000	0.081	0.286
100-Year Water Level	248.32	1.72	1.72	2,902	0.007	0.078	0.000	0.085	0.290
	248.40	1.80	1.80	3,099	0.007	0.083	0.000	0.090	0.310
	248.50	1.90	1.90	3,342	0.007	0.091	0.033	0.131	0.334
	248.60	2.00	2.00	3,593	0.008	0.098	0.054	0.160	0.359
	248.70	2.10	2.10	3,852	0.008	0.105	0.069	0.182	0.385
	248.80	2.20	2.20	4,119	0.008	0.112	0.081	0.201	0.412
	248.90	2.30	2.30	4,395	0.008	0.118	0.091	0.218	0.440
	249.00	2.40	2.40	4,679	0.008	0.124	0.101	0.233	0.468
	249.10	2.50	2.50	4,972	0.008	0.130	0.109	0.247	0.497
	249.20	2.60	2.60	5,274	0.009	0.135	0.117	0.261	0.527
	249.30	2.70	2.70	5,585	0.009	0.140	0.125	0.274	0.559
	249.40	2.80	2.80	5,905	0.009	0.145	0.132	0.286	0.591
	249.50	2.90	2.90	6,234	0.009	0.150	0.139	0.298	0.623
	249.60	3.00	3.00	6,572	0.009	0.155	0.145	0.309	0.657
Regional Water Level	249.61	3.01	3.01	6,597	0.009	0.155	0.146	0.310	0.660
Top of Active Storage	249.70	3.10	3.10	6,920	0.009	0.159	0.151	0.320	0.692
	249.80	3.20	3.20	7,277	0.010	0.163	0.157	0.330	0.728
	249.90	3.30	3.30	7,644	0.010	0.168	0.163	0.340	0.764
Top of Pond	250.00	3.40	3.40	8,020	0.010	0.172	0.168	0.350	0.802



Project: Broccolini Site Plan
File: 2278-7228
Designed by: IF
Checked by:
Date: 4-May-26

Extended Detention Volume Calculations - Dry Pond

Area Name	Drainage Area		Percent Impervious
Ato-pond =	5.16	ha	79.0%
Vextended detention (25 mm event) =	978	m ³	

Base Runoff: $V = 18.96$ mm Runoff volume from 25mm VO Model for 71.08 ha upstream drainage area

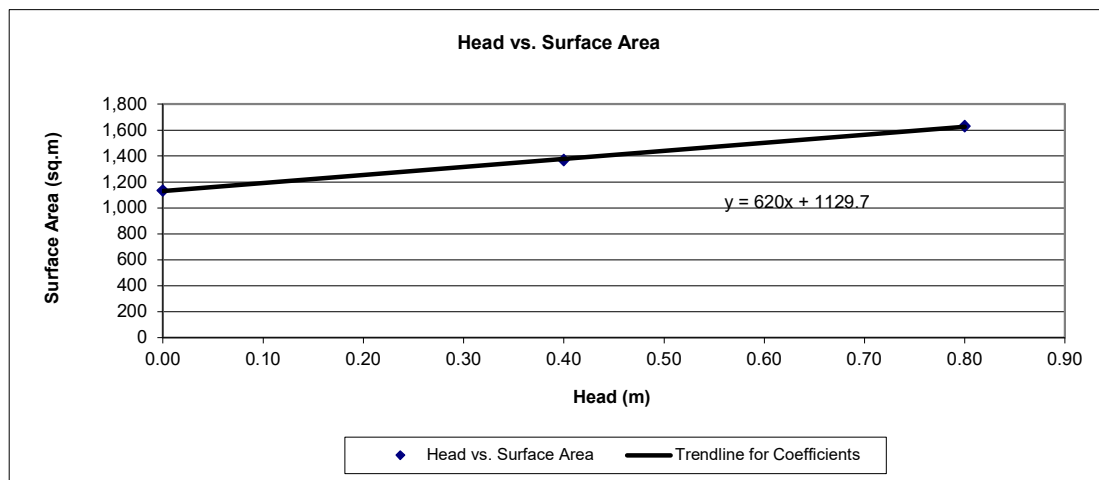
therefore, require extended detention storage of 978 m³

Extended Detention Drawdown Calculations - Dry Pond

MOE Detention Time Calculation:

Active Storage:

	Elevation (m)	Head (m)	Surface Area (m ²)
NWL	246.60	0.00	1,134
	247.00	0.40	1,369
Ext. Det.	247.40	0.80	1,630



$D_o = 50$ mm

$A_o = 0.0020$ m²

$C_1 = 620.00$

$C_2 = 1,129.7$

$t = ((0.66 * C_1 * h^{1.5}) + (2 * C_2 * h^{0.5})) / (2.75 A_o)$

$t = ((0.66 * (620) * h^{1.5}) + (2 * (1129.7) * h^{0.5})) / (2.75 A_o)$

$h = 0.80$ m

$t = 420071.7$ s

Ext. Det. = **116.7 hr**

Stone Sizing Calculations - Pond Inlet

Actual Shear Stress Based on Outflow Velocity

Input:

Flow Area	=	0.27	m ²	(From FlowMaster model output)
Wetted Perimeter	=	1.33		(From FlowMaster model output)
Hydraulic Radius, R	=	0.20	m	Flow Area / Wetted Perimeter
Q _{5-year}	=	0.54	m ³ /s	(From storm sewer design sheet)
Velocity	=	2.04	m/s	(From FlowMaster model output)

Output:

1/R	=	4.93		
Tractive force, T	=	5.1	kg/m ²	(From MTO Design Chart 2.24)

Critical Shear Stress Resistance of Stone

Input:

Stone Size, D ₅₀	=	100	mm
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Output:

Critical Shear Stress Resistance, T _{cb}	=	0.0642 x D50	(MTO, Equation 5.31)
	=	6.42	kg/m ²

Conclusion

Since the critical shear stress capacity of the stone is greater than the tractive force of the flow, the proposed stone size is appropriate.

Stone Sizing Calculations - Pond Inlet Spillway

Actual Shear Stress Based on Outflow Velocity

Input:

Flow Area	=	0.29	m ²	(From FlowMaster model output)
Wetted Perimeter	=	5.12		(From FlowMaster model output)
Hydraulic Radius, R	=	0.06	m	
Q _{regional}	=	0.66	m ³ /s	(From VO model output)
Velocity	=	2.25	m/s	(From FlowMaster model output)

Output:

1/R	=	17.66		
Tractive force, T	=	9.0	kg/m ²	(From MTO Design Chart 2.24)

Critical Shear Stress Resistance of Stone

Input:

Stone Size, D ₅₀	=	150	mm
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Output:

Critical Shear Stress Resistance, T _{cb}	=	0.0642 x D50	(MTO, Equation 5.31)
	=	9.63	kg/m ²

Conclusion

Since the critical shear stress capacity of the stone is greater than the tractive force of the flow, the proposed stone size is appropriate.

Stone Sizing Calculations - Pond Emergency Spillway

Actual Shear Stress Based on Outflow Velocity

Input:

Flow Area	=	1.2	m ²	(From FlowMaster model output)
Wetted Perimeter	=	5.48		(From FlowMaster model output)
Hydraulic Radius, R	=	0.22	m	
Q _{regulatory}	=	0.90	m ³ /s	(From VO model output)
Velocity	=	0.75	m/s	(From FlowMaster model output)

Output:

1/R	=	4.57		
Tractive force, T	=	0.9	kg/m ²	(From MTO Design Chart 2.24)

Critical Shear Stress Resistance of Stone

Input:

Stone Size, D ₅₀	=	100	mm
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Output:

Critical Shear Stress Resistance, T _{cb}	=	0.0642 x D50	(MTO, Equation 5.31)
	=	6.42	kg/m ²

Conclusion

Since the critical shear stress capacity of the stone is greater than the tractive force of the flow, the proposed stone size is appropriate.

Stone Sizing Calculations - Pond Outlet

Actual Shear Stress Based on Outflow Velocity

Input:

Pipe Diameter	=	600	mm	
Pipe Slope	=	0.010	m/m	
Hydraulic Radius, R	=	0.15	m	(From FlowMaster model output)
Q_{regional}	=	0.327	m ³ /s	(Allowable release rate)
Velocity	=	2.21	m/s	(From FlowMaster model output)

Output:

$1/R$	=	<u>6.67</u>		
Tractive force, T	=	<u>7.1</u>	kg/m ²	(From MTO Design Chart 2.24)

Critical Shear Stress Resistance of Stone

Input:

Stone Size, D ₅₀	=	<u>150</u>	mm
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Output:

Critical Shear Stress Resistance, T _{cb}	=	0.0642 x D50		(MTO, Equation 5.31)
	=	<u>9.63</u>	kg/m ²	

Conclusion

Since the critical shear stress capacity of the stone is greater than the tractive force of the flow, the proposed stone size is appropriate.

Worksheet for Pond Inlet

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.00600	m/m
Diameter	0.675	m
Discharge	0.543	m³/s

Results

Normal Depth	0.47	m
Flow Area	0.27	m²
Wetted Perimeter	1.33	m
Hydraulic Radius	0.20	m
Top Width	0.62	m
Critical Depth	0.47	m
Percent Full	69.8	%
Critical Slope	0.00607	m/m
Velocity	2.04	m/s
Velocity Head	0.21	m
Specific Energy	0.68	m
Froude Number	0.99	
Maximum Discharge	0.70	m³/s
Discharge Full	0.65	m³/s
Slope Full	0.00417	m/m
Flow Type	SubCritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	69.78	%
Downstream Velocity	Infinity	m/s

Worksheet for Pond Inlet

GVF Output Data

Upstream Velocity	Infinity	m/s
Normal Depth	0.47	m
Critical Depth	0.47	m
Channel Slope	0.00600	m/m
Critical Slope	0.00607	m/m

Worksheet for Inlet Spillway

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.033	
Channel Slope	4.00000	m/m (H:V)
Bottom Width	5.00	m
Discharge	0.662	m³/s

Results

Normal Depth	0.06	m
Flow Area	0.29	m²
Wetted Perimeter	5.12	m
Hydraulic Radius	0.06	m
Top Width	5.00	m
Critical Depth	0.12	m
Critical Slope	0.02298	m/m
Velocity	2.25	m/s
Velocity Head	0.26	m
Specific Energy	0.32	m
Froude Number	2.97	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.06	m
Critical Depth	0.12	m
Channel Slope	4.00000	m/m (H:V)
Critical Slope	0.02298	m/m

Worksheet for Emergency Spillway

Project Description

Solve For Headwater Elevation

Input Data

Discharge		0.902	m³/s
Crest Elevation		249.70	m
Tailwater Elevation		0.00	m
Crest Surface Type	Gravel		
Crest Breadth		3.00	m
Crest Length		5.00	m

Results

Headwater Elevation	249.94	m
Headwater Height Above Crest	0.24	m
Tailwater Height Above Crest	-249.70	m
Weir Coefficient	1.53	SI
Submergence Factor	1.00	
Adjusted Weir Coefficient	1.53	SI
Flow Area	1.20	m²
Velocity	0.75	m/s
Wetted Perimeter	5.48	m
Top Width	5.00	m

Worksheet for Pond Outlet

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.013	
Channel Slope	0.01000	m/m
Diameter	0.60	m
Discharge	0.327	m³/s

Results

Normal Depth	0.31	m
Flow Area	0.15	m²
Wetted Perimeter	0.97	m
Hydraulic Radius	0.15	m
Top Width	0.60	m
Critical Depth	0.37	m
Percent Full	51.9	%
Critical Slope	0.00562	m/m
Velocity	2.21	m/s
Velocity Head	0.25	m
Specific Energy	0.56	m
Froude Number	1.42	
Maximum Discharge	0.66	m³/s
Discharge Full	0.61	m³/s
Slope Full	0.00284	m/m
Flow Type	SuperCritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Average End Depth Over Rise	0.00	%
Normal Depth Over Rise	51.91	%
Downstream Velocity	Infinity	m/s

Worksheet for Pond Outlet

GVF Output Data

Upstream Velocity	Infinity	m/s
Normal Depth	0.31	m
Critical Depth	0.37	m
Channel Slope	0.01000	m/m
Critical Slope	0.00562	m/m

BROCCOLLINI SITEPLAN

Town Of Caledon, Region of Peel

PROJECT NUMBER - 2278-7228 - PHASE 1

5-YEAR STORM SEWER DESIGN SHEET



FREQUENCY - 5 YEAR - TOWN OF CALEDON				
Coef. A=	1593	Coef. B=	11	Coef. C= 0.8789

MATERIAL	MANNINGS "n"
PVC	0.013
CONC.	0.013
CSP	0.013

DESIGNED BY:	VM
CHECKED BY:	JS
DATE:	4/29/2026
ISSUED FOR:	SPA

INITIAL TIME OF CONCENTRATION (minutes) = 10.00																						
CATCHMENT I.D.	FROM MH	TO MH	AREA (A)	5 YEAR RUN-OFF COEFF	DESIGN STORM	A x C	5 YEAR CUMUL.	TIME OF CONC.	5 YEAR I	CONTROLLED ROOF FLOW	CONTROLLED FLOW CUMUL.	Q (RUNOFF)	DESIGN FLOW	SLOPE	PIPE DIA.	MANNING'S "n"	VELOCITY	LENGTH	TIME OF FLOW	PIPE CAPACITY	CAPACITY	
			(Ha)	(C _s)			A x C _s	(min.)	(mm/hr)	(L/sec)	(L/sec)	(L/sec)	(L/sec)	(%)	(mm)		(m/sec)	(m)	(min)	(L/sec)	(%)	
BUILDING A	C101	DCBMH101	DCBMH102	0.21	0.76	5 year	0.16	0.16	10.00		0.00	48.66	48.66	0.50%	300	0.013	1.0	55.2	0.95	68.38	71%	
		PLUG102	DCBMH102	0.00	0.00	5 year	0.00	0.00	10.00	24.25	24.25	0.00	24.25	2.00%	200	0.013	1.5	11.1	0.13	46.38	52%	
	C102	DCBMH102	DCBMH103	0.15	0.82	5 year	0.12	0.28	10.95		24.25	82.88	107.13	0.50%	375	0.013	1.1	60.5	0.90	123.98	86%	
		PLUG101	DCBMH103	0.00	0.00	5 year	0.00	0.00	10.00	24.25	24.25	0.00	24.25	2.00%	200	0.013	1.5	11.1	0.13	46.38	52%	
	C103	DCBMH103	MH105	0.14	0.85	5 year	0.12	0.40	11.85		48.50	113.69	162.19	0.50%	450	0.013	1.3	42.7	0.56	201.60	80%	
		MH105	CBMH101	0.00	0.00	5 year	0.00	0.40	12.41		48.50	111.29	159.79	0.50%	450	0.013	1.3	19.0	0.25	201.60	79%	
	C104	CBMH101	CBMH102	0.05	0.64	5 year	0.03	0.43	12.66		48.50	119.05	167.55	0.50%	450	0.013	1.3	38.8	0.51	201.60	83%	
	C105	CBMH102	MH106	0.03	0.76	5 year	0.02	0.46	13.17		48.50	122.98	171.48	0.50%	450	0.013	1.3	18.5	0.24	201.60	85%	
		MH106	DCBMH104	0.00	0.00	5 year	0.00	0.46	13.41		48.50	121.90	170.40	0.50%	450	0.013	1.3	40.3	0.53	201.60	85%	
	C106	DCBMH104	MH107	0.18	0.90	5 year	0.16	0.62	13.94		48.50	162.08	210.58	0.40%	525	0.013	1.3	43.1	0.57	272.00	77%	
	C107	CB101	MH101	0.15	0.59	5 year	0.09	0.09	10.00		0.00	26.98	26.98	1.00%	250	0.013	1.2	25.0	0.34	59.47	45%	
	C108	CB102	MH101	0.07	0.52	5 year	0.04	0.04	10.00		0.00	11.10	11.10	2.00%	250	0.013	1.7	1.5	0.01	84.10	13%	
		MH101	MH102	0.00	0.00	5 year	0.00	0.12	10.34		0.00	37.54	37.54	0.50%	300	0.013	1.0	67.9	1.17	68.38	55%	
	C109	DCB101	MH102	0.31	0.67	5 year	0.21	0.21	10.00		0.00	63.33	63.33	2.00%	300	0.013	1.9	2.9	0.02	136.76	46%	
	C110	DCB102	MH102	0.20	0.90	5 year	0.18	0.18	10.00		0.00	54.88	54.88	2.00%	300	0.013	1.9	4.0	0.03	136.76	40%	
		MH102	MH103	0.00	0.00	5 year	0.00	0.51	11.51		0.00	147.02	147.02	0.40%	450	0.013	1.1	83.0	1.22	180.32	82%	
		PLUG103	MH103R	0.00	0.00	5 year	0.00	0.00	10.00		24.25	24.25	0.00	24.25	2.00%	200	0.013	1.5	20.8	0.23	46.38	52%
		MH103R	MH103	0.00	0.00	5 year	0.00	0.00	10.23			24.25	0.00	24.25	0.50%	250	0.013	0.9	10.7	0.21	42.05	58%
	C111	DCB103	MH103	0.25	0.90	5 year	0.23	0.23	10.00		0.00	68.60	68.60	2.00%	300	0.013	1.9	3.9	0.03	136.76	50%	
		MH103	MH104	0.00	0.00	5 year	0.00	0.74	12.73		24.25	201.96	226.21	0.40%	525	0.013	1.3	90.0	1.19	272.00	83%	
		PLUG104	MH104R	0.00	0.00	5 year	0.00	0.00	10.00		24.25	24.25	0.00	24.25	1.00%	200	0.013	1.0	20.8	0.33	32.80	74%
		MH104R	MH104	0.00	0.00	5 year	0.00	0.00	10.33		24.25	24.25	0.00	24.25	6.50%	250	0.013	3.1	10.4	0.06	151.61	16%
	C112	DCB104	MH104	0.25	0.90	5 year	0.23	0.23	10.00		0.00	68.60	68.60	2.00%	300	0.013	1.9	3.4	0.03	136.76	50%	
	C113	DCB105	MH104	0.21	0.90	5 year	0.19	0.19	10.00		0.00	57.63	57.63	2.00%	300	0.013	1.9	2.1	0.02	136.76	42%	
		MH104	MH107	0.00	0.00	5 year	0.00	1.15	13.93		48.50	302.01	350.51	0.45%	600	0.013	1.5	90.0	1.03	411.89	85%	
		MH107	DIVERSION MH100	0.00	0.00	5 year	0.00	1.77	14.96		97.00	447.96	544.96	0.60%	675	0.013	1.8	11.3	0.10	651.12	84%	
		DIVERSION MH100	RETURN MH100	0.00	0.00	5 year	0.00	1.77	15.06		97.00	446.40	543.40	1.00%	675	0.013	2.3	3.5	0.02	840.59	65%	
		RETURN MH100	HW101	0.00	0.00	5 year	0.00	1.77	15.09		97.00	446.02	543.02	0.60%	675	0.013	1.8	33.7	0.31	651.12	83%	

Project: Broccolini Airport Rd
Project No.: 2278-7228

Design: K.W.
Check: J.S.
Date: 2026-04-29

Curb Cut Sizing

Required Curb Cut Length = Regional Flow Rate / Weir Coefficient*(Head^{1.5})

- Where the Weir Coefficient is equal to 1.55
- Where the Head is equal to 0.30m

Curb Cut #	Q (Regional) (m ³ /s)	Minimum Curb Cut Length (m)
Curb Cut 1	0.662	2.6



STANDARD OFFLINE Jellyfish Filter Sizing Report

Project Information

Date	Thursday, April 30, 2026
Project Name	Airport Rd. - North Parcel
Project Number	2278-7228
Location	Caledon

Jellyfish Filter Design Overview

This report provides information for the sizing and specification of the Jellyfish Filter. When designed properly in accordance to the guidelines detailed in the Jellyfish Filter Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters.

Please see www.ImbriumSystems.com for more information.

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF12-24-5 is recommended to meet the water quality objective by treating a flow of 113.7 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 1508 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF12-24-5	24	5	3.6	113.7	1508

The Jellyfish Filter System

The patented Jellyfish Filter is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact stand-alone treatment system that removes a high level and wide variety of stormwater pollutants. Exceptional pollutant removal is achieved at high treatment flow rates with minimal head loss and low maintenance costs. Each lightweight Jellyfish Filter cartridge contains an extraordinarily large amount of membrane surface area, resulting in superior flow capacity and pollutant removal capacity.

Maintenance

Regular scheduled inspections and maintenance is necessary to assure proper functioning of the Jellyfish Filter. The maintenance interval is designed to be a minimum of 12 months, but this will vary depending on site loading conditions and upstream pretreatment measures. Quarterly inspections and inspections after all storms beyond the 5-year event are recommended until enough historical performance data has been logged to comfortably initiate an alternative inspection interval.

Please see www.ImbriumSystems.com for more information.

Thank you for the opportunity to present this information to you and your client.

Performance

Jellyfish efficiently captures a high level of Stormwater pollutants, including:

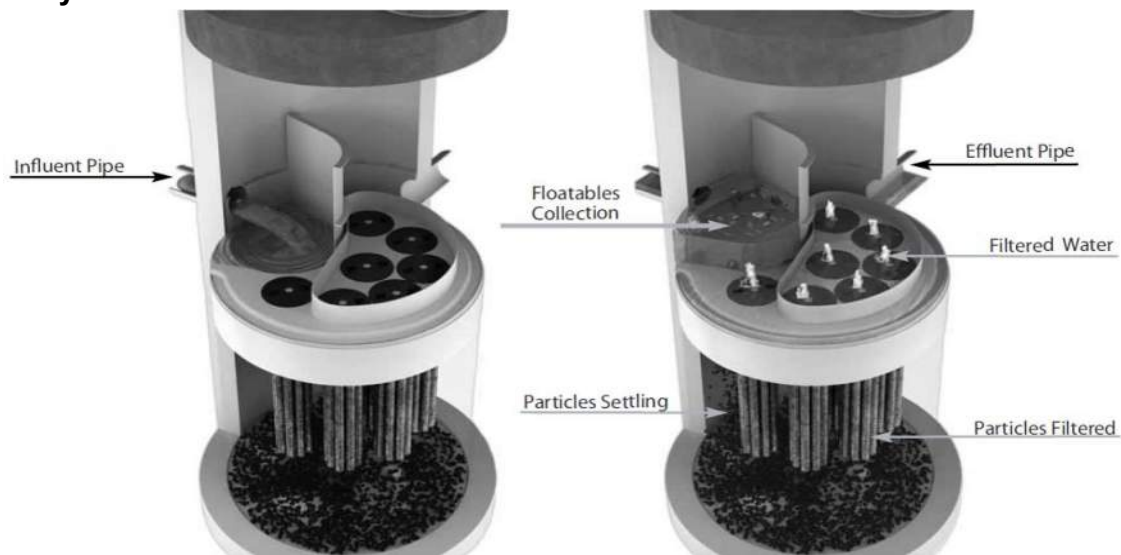
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 77% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Performance

The Jellyfish filter has been field-tested on an urban site with 25 TAPE qualifying rain events and field monitored according to the TAPE field test protocol, demonstrating:

- A median TSS removal efficiency of 90%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitored storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 77%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Project Information

Date:	Thursday, April 30, 2026
Project Name:	Airport Rd. - North Parcel
Project Number:	2278-7228
Location:	Caledon

Designer Information

Company:	C.F. Crozier & Associates Inc.
Contact:	Isabelle Forsyth
Phone #:	(416) 842-0023

Notes

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Rainfall

Name:	TORONTO CENTRAL
State:	ON
ID:	100
Record:	1982 to 1999
Co-ords:	45°30'N, 90°30'W

Drainage Area

Total Area:	4.59 ha
Runoff Coefficient:	0.83

Upstream Detention

Peak Release Rate:	n/a
Pretreatment Credit:	n/a

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 18 years of TORONTO CENTRAL rainfall data:	96 L/s
Sediment Loading	Treating 90% of the average annual runoff volume, 24335 m³, with a suspended sediment concentration of 60 mg/L.	1460 kg*

* Indicates that sediment loading is the limiting parameter in the sizing of this Jellyfish system

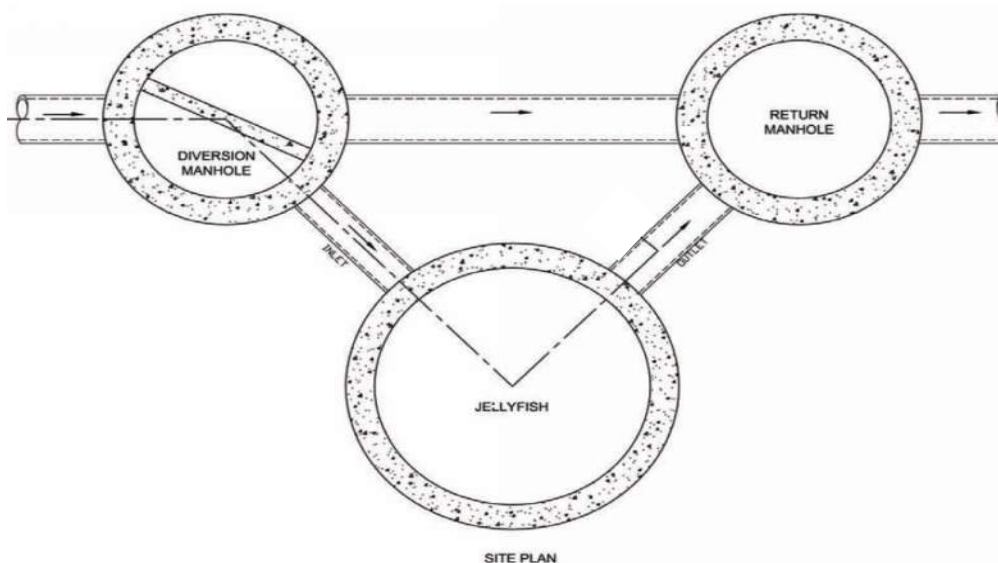
Recommendation

The Jellyfish Filter model JF12-24-5 is recommended to meet the water quality objective by treating a flow of 113.7 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 1508 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Wet Vol Below Deck (L)	Sump Storage (m³)	Oil Capacity (L)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1	1	1	1.2	2313	0.34	379	7.6	85
JF4-2-1	2	1	1.2	2313	0.34	379	12.6	142
JF6-3-1	3	1	1.8	5205	0.79	848	17.7	199
JF6-4-1	4	1	1.8	5205	0.79	848	22.7	256
JF6-5-1	5	1	1.8	5205	0.79	848	27.8	313
JF6-6-1	6	1	1.8	5205	0.79	848	28.6	370
JF8-6-2	6	2	2.4	9252	1.42	1469	35.3	398
JF8-7-2	7	2	2.4	9252	1.42	1469	40.4	455
JF8-8-2	8	2	2.4	9252	1.42	1469	45.4	512
JF8-9-2	9	2	2.4	9252	1.42	1469	50.5	569
JF8-10-2	10	2	2.4	9252	1.42	1469	50.5	626
JF10-11-3	11	3	3.0	14456	2.21	2302	63.1	711
JF10-12-3	12	3	3.0	14456	2.21	2302	68.2	768
JF10-12-4	12	4	3.0	14456	2.21	2302	70.7	796
JF10-13-4	13	4	3.0	14456	2.21	2302	75.7	853
JF10-14-4	14	4	3.0	14456	2.21	2302	78.9	910
JF10-15-4	15	4	3.0	14456	2.21	2302	78.9	967
JF10-16-4	16	4	3.0	14456	2.21	2302	78.9	1024
JF10-17-4	17	4	3.0	14456	2.21	2302	78.9	1081
JF10-18-4	18	4	3.0	14456	2.21	2302	78.9	1138
JF10-19-4	19	4	3.0	14456	2.21	2302	78.9	1195
JF12-20-5	20	5	3.6	20820	3.2	2771	113.6	1280
JF12-21-5	21	5	3.6	20820	3.2	2771	113.7	1337
JF12-22-5	22	5	3.6	20820	3.2	2771	113.7	1394
JF12-23-5	23	5	3.6	20820	3.2	2771	113.7	1451
JF12-24-5	24	5	3.6	20820	3.2	2771	113.7	1508
JF12-25-5	25	5	3.6	20820	3.2	2771	113.7	1565
JF12-26-5	26	5	3.6	20820	3.2	2771	113.7	1622
JF12-27-5	27	5	3.6	20820	3.2	2771	113.7	1679

Jellyfish Filter Design Notes

- Typically the Jellyfish Filter is designed in an offline configuration, as all stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations. Depending on the design parameters, an optional internal bypass may be incorporated into the Jellyfish Filter, however note the inspection and maintenance frequency should be expected to increase above that of an off-line system. Speak to your local representative for more information.



Jellyfish Filter Typical Layout

- Typically, 18 inches (457 mm) of driving head is designed into the system, calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. Alternative driving head values can be designed as 12 to 24 inches (305 to 610mm) depending on specific site requirements, requiring additional sizing and design assistance.
- Typically, the Jellyfish Filter is designed with the inlet pipe configured 6 inches (150 mm) above the outlet invert elevation. However, depending on site parameters this can vary to an optional configuration of the inlet pipe entering the unit below the outlet invert elevation.
- The Jellyfish Filter can accommodate multiple inlet pipes within certain restrictions.
- While the optional inlet below deck configuration offers 0 to 360 degree flexibility between the inlet and outlet pipe, typical systems conform to the following:

Model Diameter (m)	Minimum Angle Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (mm)	Minimum Outlet Pipe Diameter (mm)
1.2	62°	150	200
1.8	59°	200	250
2.4	52°	250	300
3.0	48°	300	450
3.6	40°	300	450

- The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems. For sites that require minimal depth of cover for the stormwater infrastructure, the Jellyfish Filter can be applied in a shallow application using a hatch cover. The general minimum depth of cover is 36 inches (915 mm) from top of the underslab to outlet invert.
- If driving head calculations account for water elevation during submerged conditions the Jellyfish Filter will function effectively under submerged conditions.
- Jellyfish Filter systems may incorporate grated inlets depending on system configuration.
- For sites with water quality treatment flow rates or mass loadings that exceed the design flow rate of the largest standard Jellyfish Filter manhole models, systems can be designed that hydraulically connect multiple Jellyfish Filters in series or alternatively Jellyfish Vault units can be designed.

STANDARD SPECIFICATION STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

Specifies requirements for construction and performance of an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS

ASTM C 891: Specification for Installation of Underground Precast Concrete Utility Structures
ASTM C 478: Specification for Precast Reinforced Concrete Manhole Sections
ASTM C 443: Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets
ASTM D 4101: Specification for Copolymer steps construction

CAN/CSA-A257.4-M92

Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92

Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

Shop drawings for the structure and performance are to be submitted with each order to the contractor. Contractor shall forward shop drawing submittal to the consulting engineer for approval. Shop drawings are to detail the structure's precast concrete and call out or note the fiberglass (FRP) internals/components.

1.4 PRODUCT SUBSTITUTIONS

No product substitutions shall be accepted unless submitted 10 days prior to project bid date, or as directed by the engineer of record. Submissions for substitutions require review and approval by the Engineer of Record, for hydraulic performance, impact to project designs, equivalent treatment performance, and any required project plan and report (hydrology/hydraulic, water quality, stormwater pollution) modifications that would be required by the approving jurisdictions/agencies. Contractor to coordinate with the Engineer of Record any applicable modifications to the project estimates of cost, bonding amount determinations, plan check fees for changes to approved documents, and/or any other regulatory requirements resulting from the product substitution.

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

- 2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.
- 2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.
- 2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft² (0.142 lps/m²).

Each membrane filter cartridge shall allow for manual installation and removal. Each filter cartridge shall have filtration membrane surface area and dry installation weight as follows (if length of filter cartridge is between those listed below, the surface area and weight shall be proportionate to the next length shorter and next length longer as shown below):

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft ² / m ²)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

- 2.1.4 Backwashing Cartridges The filter device shall have a weir extending above the cartridge deck, or other mechanism, that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir, or other mechanism, shall collect a pool of filtered water during inflow events that backwashes the high flow rate cartridges when the inflow

event subsides. All filter cartridges and membranes shall be reusable and allow for the use of filtration membrane rinsing procedures to restore flow capacity and sediment capacity; extending cartridge service life.

- 2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.
- 2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.
- 2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.
- 2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.
- 2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be manufactured to a minimum live load of HS-20 truck loading or greater based on local regulatory specifications, unless otherwise modified or specified by the design engineer, and shall be watertight.

2.3 JOINTS All precast concrete manhole configuration joints shall use nitrile rubber gaskets and shall meet the requirements of ASTM C443, Specification C1619, Class D or engineer approved equal to ensure oil resistance. Mastic sealants or butyl tape are not an acceptable alternative.

2.4 GASKETS Only profile neoprene or nitrile rubber gaskets in accordance to CSA A257.3-M92 will be accepted. Mastic sealants, butyl tape or Con Seal CS-101 are not acceptable gasket materials.

2.5 FRAME AND COVER Frame and covers must be manufactured from cast-iron or other composite material tested to withstand H-20 or greater design loads, and as approved by the

local regulatory body. Frames and covers must be embossed with the name of the device manufacturer or the device brand name.

- 2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.
- 2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.
- 2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.
- 2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.
- 2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

- 3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).
- 3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.
- 3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.
- 3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.
- 3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

At a minimum, the stormwater quality filter device shall have been field tested and verified with a minimum 25 TARP qualifying storm events and field monitoring shall have been conducted according to the TARP 2009 NJDEP TARP field test protocol, and have received NJCAT verification.

- 3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.
- 3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.
- 3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.
- 3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.
- 3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.
- 3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

- 3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.
- 3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).
- 3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

- 3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.
- 3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.
- 3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.
- 3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 INSTALLATION

4.1.1 PRECAST DEVICE CONSTRUCTION SEQUENCE

The installation of a watertight precast concrete device should conform to ASTM C 891 and to any state highway, municipal or local specifications for the construction of manholes, whichever is more stringent. Selected sections of a general specification that are applicable are summarized below.

4.1.1.1 The watertight precast concrete device is installed in sections in the following sequence:

- aggregate base
- base slab
- treatment chamber and cartridge deck riser section(s)
- bypass section
- connect inlet and outlet pipes
- concrete riser section(s) and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

4.1.2 The precast base should be placed level at the specified grade. The entire base should be in contact with the underlying compacted granular material. Subsequent sections, complete with joint seals, should be installed in accordance with the precast concrete manufacturer's recommendations.

4.1.3 Adjustment of the stormwater quality treatment device can be performed by lifting the upper sections free of the excavated area, re-leveling the base, and re-installing the sections. Damaged sections and gaskets should be repaired or replaced as necessary to restore original condition and watertight seals. Once the stormwater quality treatment device has been constructed, any/all lift holes must be plugged watertight with mortar or non-shrink grout.

- 4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.
- 4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

4.2 MAINTENANCE ACCESS WALL

In some instances the Maintenance Access Wall, if provided, shall require an extension attachment and sealing to the precast wall and cartridge deck at the job site, rather than at the precast facility. In this instance, installation of these components shall be performed according to instructions provided by the manufacturer.

4.3 FILTER CARTRIDGE INSTALLATION Filter cartridges shall be installed in the cartridge deck only after the construction site is fully stabilized and in accordance with the manufacturer's guidelines and recommendations. Contractor to contact the manufacturer to schedule cartridge delivery and review procedures/requirements to be completed to the device prior to installation of the cartridges and activation of the system.

PART 5 – QUALITY ASSURANCE

5.1 FILTER CARTRIDGE INSTALLATION Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed complete after site is stabilized and unit is ready to accept cartridges. Unit is ready to accept cartridges after it has been cleaned out and any standing water, debris, and other materials have been removed. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction, and in accordance with the manufacturer's recommendations and guidance. For systems with cartridges installed prior to full site stabilization and prior to system activation, the contractor can plug inlet and outlet pipes to prevent stormwater and other influent from entering the device. Plugs must be removed during the activation process.

5.2 INSPECTION AND MAINTENANCE

5.2.1 The manufacturer shall provide an Owner's Manual upon request.

5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

5.3 REPLACEMENT FILTER CARTRIDGES When replacement membrane filter elements and/or other parts are required, only membrane filter elements and parts approved by the manufacturer for use with the stormwater quality filter device shall be installed.

END OF SECTION

STANDARD PERFORMANCE SPECIFICATION STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental Management – Environmental Technology Verification (ETV)

1.3 SUBMITTALS

- 1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.
- 1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: filtration surface area, treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.
- 1.3.3 Unless directed otherwise by the Engineer of Record, filtration treatment device product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

PART 2 – PRODUCTS

2.1 GENERAL

- 2.1.1 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the internal components. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of their installed placement for the entire length of the cartridge.
- 2.1.2 Pollutant Storage: The Filter device shall include a sump for sediment storage, and a protected volume for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants.

PART 3 – PERFORMANCE

3.1 GENERAL

- 3.1.1 Verification – The stormwater quality filter treatment device shall have been field tested in accordance with either TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements – Amendments to TARP Tier II Protocol (NJDEP, 2009) or Washington State Technology Assessment Protocol – Ecology (TAPE), 2011 or later version. The field test shall have been verified in accordance with ISO 14034:2016 Environmental Management – Environmental Technology Verification (ETV). See Section 3.2 of this specification for field test performance requirements.

3.2 FIELD TEST PERFORMANCE

The field test (as specified in section 3.1.1) shall have monitored a minimum of twenty (20) TARP or TAPE qualifying storm events, and report at **minimum** the following results:

- 3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have ISO 14034 ETV verified load based median TSS removal efficiency of at least 85% and load based median SSC removal efficiency of at least 98%.
- 3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.
- 3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, and an effluent d_{50} of 15 microns or lower for all monitored storm events.
- 3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce turbidity such that effluent turbidity is 15 NTU or lower.
- 3.2.5 Nutrients & Metals – The stormwater quality filter treatment device shall have ISO 14034 ETV Verified minimum load based removal efficiencies for the following:
- 3.2.5.1 Total Phosphorus (TP) Removal - Median TP removal efficiency of at least 49%.
- 3.2.5.2 Total Nitrogen (TN) Removal - Median TN removal efficiency of at least 39%.
- 3.2.5.3 Total Zinc (Zn) Removal - Median Zn removal efficiency of at least 69%.
- 3.2.5.4 Total Copper (Cu) Removal - Median Cu removal efficiency of at least 91%.

END OF SECTION

Infiltration Gallery

Infiltration Rate

12.70 mm/hr

Infiltration rate at the proposed bottom elevation of the infiltration gallery must be divided by a safety correction factor:

Safety Factor = 1.5

Design infiltration Rate determined by dividing mean infiltration rate at bottom of infiltration trench by the safety correction factor:

Design Infiltration Rate = 8.5 mm/hr

$$d_{r\max} = i * t_s / V_r$$

Where:

- $d_{r\max}$ = Maximum stone reservoir depth (mm)
- i = Infiltration rate for native soils (mm/hr)
- V_r = Void space ratio for aggregate used (typically 0.4 for 50 mm clear stone)
- t_s = Time to drain (design for 48 hour time to drain is recommended)

equation from TRCA LID Manual (2010) page 4-58

Size Infiltration Gallery Area

Required Volume in Trench (m ³)	Design Depth of Tank (m)	Void Ratio	Surface Area (m ²)	Volume Provided (m ³)	Drawdown Time (hours)
203	0.65	0.40	800	208	31

APPENDIX D

Erosion Control Calculations



PROJECT: Broccolini
PROJECT NO.: 2278-7228

CREATED BY: LE
CHECKED BY: JS

DATE: 2025.10.17
UPDATED: 2026.04.29

Temporary Settlement Basin - Required Volume Specifications

Specifications

Active Storage Design Requirement:	125	m ³ /ha
Permanent Pool Design Requirement:	185	m ³ /ha
Drawdown Time:	48	hrs
Minimum Depth of Basin:	1	m
Maximum Depth of Basin:	5	m
Length to Width Ratio	4:1	L:W
Maximum Internal Side Slopes	4:1	H:V
Maximum External Side Slopes	2:1	H:V

Required Volume Calculations

Temporary Drainage Area:	13.44	ha
Permanent Pool Volume:	2,486	m ³
Active Storage Volume:	1,680	m ³
Total:	4,166	m ³

References/Notes

TRCA Erosion and Sediment Control Guide -
Appendix B



PROJECT: Broccolini
PROJECT NO.: 2278-7228

CREATED BY: LE
CHECKED BY: JS

DATE: 2025.10.17
UPDATED: 2026.04.29

Temporary Settlement Basin Sizing

Proposed Basin Dimensions

Depth

Freeboard: 0.30 m
Active Storage: 0.80 m
Permanent Pool: 2.20 m

Total Depth of Basin: 3.30 m

Side Slope

Freeboard: 4 :1
Active Storage: 4 :1
Permanent Pool: 4 :1

Basin Areas Measured in AutoCAD:

Total Basin Surface Area: 2,996 m²
0.30 ha

Total Active Storage Area: 2,707 m²
Total Permanent Pool Area: 1,979 m²

Basin Volume Calculation

	<u>Provided</u>		<u>Required</u>	
Freeboard Volume:	855	m ³	-	m ³
Active Storage Volume:	1,874	m ³	1,680	m ³
Permanent Pool Volume:	2,534	m ³	2,486	m ³
Total Basin Volume:	5,264	m³	4,166	m³



PROJECT: Broccolini
PROJECT NO.: 2278-7228

CREATED BY: LE
CHECKED BY: JS

DATE: 2025.10.17
UPDATED: 2026.04.29

Hickenbottom Drawdown Time - ESC Pond

Calculations

REQUIRED Pond Active Storage Volume

Depth for Active Storage	1,680	m ³
Target Drawdown Time	0.80	m
	48	hrs

Hickenbottom Specifications

0.127m intake with 24 - 25mm holes per 0.305m

Hickenbottom Capacity 25.24 L/s

Note: Hickenbottom specifications and capacity based on Hickenbottom Inc. Product Specifications
<http://www.hickenbottominc.com/index.html>

Hickenbottom Drawdown Time	66559	s
Hickenbottom Drawdown Time	18	hrs

PROVIDED Pond Active Storage Volume

Depth for Active Storage	1,874	m ³
Target Drawdown Time	0.80	m
	48	hrs

Hickenbottom Specifications

0.127m intake with 24 - 25mm holes per 0.305m

Hickenbottom Capacity 25.24 L/s

Note: Hickenbottom specifications and capacity based on Hickenbottom Inc. Product Specifications
<http://www.hickenbottominc.com/index.html>

Hickenbottom Drawdown Time	74261	s
Hickenbottom Drawdown Time	21	hrs

PROJECT#: 2278-7228
PROJECT: Broccolini
ESC SWALE SIZING

RATIONAL METHOD - 100 YEAR

Computed 100-year peak flow to interceptor swale

Outlet ID	Drainage Area (ha)	Runoff Coefficient	Time t_c (min)	Intensity $i=a/(t_c+b)$ (mm/hr)	Flow $Q=CiA/360$ (m ³ /s)
Swale A	6.72	0.50	10	167.1	1.56
Swale B	6.72	0.50	10	167.1	1.56

Where: a, b, c = rainfall equation coefficients

a = 4688

b = 17

c = 0.9624



100-Year Storm
IDF Parameters as per
Town of Caledon

Swale A

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.40	%
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	2.50	m
Discharge	1.56	m³/s

Results

Normal Depth	0.46	m
Flow Area	1.80	m²
Wetted Perimeter	5.43	m
Hydraulic Radius	0.33	m
Top Width	5.28	m
Critical Depth	0.30	m
Critical Slope	0.01999	m/m
Velocity	0.87	m/s
Velocity Head	0.04	m
Specific Energy	0.50	m
Froude Number	0.47	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.46	m
Critical Depth	0.30	m
Channel Slope	0.40	%

Swale A

GVF Output Data

Critical Slope	0.01999	m/m
----------------	---------	-----

Swale B

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.80	%
Left Side Slope	3.00	m/m (H:V)
Right Side Slope	3.00	m/m (H:V)
Bottom Width	2.50	m
Discharge	1.56	m³/s

Results

Normal Depth	0.39	m
Flow Area	1.41	m²
Wetted Perimeter	4.94	m
Hydraulic Radius	0.29	m
Top Width	4.81	m
Critical Depth	0.30	m
Critical Slope	0.01999	m/m
Velocity	1.11	m/s
Velocity Head	0.06	m
Specific Energy	0.45	m
Froude Number	0.65	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.39	m
Critical Depth	0.30	m
Channel Slope	0.80	%

Swale B

GVF Output Data

Critical Slope 0.01999 m/m

Stone Sizing Calculations - ESC Sediment Basin Inlet

Actual Shear Stress Based on Outflow Velocity

Input:

Flow Area	=	0.90	m ²	(From FlowMaster model output)
Wetted Perimeter	=	5.92		(From FlowMaster model output)
Hydraulic Radius, R	=	0.15	m	
Q	=	3.12	m ³ /s	
Velocity	=	3.47	m/s	(From FlowMaster model output)

Output:

1/R	=	6.58		
Tractive force, T	=	16.0	kg/m ²	(From MTO Design Chart 2.24)

Critical Shear Stress Resistance of Stone

Input:

Stone Size, D ₅₀	=	300	mm
-----------------------------	---	-----	----

Output:

Critical Shear Stress Resistance, T _{cb}	=	0.0642 x D ₅₀	(MTO, Equation 5.31)
	=	19.26	kg/m ²

Conclusion

Since the critical shear stress capacity of the stone is greater than the tractive force of the flow, the proposed stone size is appropriate.

Worksheet for ESC Basin Inlet

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.041	
Channel Slope	4.00000	m/m (H:V)
Bottom Width	5.60	m
Discharge	3.120	m³/s

Results

Normal Depth	0.16	m
Flow Area	0.90	m²
Wetted Perimeter	5.92	m
Hydraulic Radius	0.15	m
Top Width	5.60	m
Critical Depth	0.32	m
Critical Slope	0.02791	m/m
Velocity	3.47	m/s
Velocity Head	0.61	m
Specific Energy	0.77	m
Froude Number	2.77	
Flow Type	Supercritical	

GVF Input Data

Downstream Depth	0.00	m
Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.16	m
Critical Depth	0.32	m
Channel Slope	4.00000	m/m (H:V)
Critical Slope	0.02791	m/m

Stone Sizing Calculations - ESC Sediment Basin Inlet

Actual Shear Stress Based on Outflow Velocity

Input:

Flow Area	=	0.52	m ²	(From FlowMaster model output)
Wetted Perimeter	=	4.26		(From FlowMaster model output)
Hydraulic Radius, R	=	0.12	m	
Q	=	3.12	m ³ /s	
Velocity	=	3.00	m/s	(From FlowMaster model output)

Output:

1/R	=	8.19		
Tractive force, T	=	13.0	kg/m ²	(From MTO Design Chart 2.24)

Critical Shear Stress Resistance of Stone

Input:

Stone Size, D ₅₀	=	300	mm
-----------------------------	---	-----	----

Output:

Critical Shear Stress Resistance, T _{cb}	=	0.0642 x D ₅₀	(MTO, Equation 5.31)
	=	19.26	kg/m ²

Conclusion

Since the critical shear stress capacity of the stone is greater than the tractive force of the flow, the proposed stone size is appropriate.

Worksheet for ESC Basin Inlet

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.041	
Channel Slope	4.00000	m/m (H:V)
Bottom Width	4.00	m
Discharge	1.560	m³/s

Results

Normal Depth	0.13	m
Flow Area	0.52	m²
Wetted Perimeter	4.26	m
Hydraulic Radius	0.12	m
Top Width	4.00	m
Critical Depth	0.25	m
Critical Slope	0.03063	m/m
Velocity	3.00	m/s
Velocity Head	0.46	m
Specific Energy	0.59	m
Froude Number	2.66	
Flow Type	Supercritical	

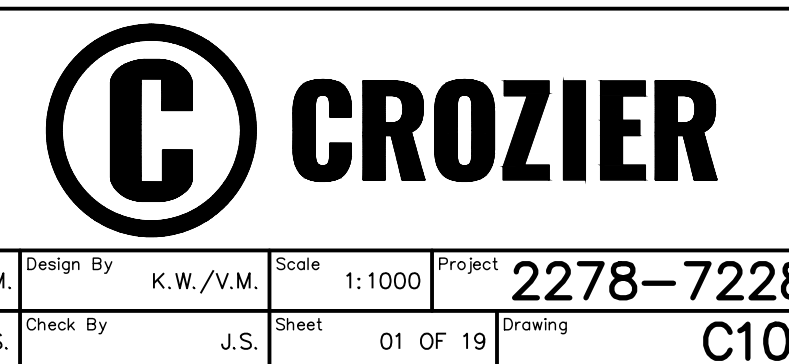
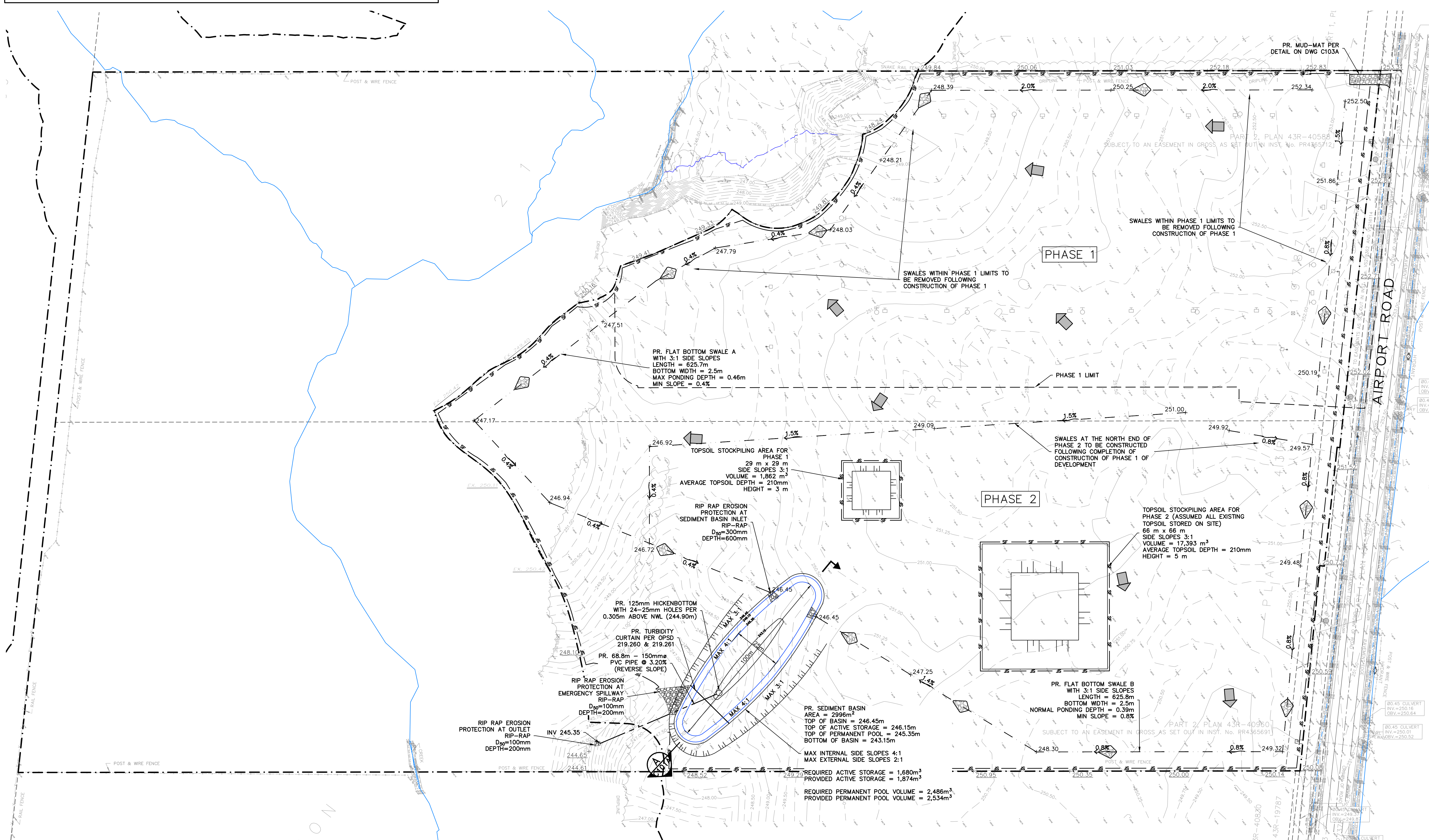
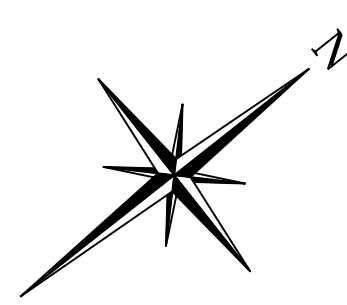
GVF Input Data

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Length	0.00	m
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	m
Profile Description		
Profile Headloss	0.00	m
Downstream Velocity	Infinity	m/s
Upstream Velocity	Infinity	m/s
Normal Depth	0.13	m
Critical Depth	0.25	m
Channel Slope	4.00000	m/m (H:V)
Critical Slope	0.03063	m/m

DRAWINGS



THE EROSION AND SEDIMENT CONTROLS SHALL BE A MULTI BARRIER APPROACH TO PREVENT EROSION DURING CONSTRUCTION TO DEAL WITH SEDIMENT TRANSPORT AT SOURCE AND TO MINIMIZE SEDIMENT TRANSPORT FROM LEAVING THE SITE. THE MITIGATION MEASURES OUTLINED BELOW SHALL BE MAINTAINED BY THE CONTRACTOR THROUGH REGULAR INSPECTIONS, MONITORING AND MAINTENANCE UNTIL THE SOIL HAS BEEN STABILIZED. THE CONTRACTOR SHALL KEEP A COPY OF THE ESC PLANS AND THE TORONTO AND REGION CONSERVATION AUTHORITY, EROSION AND SEDIMENT CONTROL GUIDELINE, DECEMBER 2006, ON SITE AT ALL TIMES.

EROSION AND SEDIMENT CONTROL (ESC) MEASURES WILL BE IMPLEMENTED PRIOR TO, AND MAINTAINED DURING THE CONSTRUCTION PHASES TO PREVENT ENTRY OF SEDIMENT INTO THE WATER. ALL DAMAGED EROSION AND SEDIMENT CONTROL MEASURES SHOULD BE REPAIRED AND/OR REPLACED WITHIN 48 HOURS OF THE INSPECTION.

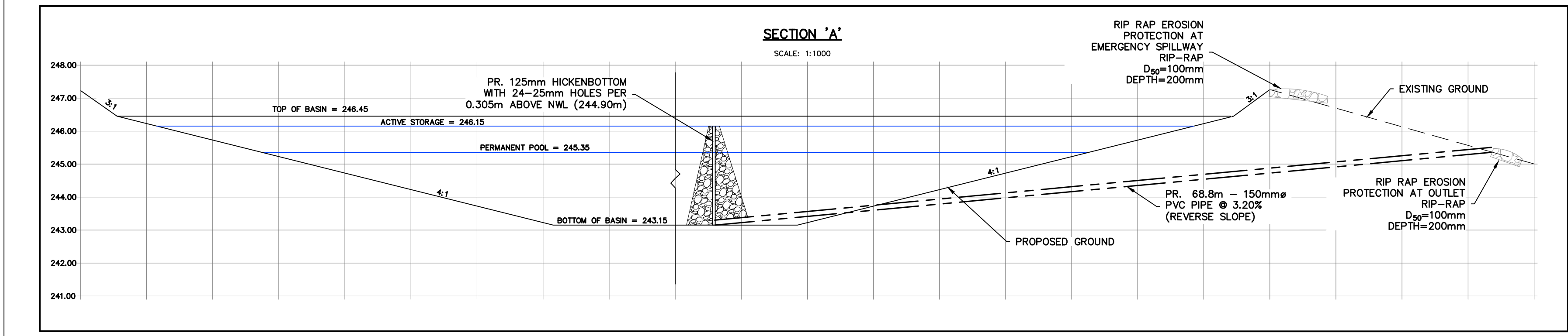
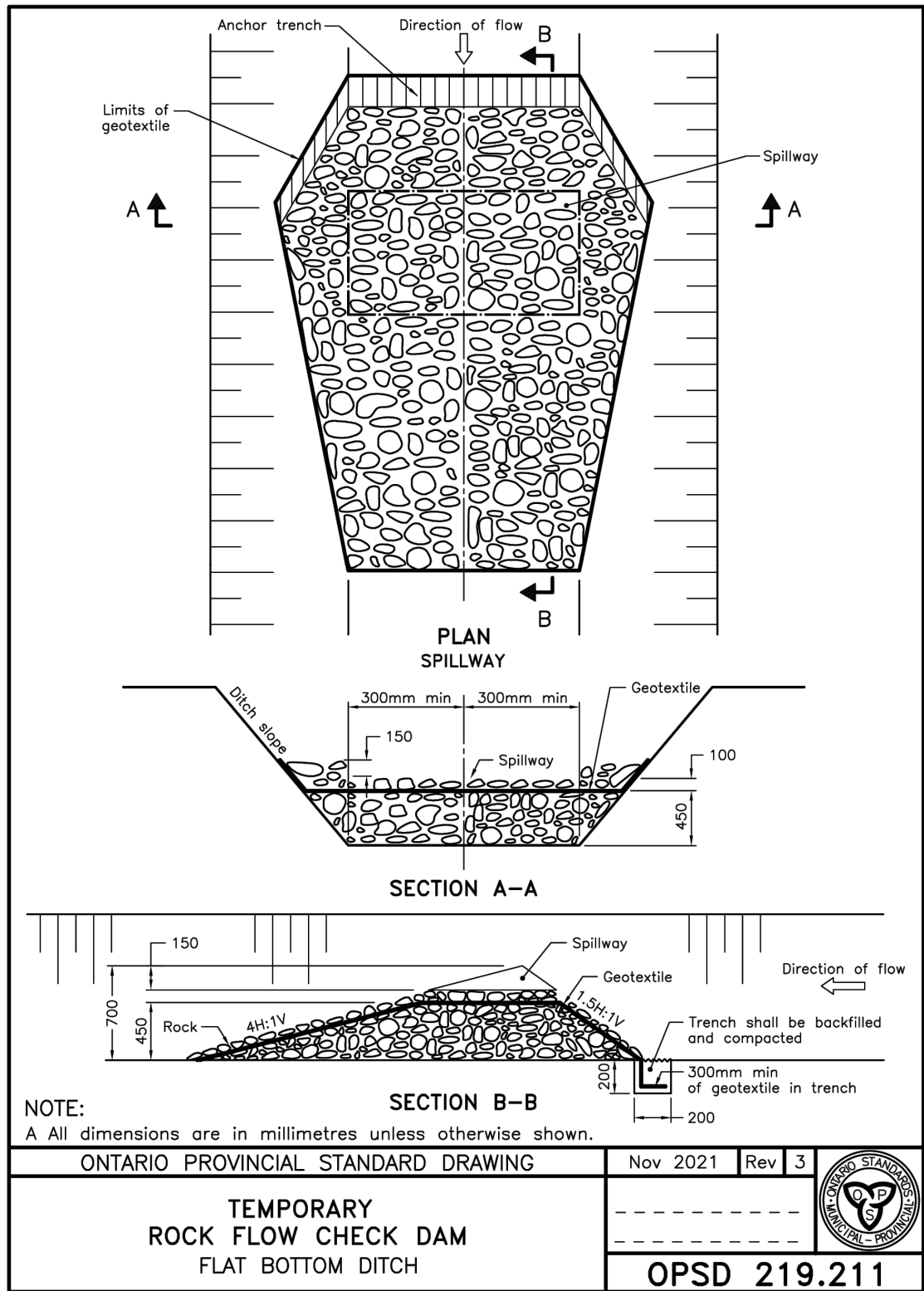
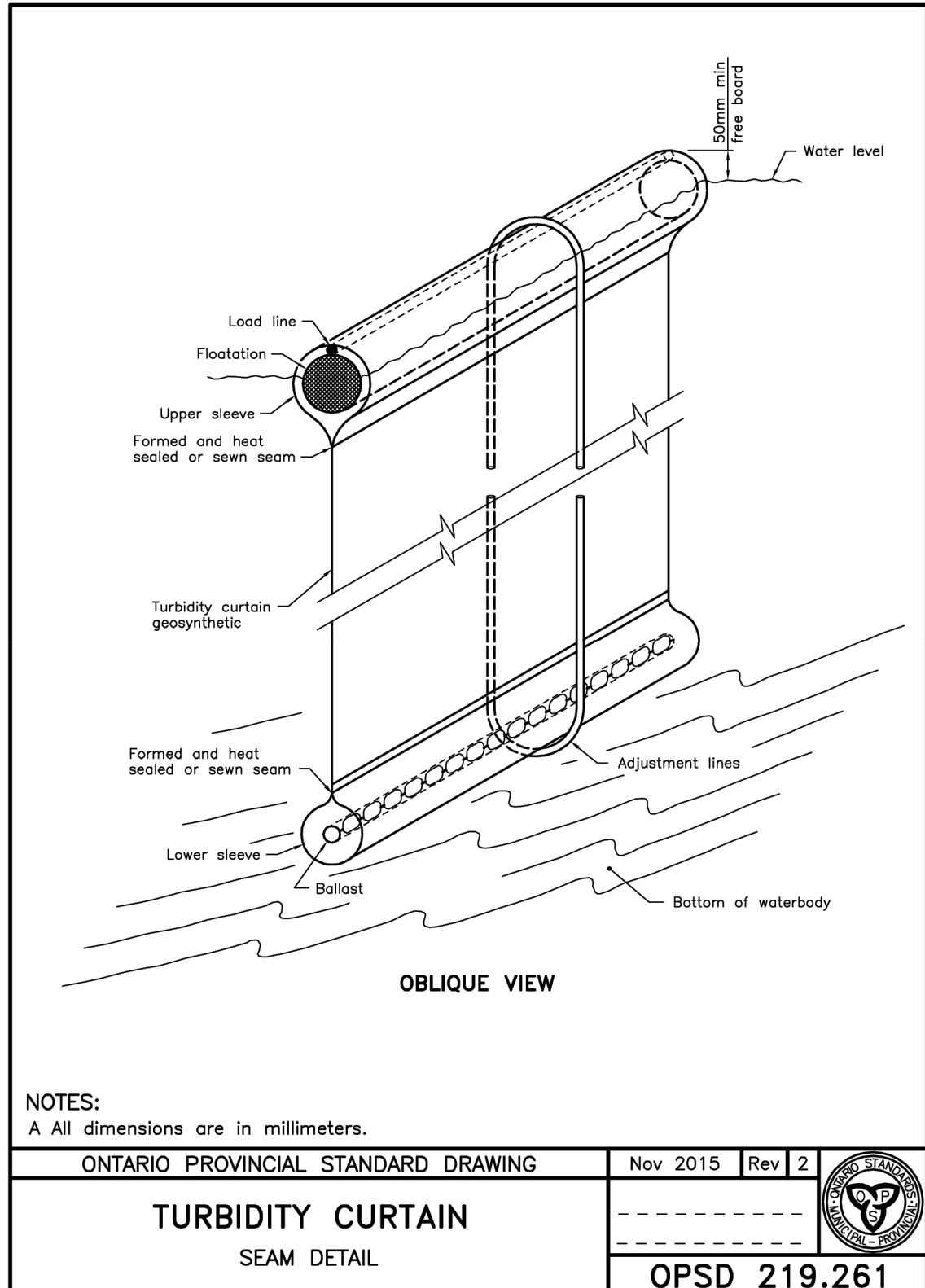
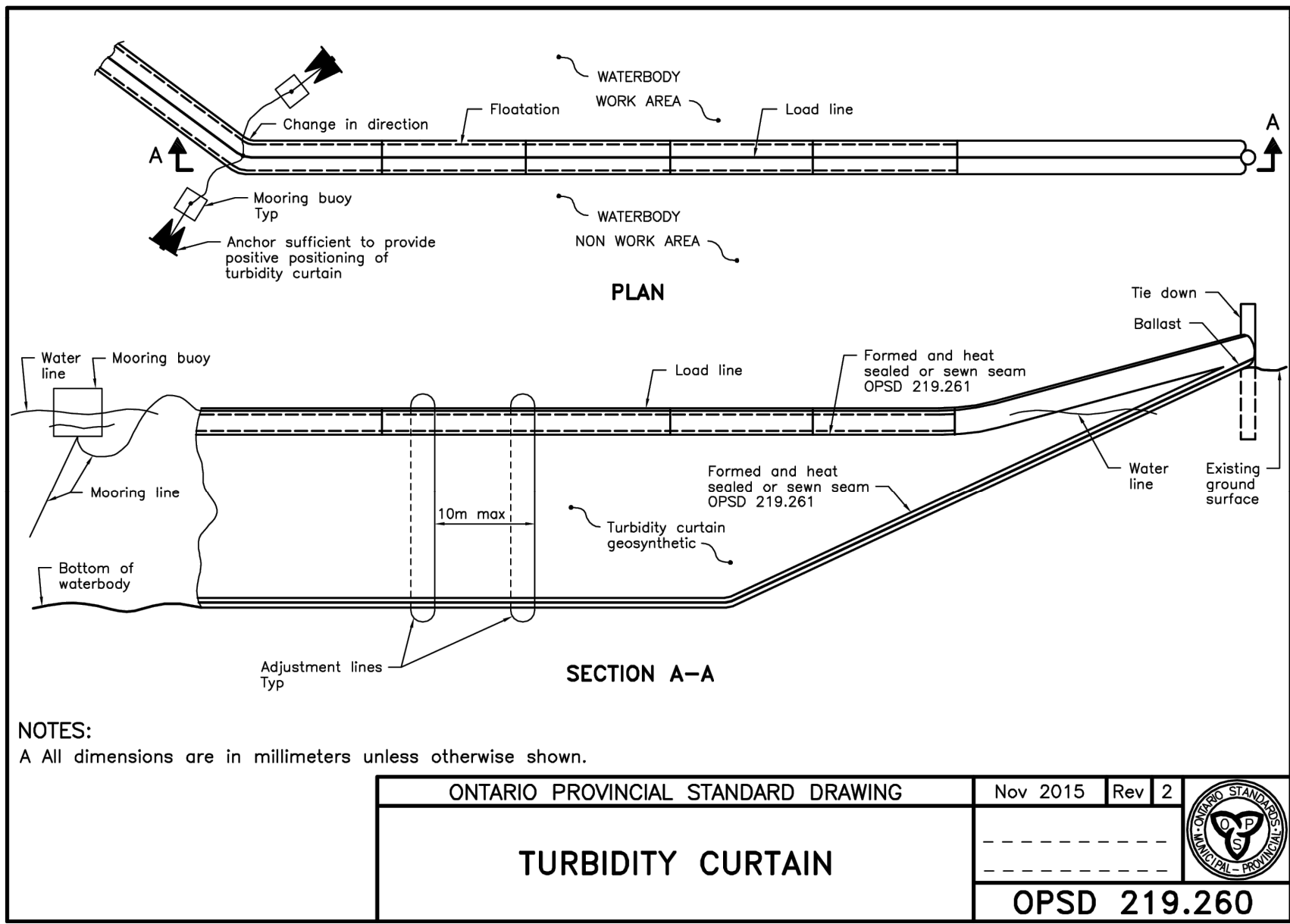
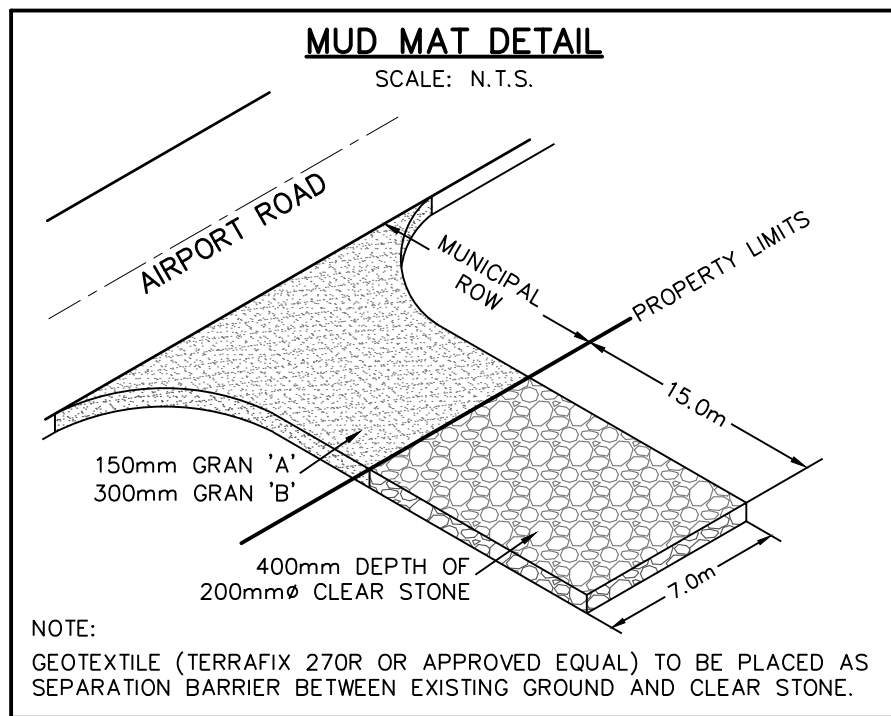
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THE FOLLOWING STANDARD NOTES ARE TO APPEAR ON THE EROSION AND SEDIMENT CONTROL PLANS FOR SITE PLAN SUBMISSIONS:

1. ALL EROSION AND SEDIMENT CONTROL (ESC) MEASURES MUST BE INSTALLED PRIOR TO THE COMMENCEMENT OF WORKS ON SITE AND SHALL BE MAINTAINED DURING ALL CONSTRUCTION PHASES OF THE WORKS.
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR NOTIFYING THE TOWN OF ANY UPDATES/UPGRADES/AMENDS AS THE PROJECT CONDITIONS CHANGE TO PREVENT SEDIMENT RELEASE OUTSIDE THE WORK AREA. THE TOWN IS TO BE ADVISED OF ANY CHANGES TO ESC MEASURES, AND AT THE DISCRETION OF THE TOWN, UPDATE PLANS MAY BE REQUIRED.
3. THE CONTRACTOR IS RESPONSIBLE FOR OBTAINING ALL NECESSARY PERMITS IN ACCORDANCE WITH THE TOWN OF COVINGTON ENVIRONMENTAL PROVISIONS AND AGENCY PERMITS. THE EM SHALL BE A QUALIFIED PERSON AS DETERMINED BY THE TOWN AND/OR CONSERVATION AUTHORITY. THE EM SHOULD SUBMIT INSPECTION FORMS ELECTRONICALLY TO THE TOWN AND CONSERVATION AUTHORITY AS REQUESTED.
4. THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING ALL ESC MEASURES IN WORKING CONDITION AT ALL TIMES TO THE SATISFACTION OF THE TOWN AND/OR CONSERVATION AUTHORITY. ALL DAMAGED ESC MEASURES SHOULD BE REPAIRED AND/OR REPLACED WITHIN 48 HOURS OF THE INSPECTION.
5. THE CONTRACTOR SHALL ROUTINELY INSPECT ESC DEVICES TO ENSURE PROPER WORKING ORDER. FREQUENCY OF INSPECTIONS OF ESC MEASURES IS AS FOLLOWS: DURING EARTHWORKS ACTIVITIES:
 - 6. ALL CONSTRUCTION VEHICLES MUST ENTER AND EXIT THE SITE ONLY FROM THE APPROVED ACCESS ROUTE(S) AS SHOWN ON THE CONSTRUCTION ACCESS PLAN. ACCESS SHALL BE MAINTAINED TO THE SATISFACTION OF THE TOWN/REGULATORY. STREET SWEEPING IS REQUIRED AS NEEDED.

THE FOLLOWING STANDARD NOTES ARE TO APPEAR ON THE EROSION AND SEDIMENT CONTROL PLANS FOR SITE PLAN SUBMISSIONS:

1. ALL EROSION AND SEDIMENT CONTROL (ESC) MEASURES MUST BE INSTALLED PRIOR TO THE COMMENCEMENT OF WORKS ON SITE AND SHALL BE MAINTAINED DURING ALL CONSTRUCTION PHASES OF THE WORKS.
2. MAJOR CONSTRUCTION EROSION AND SEDIMENT CONTROL, THROUGH DEVELOPMENT AND IMPLEMENTATION OF AN EROSION AND SEDIMENT CONTROL PLAN, SHALL BE REQUIRED. THE ESC PLAN SHALL HAVE REGARD TO:
 - CANADIAN STANDARDS ASSOCIATION (CSA) W202 EROSION AND SEDIMENT CONTROL, INSPECTION AND MONITORING STANDARD (AS AMENDED); OR
 - EROSION AND SEDIMENT CONTROL GUIDELINE FOR URBAN CONSTRUCTION 2019 BY TRCA (AS AMENDED).
3. INSTALLATION AND MAINTENANCE OF THE ESC MEASURES SPECIFIED IN THE ESC PLAN SHALL HAVE REGARD TO CSA W202-20 EROSION AND SEDIMENT CONTROL, INSPECTION AND MAINTENANCE (AS AMENDED).
4. THE ESC STRATEGIES OUTLINED ON THIS PLAN ARE NOT STATUTORY AND MAY NEED TO BE UPGRADED/AMENDED AS SITE CONDITIONS CHANGE TO PREVENT SEDIMENT RELEASE OUTSIDE THE WORK AREA. THE TOWN IS TO BE ADVISED OF ANY CHANGES TO ESC MEASURES, AND AT THE DISCRETION OF THE TOWN, UPDATED PLANS MAY BE REQUIRED.
5. THE CONTRACTOR IS TO ASSIGN AN ENVIRONMENTAL MONITOR (EM) TO ASSURE CONSTRUCTION ACTIVITIES COMPLY WITH THE ENVIRONMENTAL PROVISIONS AND AGENCY PERMITS. THE EM WILL BE A QUALIFIED PERSON AS DETERMINED BY THE TOWN AND/OR CONSERVATION AUTHORITY. THE EM SHOULD SUBMIT INSPECTION REPORTS TO THE TOWN AND/OR CONSERVATION AUTHORITY UPON REQUEST.
6. THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING ALL ESC MEASURES IN WORKING CONDITION AT ALL TIMES TO THE SATISFACTION OF THE TOWN AND/OR CONSERVATION AUTHORITY. ALL DAMAGED ESC MEASURES SHOULD BE REPAIRED AND/OR REPLACED WITHIN 48 HOURS OF THE INSPECTION.
7. THE CONTRACTOR SHALL ROUTINELY INSPECT ALL ESC DEVICES TO ENSURE PROPER WORKING ORDER. FREQUENCY OF INSPECTIONS OF ESC DEVICES IS AS FOLLOWS:
 - ONCE PER WEEK DURING RAINFALLY ACTIVITIES
 - ONCE EVERY TWO WEEKS DURING TIMES OF NO RAINFALLWORKS
 - PRIOR TO PREDICTED RAINFALL EVENTS OF 10 MM OR MORE OVER 12 HOURS
 - WITHIN 48 HOURS OF EVERY RAINFALL EVENT OF 10 MM OR MORE OVER 12 HOURS
 - PRIOR TO SIGNIFICANT SNOWMELT EVENTS
 - WITHIN 48 HOURS OF SIGNIFICANT SNOWMELT EVENTS; AND
 - DAILY DURING EXTENDED RAIN OR SNOWMELT PERIODS;
 - AFTER EXTREME WEATHER EVENTS (FOR EXAMPLE: WIND)
7. RECORD OF ESC INSPECTIONS SHALL BE ROUTINELY SUBMITTED TO THE TOWN AND INCLUDE AT MINIMUM: THE NAME OF THE INSPECTOR, DATE OF INSPECTION, VISUAL OBSERVATIONS, AND THE REMEDIAL MEASURES, IF ANY, UNDERTAKEN TO MAINTAIN THE TEMPORARY ESC MEASURES.
8. ALL CONSTRUCTION VEHICLES MUST ENTER AND EXIT THE SITE ONLY FROM THE APPROVED ACCESS ROUTE(S) AS SHOWN ON THE PLANS. CONSTRUCTION ACCESS WILL BE MAINTAINED TO THE SATISFACTION OF THE TOWN AND/OR CONSERVATION AUTHORITY.
9. NO CONSTRUCTION ACTIVITY OR MACHINERY SHALL BE ALLOWED BEYOND THE SILT FENCE OR LIMITS OF THE SITE WORKS.
10. THE CONTRACTOR IS RESPONSIBLE TO IMPLEMENT DUST CONTROL MEASURES AND CONSTRUCTION BEST PRACTICE GUIDELINES AS APPROVED BY THE TOWN AND/OR CONSERVATION AUTHORITY.
11. ALL DISTURBED GROUND LEFT INACTIVE FOR 30 DAYS SHALL BE VEGETATED, SUBJECT TO WEATHER CONDITIONS. DISTURBED AREAS ARE TO BE HYDROSEED (OR APPROVED EQUIVALENT) TO THE SATISFACTION OF THE TOWN
12. ALL TOPSOIL STOCKPILES SHALL BE SURROUNDED WITH SEDIMENT CONTROL FENCE. THE MAXIMUM SIDE SLOPE SHALL BE 2:1 (H:V). ONCE TOPSOIL STRIPPING IS COMPLETED THE STOCKPILE SHALL BE VEGETATED, SUBJECT TO WEATHER CONDITIONS, BY HYDROSEEDING, OR AN APPROVED EQUIVALENT, TO THE SATISFACTION OF THE TOWN.
13. THE CONTRACTOR SHALL TAKE CARE AND CONTROL SPILLS, FLUIDS AND MATERIALS DURING CONSTRUCTION TO MINIMIZE RISK TO THE ENVIRONMENT.



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4. ALL TIEING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.
5. DO NOT SCALE DRAWINGS.

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO CITY OF BRAMPTON VERTICAL BENCHMARK NUMBER 042200365 HAVING AN ORTHOMETRIC ELEVATION OF 242.135 METRES. ELEVATIONS ARE REFERENCED TO THE CANADIAN GEODETIC VERTICAL DATUM OF 1928, 1978 ADJUSTMENT (CGVD-1928/1978 ADJ.). BRASS CAP SET IN CONCRETE, 72.80 METERS SOUTH OF THE CENTRELINE OF MAYFIELD ROAD AND 19.81 METERS EAST OF THE CENTRELINE OF TORBRAM ROAD.

SURVEY COMPLETED BY R-PE SURVEYING LTD. (2022/DEC/12)
REFERENCE NO. 22-317

SITE PLAN NOTES

DESIGN ELEMENTS ARE BASED ON SITE PLAN PROVIDED BY POWERS BROWN ARCHITECTURE PROJECT NO. 255025, DRAWING NO. 45101 (2023/09/09)



SPA # 2025-0067

No.	ISSUE	DATE: YYYY/MM/
1	ISSUED FOR FIRST ZBA/SPA SUBMISSION	2025/10/30
2	ISSUED FOR SECOND SPA SUBMISSION	2026/05/05

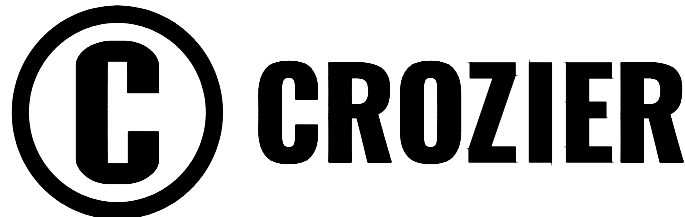


Project

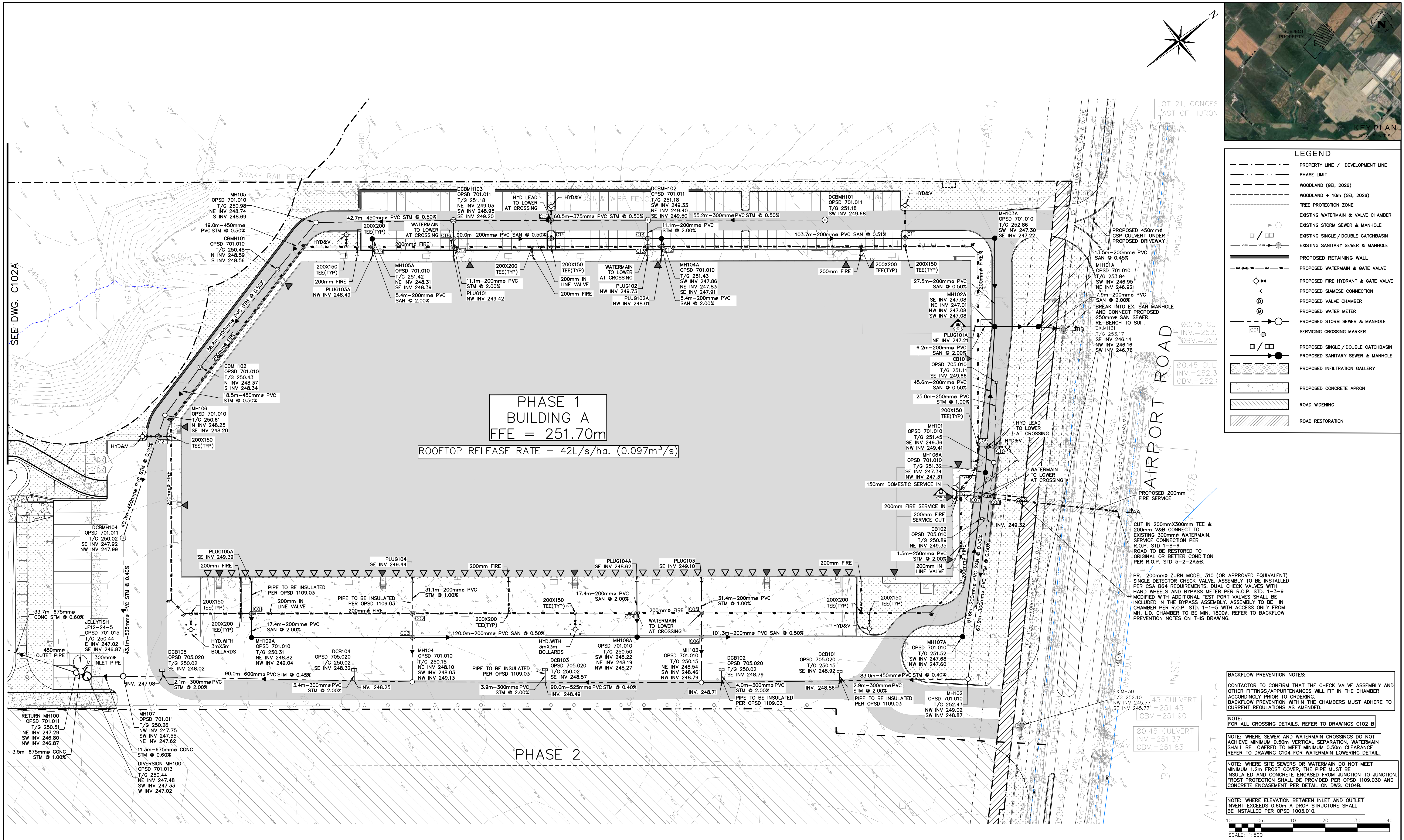
Drawing

0 AIRPORT ROAD
TOWN OF CALEDON

EROSION & SEDIMENT CONTROL DETAILS



Drawn By K. W. /V.M.	Design By K. W. /V.M.	Scale NA	Project 2278-7228
Check By J.S.	Check By J.S.	Sheet 02 OF 19	Drawing C101A



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3. THIS DRAWING IS TO BE READ AND UNDERSTOOD IN CONJUNCTION WITH ALL OTHER PLANS AND DOCUMENTS APPLICABLE TO THIS PROJECT.

4. ALL EXISTING UNDERGROUND UTILITIES TO BE VERIFIED IN THE FIELD BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

5. DO NOT SCALE DRAWINGS.

BENCHMARKS

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO CITY OF BRAMPTON VERTICAL BENCHMARK NUMBER 04220365 HAVING AN ORTHOMETRIC ELEVATION OF 242.135 METRES. ELEVATIONS ARE REFERENCED TO THE CANADIAN GEODETIC VERTICAL DATUM OF 1928, 1978 ADJUSTMENT (CGVD-1928/1978 ADJ). BRASS CAP SET IN CONCRETE, 72.80 METERS SOUTH OF THE CENTRELINE OF MAYFIELD ROAD AND 19.81 METERS EAST OF THE CENTRELINE OF TORBRAM ROAD.

SURVEY COMPLETED BY R-PE SURVEYING LTD. (2022/DEC/12)

REFERENCE NO. 22-317

SITE PLAN NOTES

DESIGN ELEMENTS ARE BASED ON SITE PLAN PROVIDED BY POWERS BROWN ARCHITECTURE PROJECT NO: 255025, DRAWING NO: AS101 (2025/09/09)

Town

TOWN OF CALEDON

SPA # 2025-0067

No.	ISSUE	DATE: YYYY/MM/DD	Stamp
1	ISSUED FOR FIRST ZBA/SPA SUBMISSION	2025/10/30	
2	ISSUED FOR SECOND SPA SUBMISSION	2026/05/05	

Project

0 AIRPORT ROAD
TOWN OF CALEDON

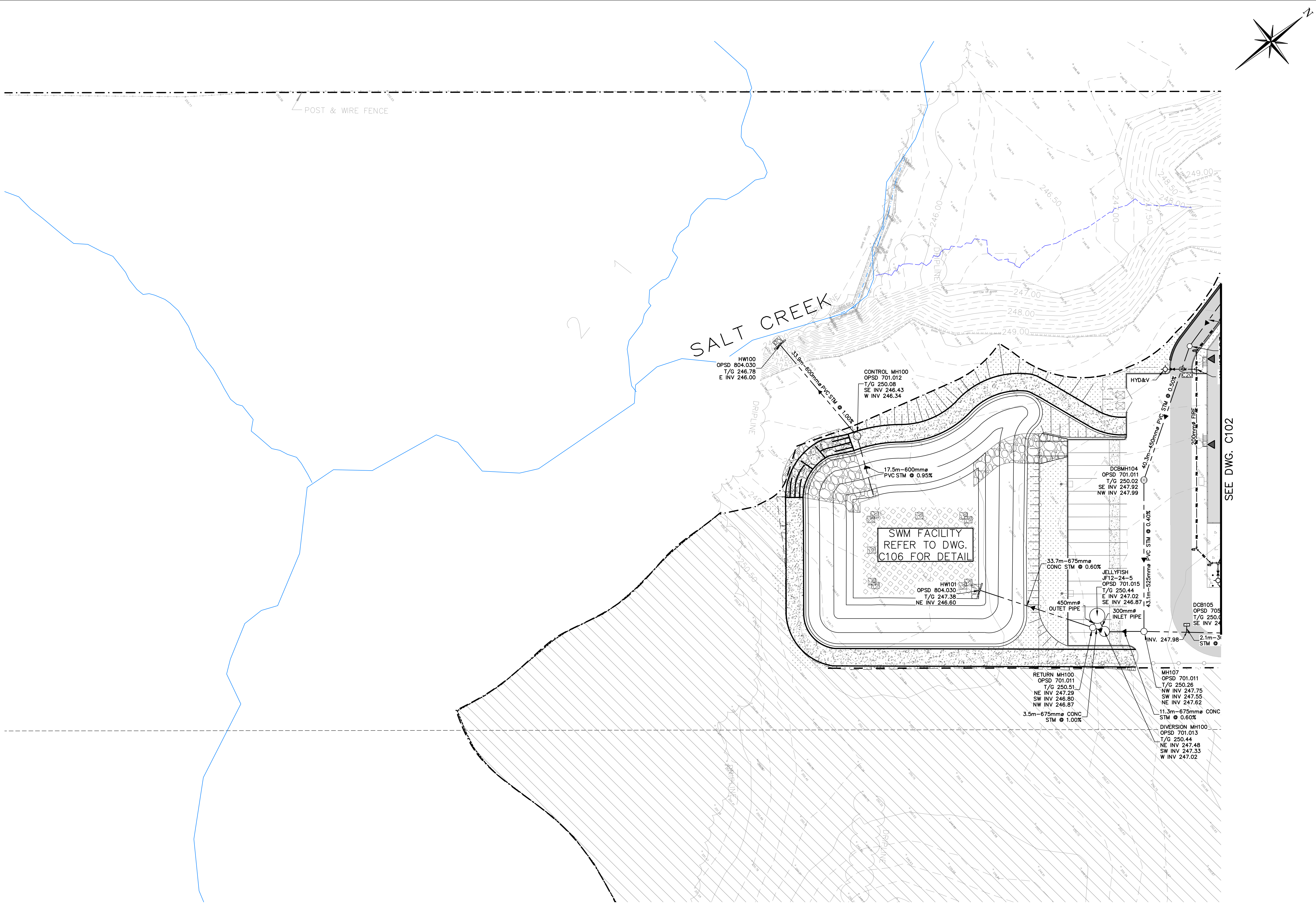
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SITE SERVICING PLAN

Drawn By K.W./V.M. Design By K.W./V.M. Scale 1:500 Project 2278-7228

Check By J.S. Check By J.S. Sheet 03 OF 19 Drawing C102

CROZIER



LEGEND	
	PROPERTY LINE / DEVELOPMENT LINE
	PHASE LIMIT
	WOODLAND (GEI, 2026)
	WOODLAND + 10m (GEI, 2026)
	TREE PROTECTION ZONE
	EXISTING WATERMAIN & VALVE CHAMBER
	EXISTING STORM SEWER & MANHOLE
	EXISTING SINGLE / DOUBLE CATCHBASIN
	EXISTING SANITARY SEWER & MANHOLE
	PROPOSED RETAINING WALL
	PROPOSED WATERMAIN & GATE VALVE
	PROPOSED FIRE HYDRANT & GATE VALVE
	PROPOSED SIAMASE CONNECTION
	PROPOSED VALVE CHAMBER
	PROPOSED WATER METER
	PROPOSED STORM SEWER & MANHOLE
	SERVICING CROSSING MARKER
	PROPOSED SINGLE / DOUBLE CATCHBASIN
	PROPOSED SANITARY SEWER & MANHOLE
	PROPOSED INFILTRATION GALLERY
	PROPOSED CONCRETE APRON
	ROAD WIDENING
	ROAD RESTORATION

BACKFLOW PREVENTION NOTES:

CONTACTOR TO CONFIRM THAT THE CHECK VALVE ASSEMBLY AND OTHER FITTINGS/APPURTENANCES WILL FIT IN THE CHAMBER ACCORDINGLY PRIOR TO ORDERING.

BACKFLOW PREVENTION WITHIN THE CHAMBERS MUST ADHERE TO CURRENT REGULATIONS AS AMENDED.

NOTE: WHERE SEWER AND WATERMAIN CROSSINGS DO NOT ACHIEVE MINIMUM 0.50m VERTICAL SEPARATION, WATERMAIN SHALL BE LOWERED TO MEET MINIMUM 0.50m CLEARANCE REFER TO DRAWING C104 FOR WATERMAIN LOWERING DETAIL.

NOTE: WHERE SITE SEWERS OR WATERMAIN DO NOT MEET MINIMUM 1.2m FROST COVER, THE PIPE MUST BE INSULATED AND CONCRETE ENCASED FROM JUNCTION TO JUNCTION. FROST PROTECTION SHALL BE PROVIDED PER OPSD 1109.030 AND CONCRETE ENCASEMENT PER DETAIL ON DWG. C104B.

NOTE: WHERE ELEVATION BETWEEN INLET AND OUTLET INVERT EXCEEDS 0.60m A DROP STRUCTURE SHALL BE INSTALLED PER OPSD 1003.010.

10 0m 10 20 30 40
SCALE: 1:500

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BENCHMARKS

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BRASS CAP SET IN CONCRETE, 72.80 METERS SOUTH OF THE CENTRELINE OF MAYFIELD ROAD AND 19.81 METERS EAST OF THE CENTRELINE OF TORBRAM ROAD.

SURVEY COMPLETED BY R-PE SURVEYING LTD. (2022/DEC/12)
REFERENCE NO. 22-317

SITE PLAN NOTES

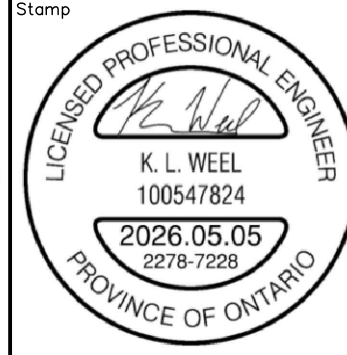
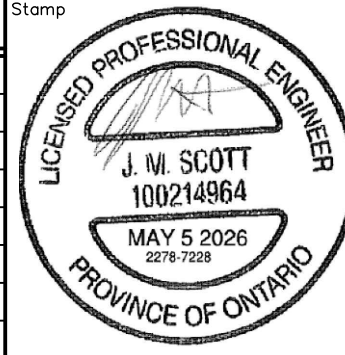
DESIGN ELEMENTS ARE BASED ON SITE PLAN PROVIDED BY POWERS BROWN ARCHITECTURE
PROJECT NO: 255025, DRAWING NO: ASI01 (2025/09/09)

Town

TOWN OF CALEDON

SPA # 2025-0067

No.	ISSUE	DATE: YYYY/MM/DD
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Project

0 AIRPORT ROAD
TOWN OF CALEDON

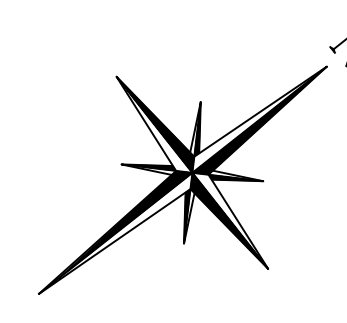
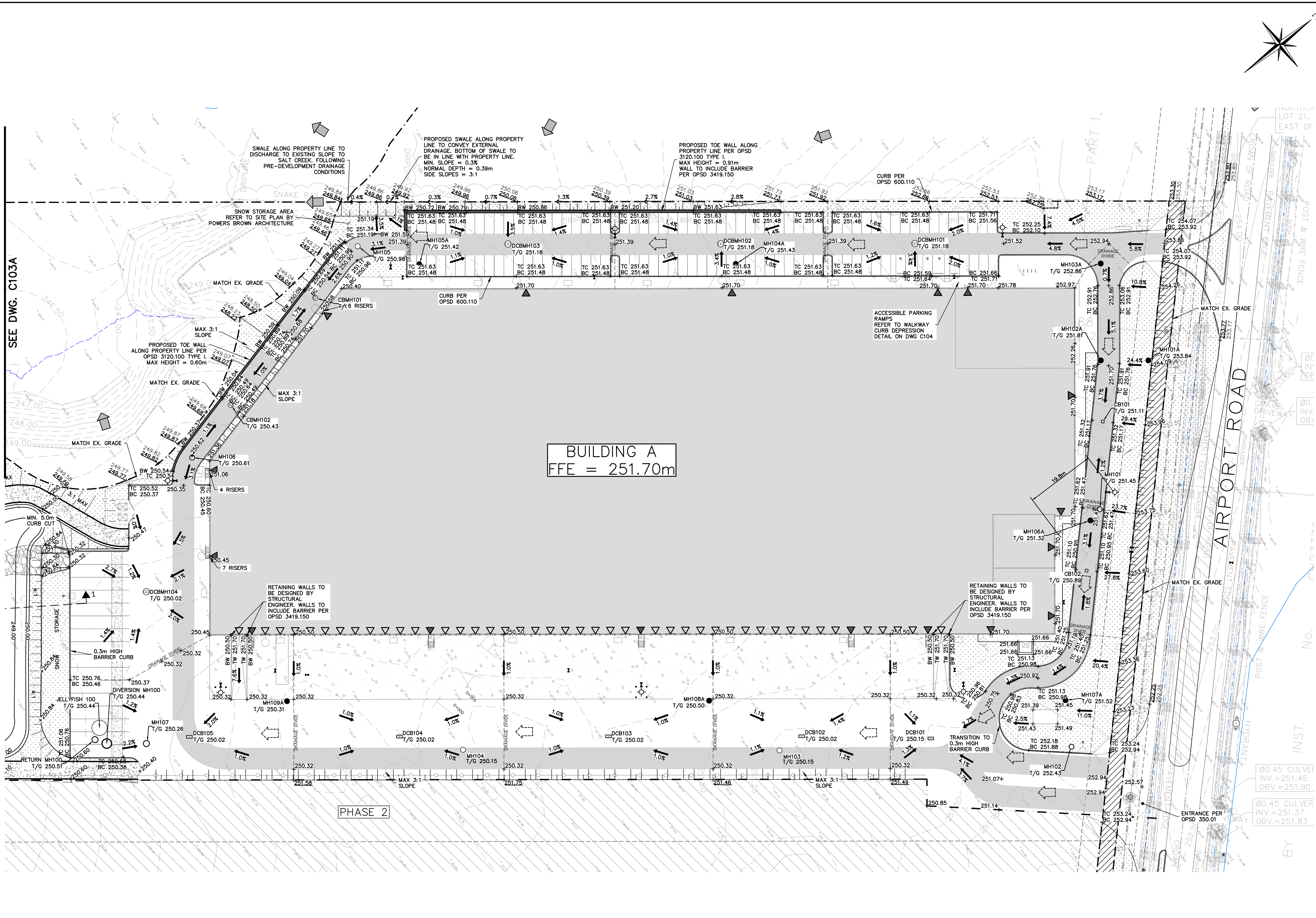
Drawing

SITE SERVICING PLAN

CROZIER

Drawn By K.W./V.M. Design By K.W./V.M. Scale 1:500 Project 2278-7228

Check By J.S. Check By J.S. Sheet 04 OF 19 Drawing C102 A



- LEGEND**
- PROPERTY LINE
 - WOODLAND (GEI, 2026)
 - WOODLAND + 10m (GEI, 2026)
 - PHASE 1 LIMIT
 - TREE PROTECTION ZONE
 - EXISTING GRADE
 - MATCH GRADE TO EXISTING SURFACE
 - PROPOSED GRADE
 - PROPOSED MINOR FLOW DIRECTION
 - PROPOSED GRASSED SWALE
 - PROPOSED RETAINING WALL
 - PROPOSED SLOPE (3:1 MAX.)
 - BUILDING ENTRANCE (PERSONNEL DOOR)
 - BUILDING ENTRANCE (OVERHEAD DOOR)
 - PROPOSED MAJOR OVERLAND FLOW DIRECTION
 - EXISTING MAJOR OVERLAND FLOW DIRECTION
 - PROPOSED FIRE HYDRANT & GATE VALVE
 - ROAD WIDENING



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REFERENCE NO. 22-317

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PROJECT NO. 255025, DRAWING NO. A5101 (2025/09/09)

Town

**TOWN OF CALEDON**

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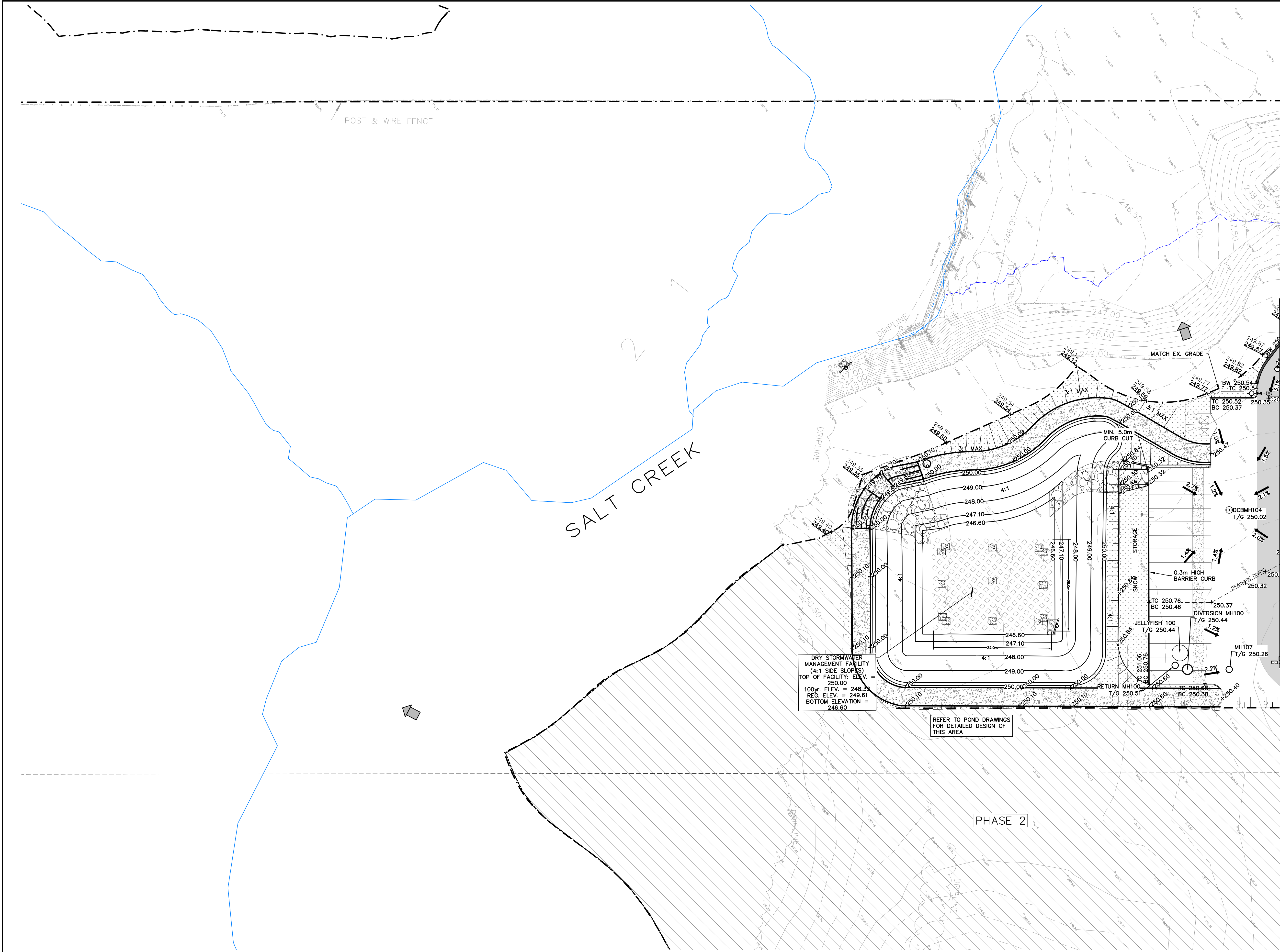


Project
0 AIRPORT ROAD
TOWN OF CALEDON

Drawing
SITE GRADING PLAN

**CROZIER**

Drawn By	K.W./V.M.	Design By	K.W./V.M.	Scale	1:500	Project	2278-7228
Check By	J.S.	Check By	J.S.	Sheet	07 OF 19	Drawing	C103



LEGEND

- PROPERTY LINE
- WOODLAND (GEI, 2026)
- WOODLAND + 10m (GEI, 2026)
- PHASE 1 LIMIT
- TREE PROTECTION ZONE
- EXISTING GRADE
- MATCH GRADE TO EXISTING SURFACE
- PROPOSED GRADE
- PROPOSED MINOR FLOW DIRECTION
- PROPOSED GRASSSED SWALE
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- BUILDING ENTRANCE (PERSONNEL DOOR)
- BUILDING ENTRANCE (OVERHEAD DOOR)
- PROPOSED MAJOR OVERLAND FLOW DIRECTION
- EXISTING MAJOR OVERLAND FLOW DIRECTION
- PROPOSED FIRE HYDRANT & GATE VALVE
- ROAD WIDENING

SEE DWG. C103



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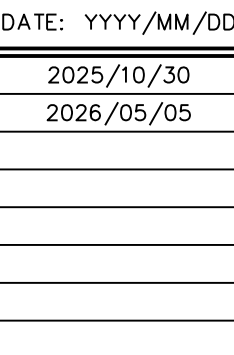
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TOWN OF CALEDON

SPA # 2025-0067

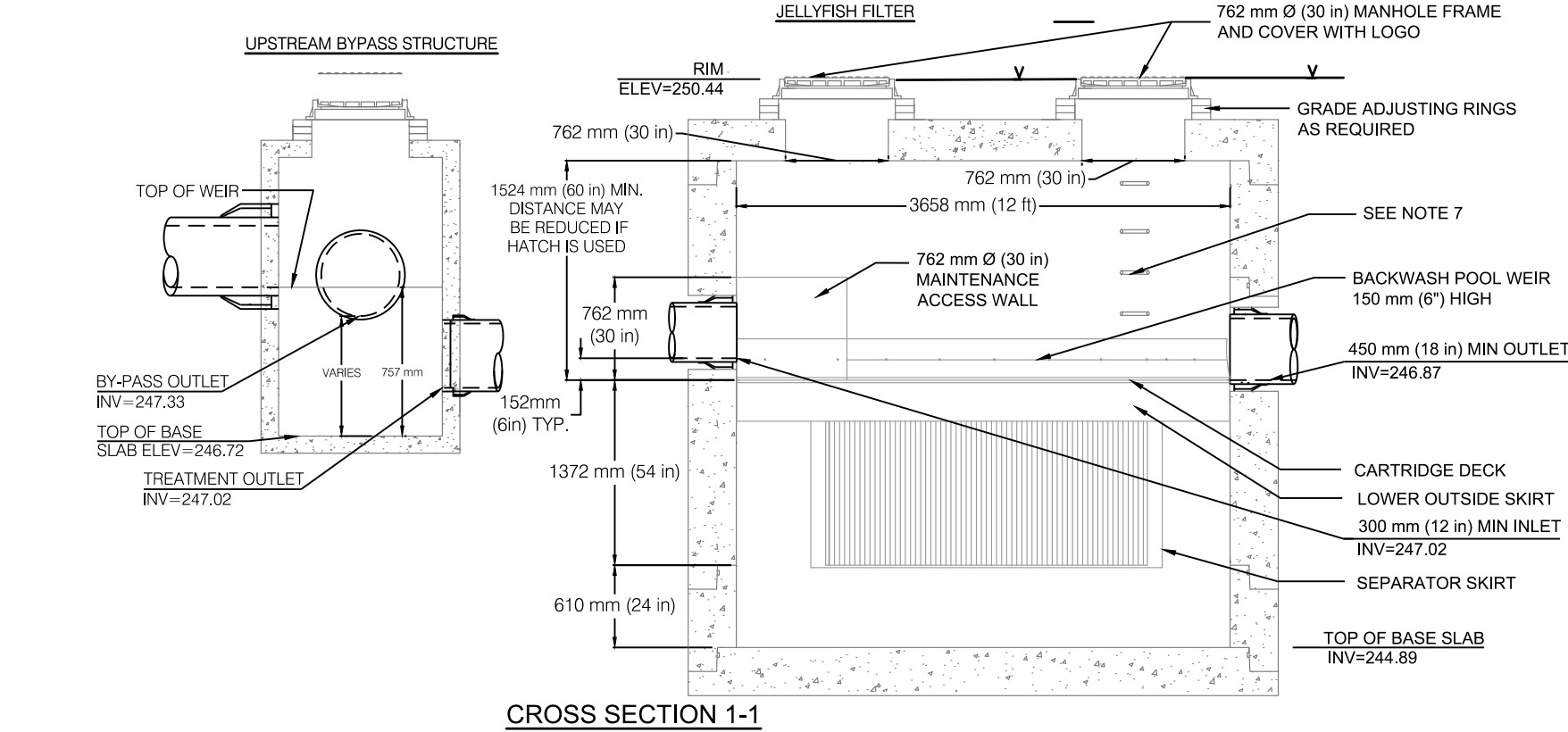
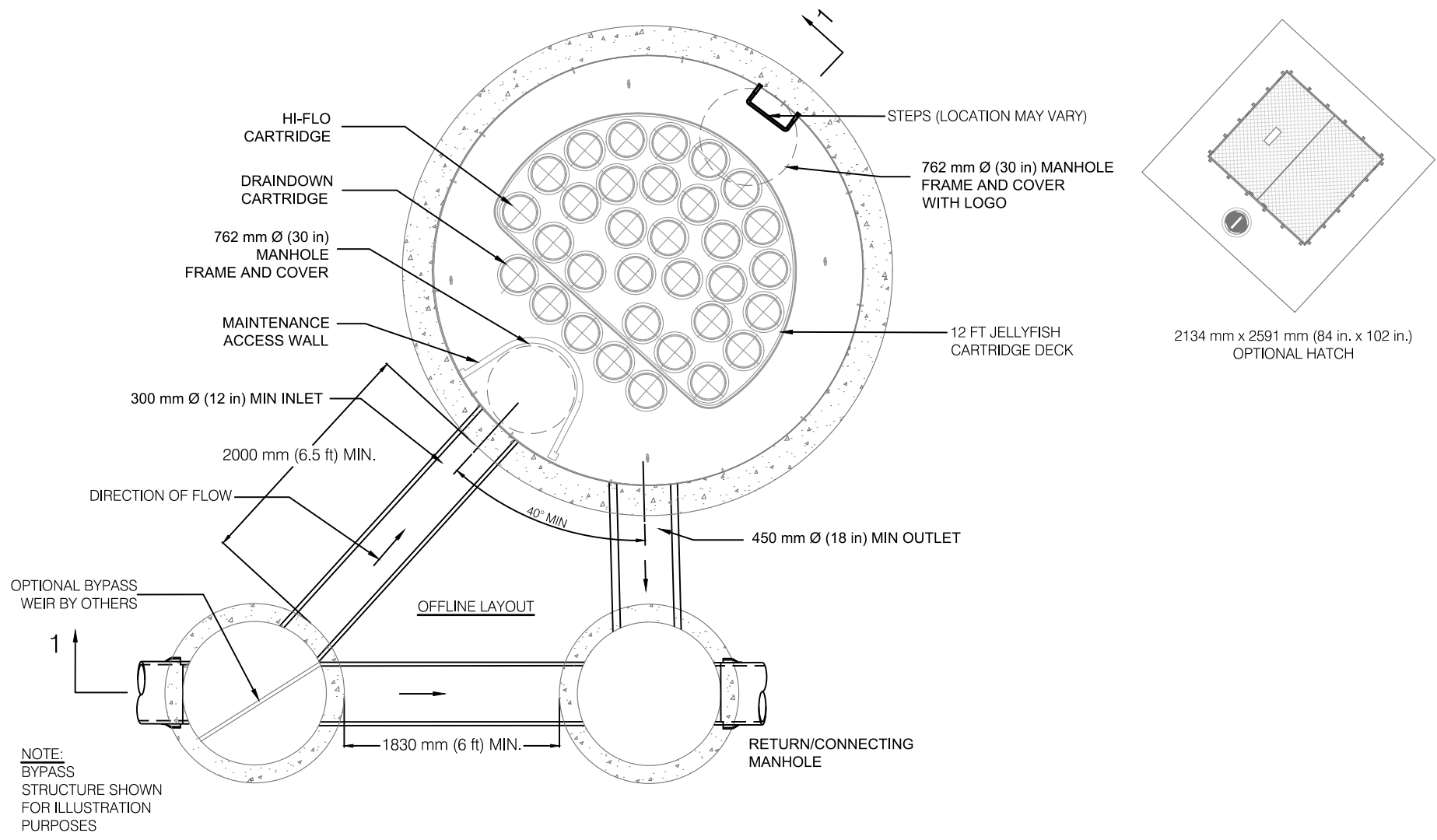
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Project
0 AIRPORT ROAD
TOWN OF CALEDON

Drawing
SITE GRADING PLAN

Drawn By	K.W./V.M.	Design By	K.W./V.M.	Scale	1:500	Project	2278-7228
Check By	J.S.	Check By	J.S.	Sheet	## OF 19	Drawing	C103 A



NOTE: SHOP DRAWINGS TO BE PROVIDED BY RINKER AT THE TIME OF CONSTRUCTION



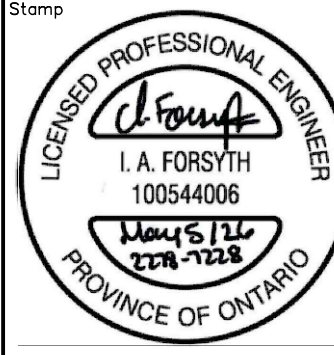
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PROJECT NO: 255025, DRAWING NO: ASI01 (2025/09/09)

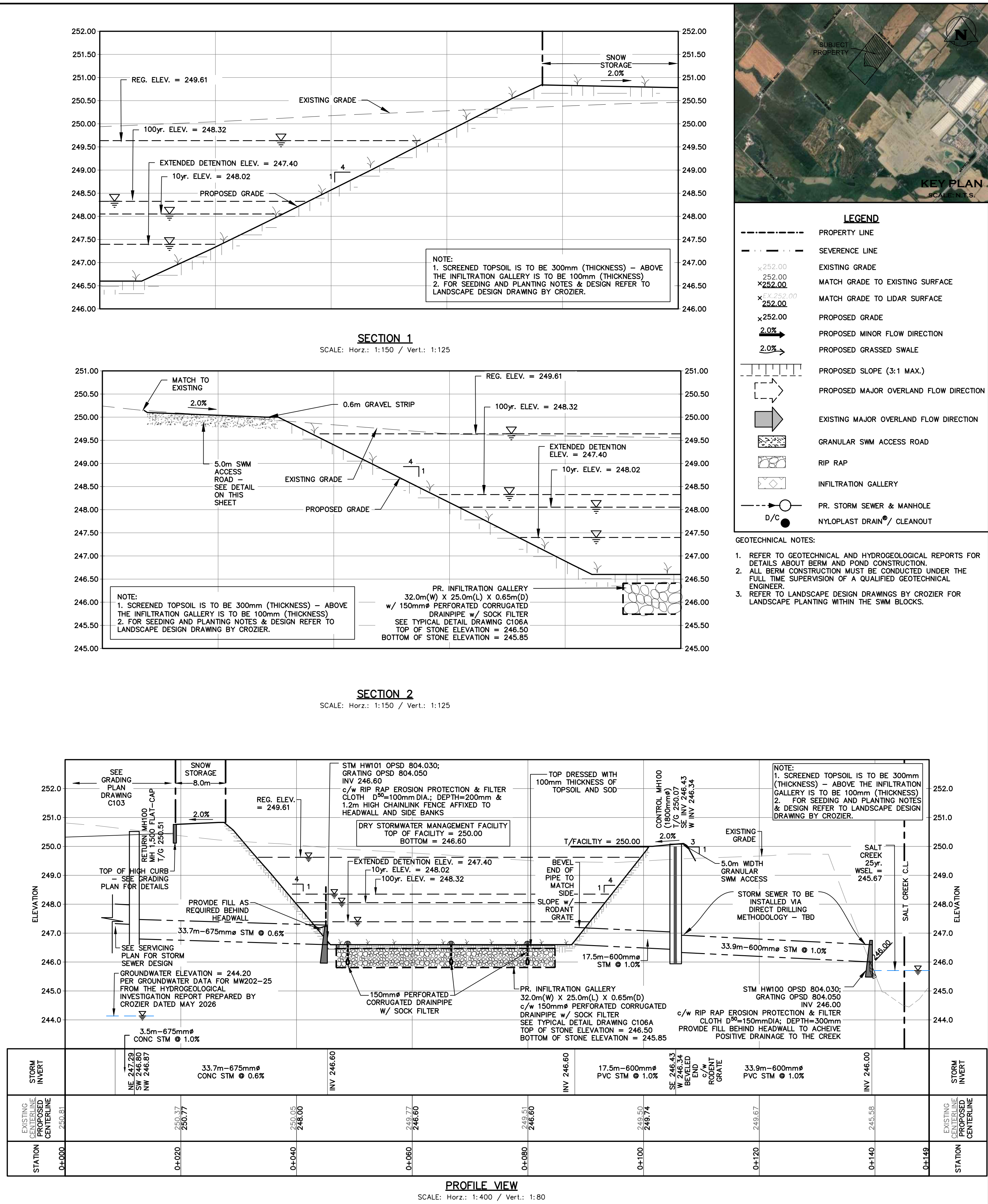
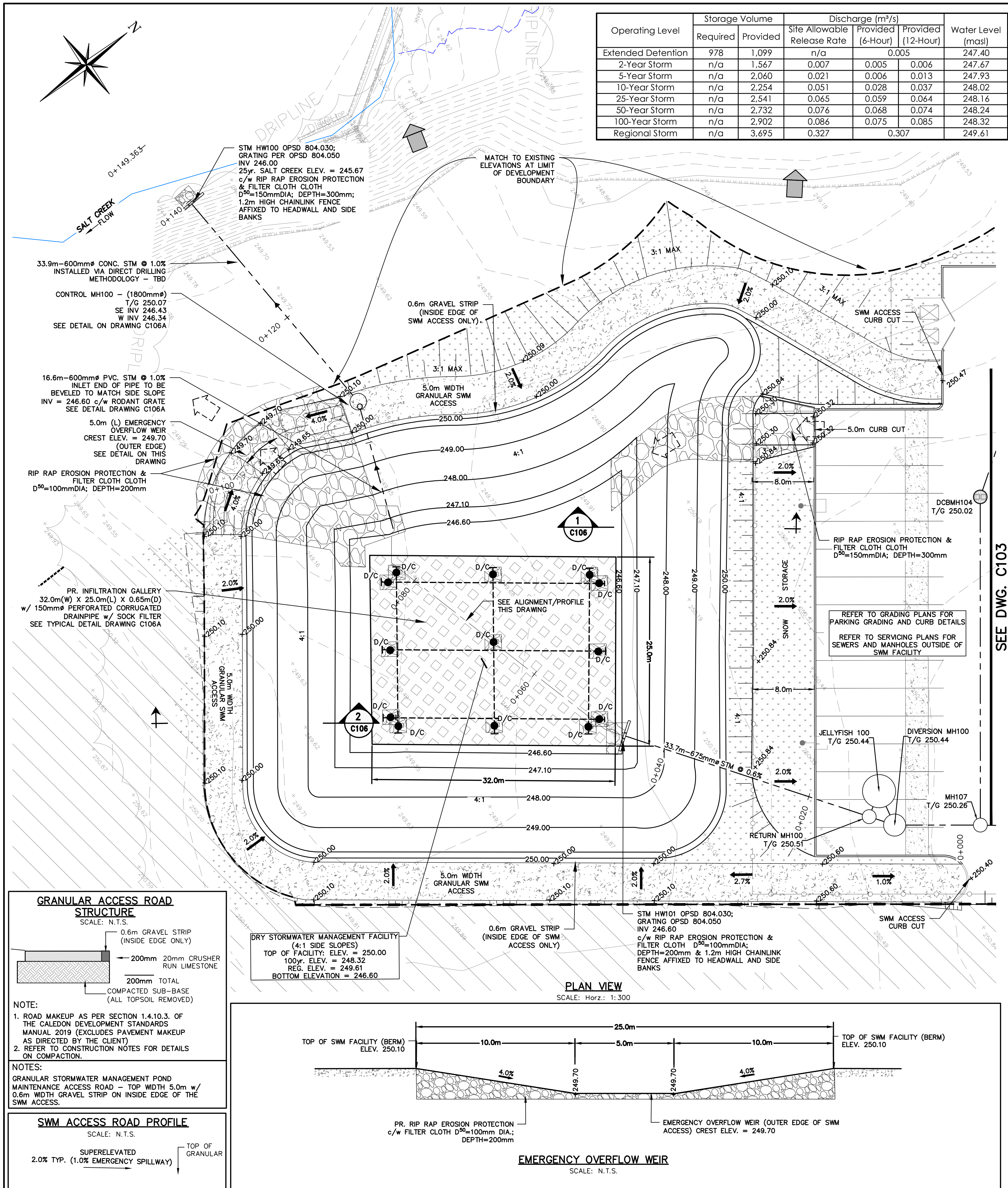


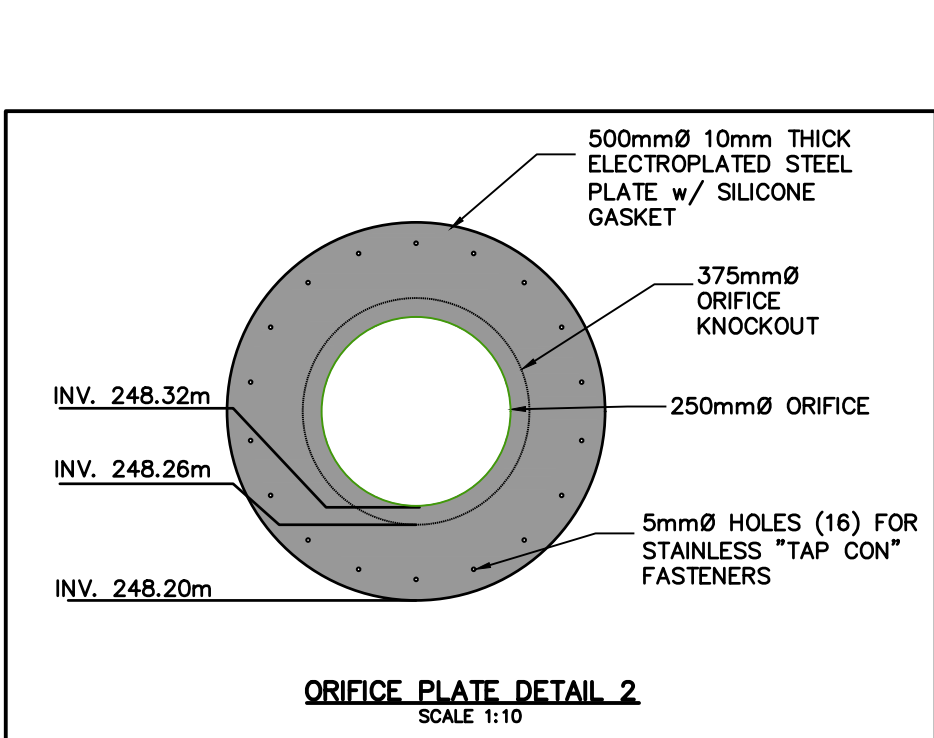
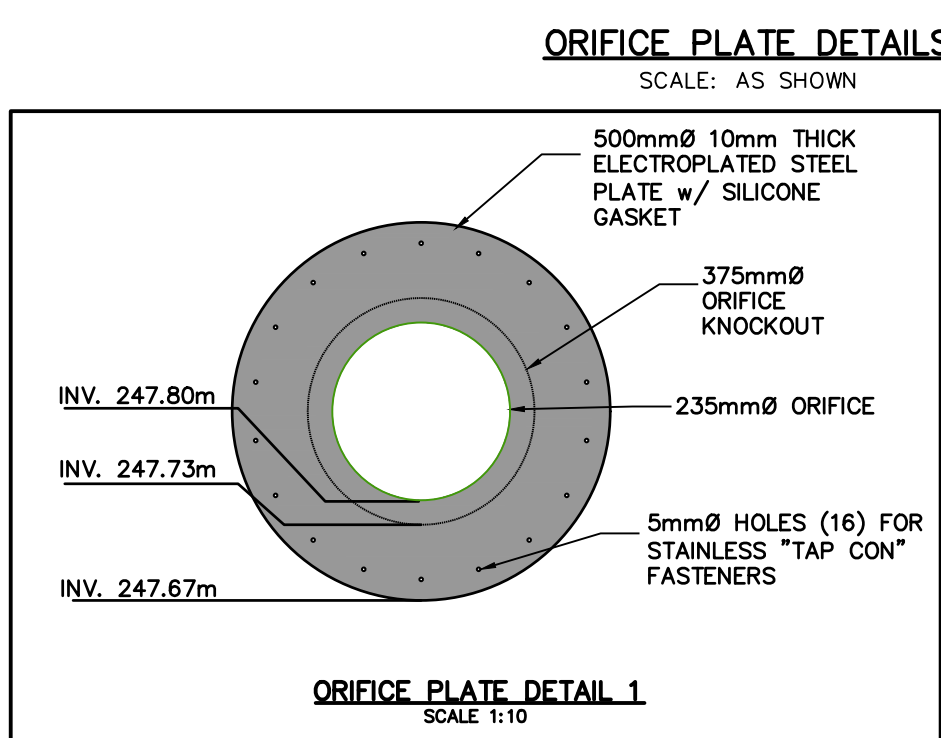
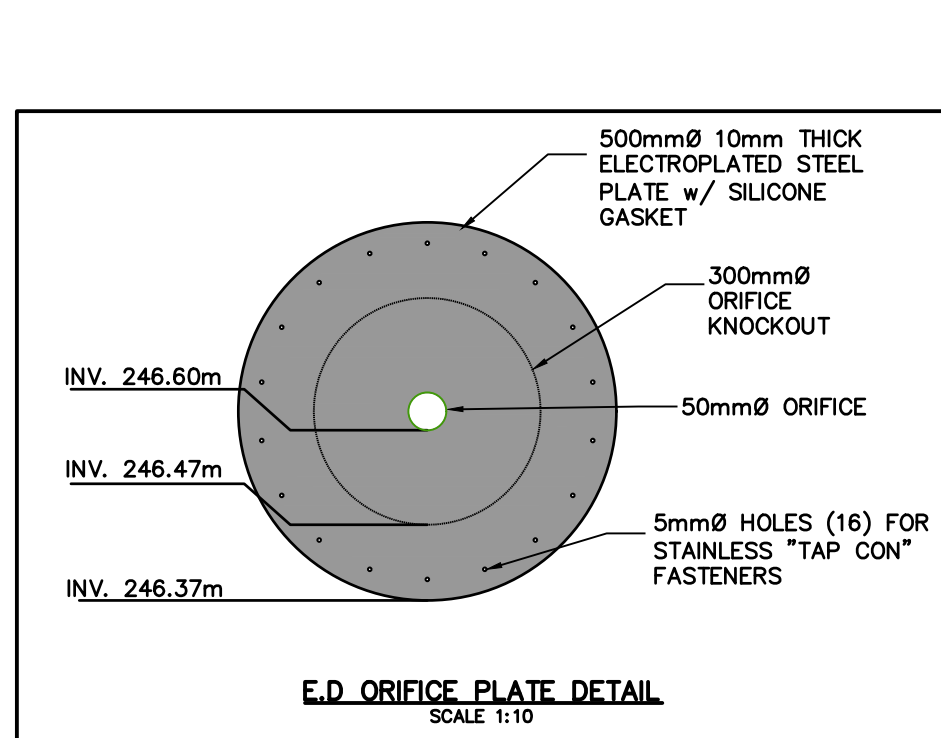
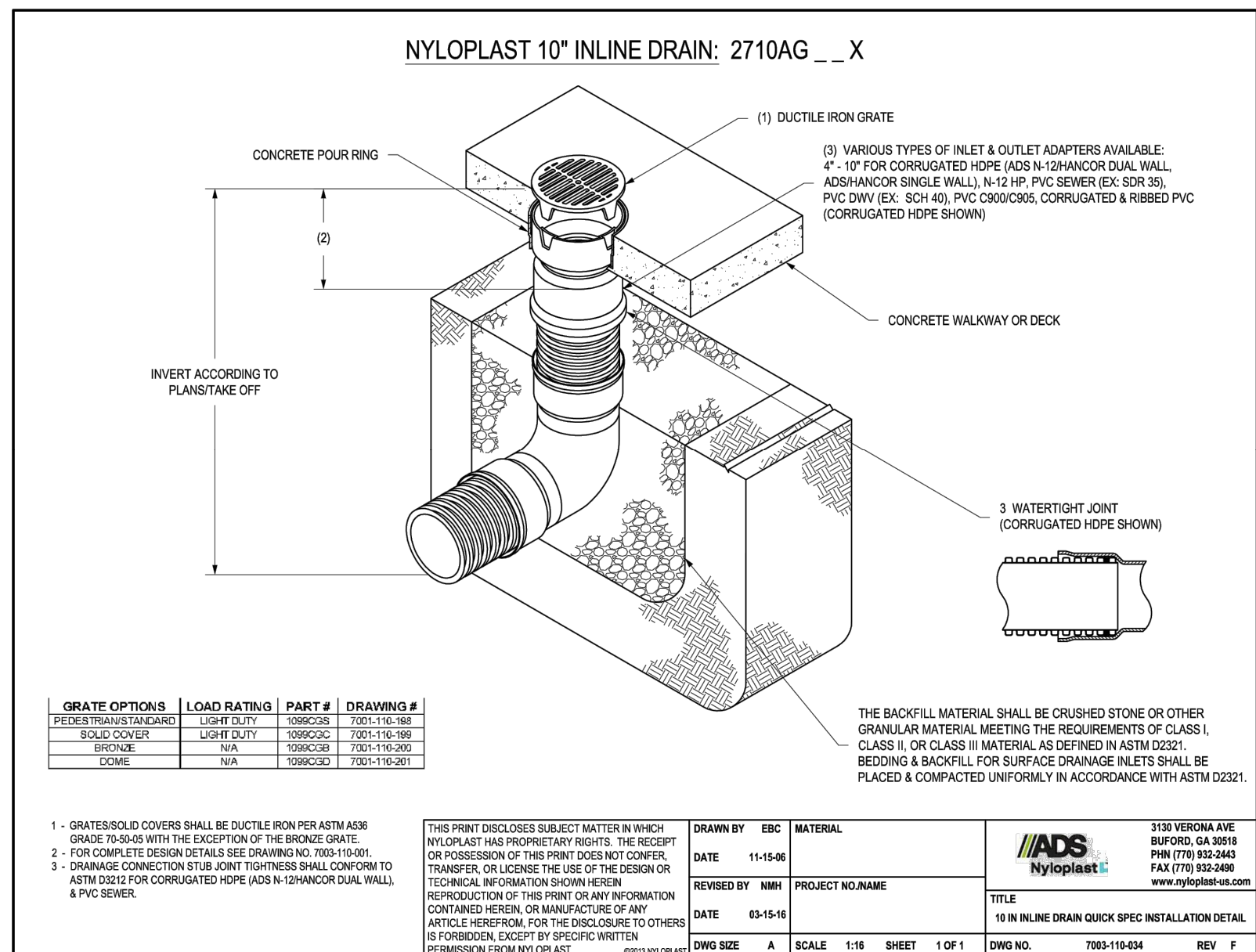
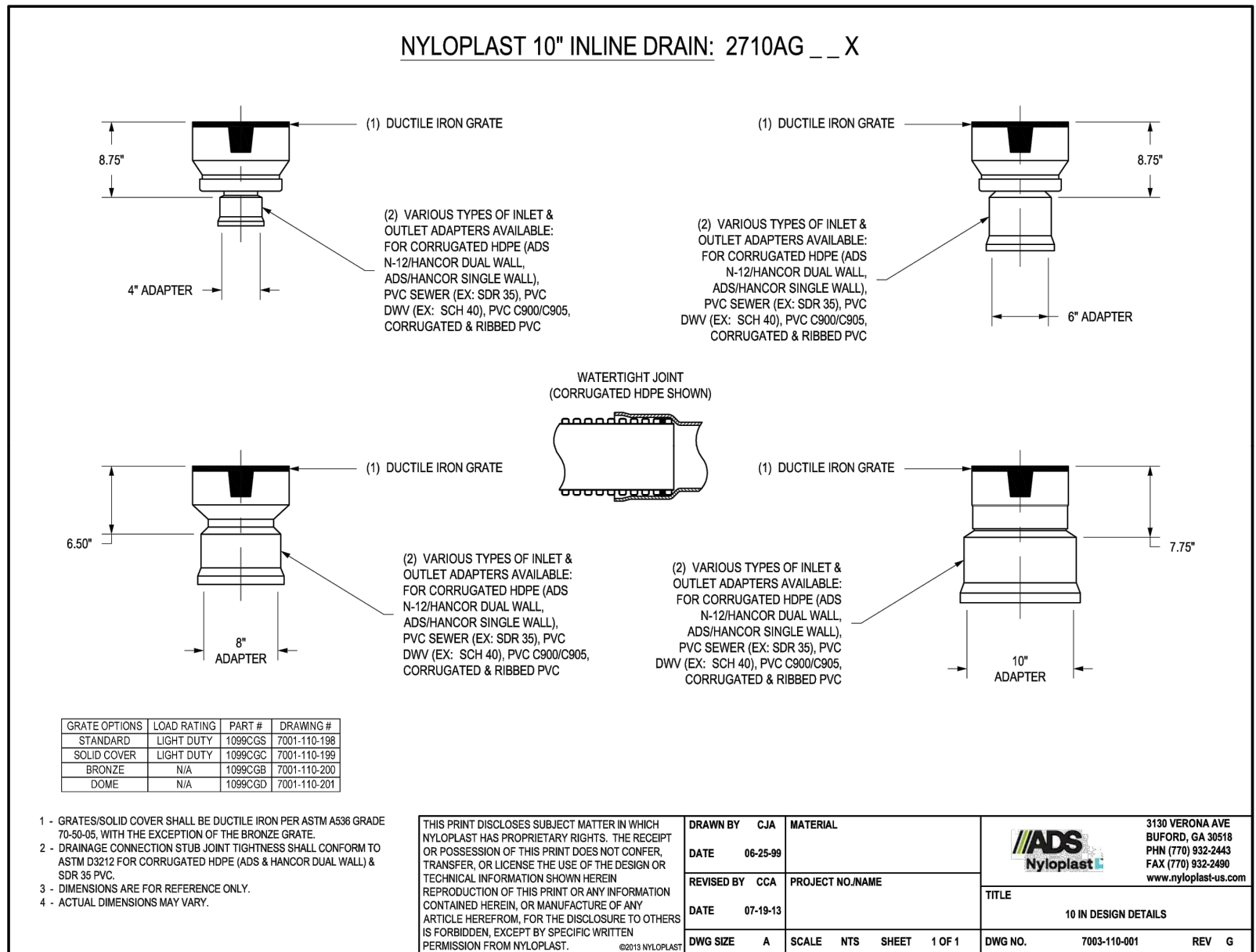
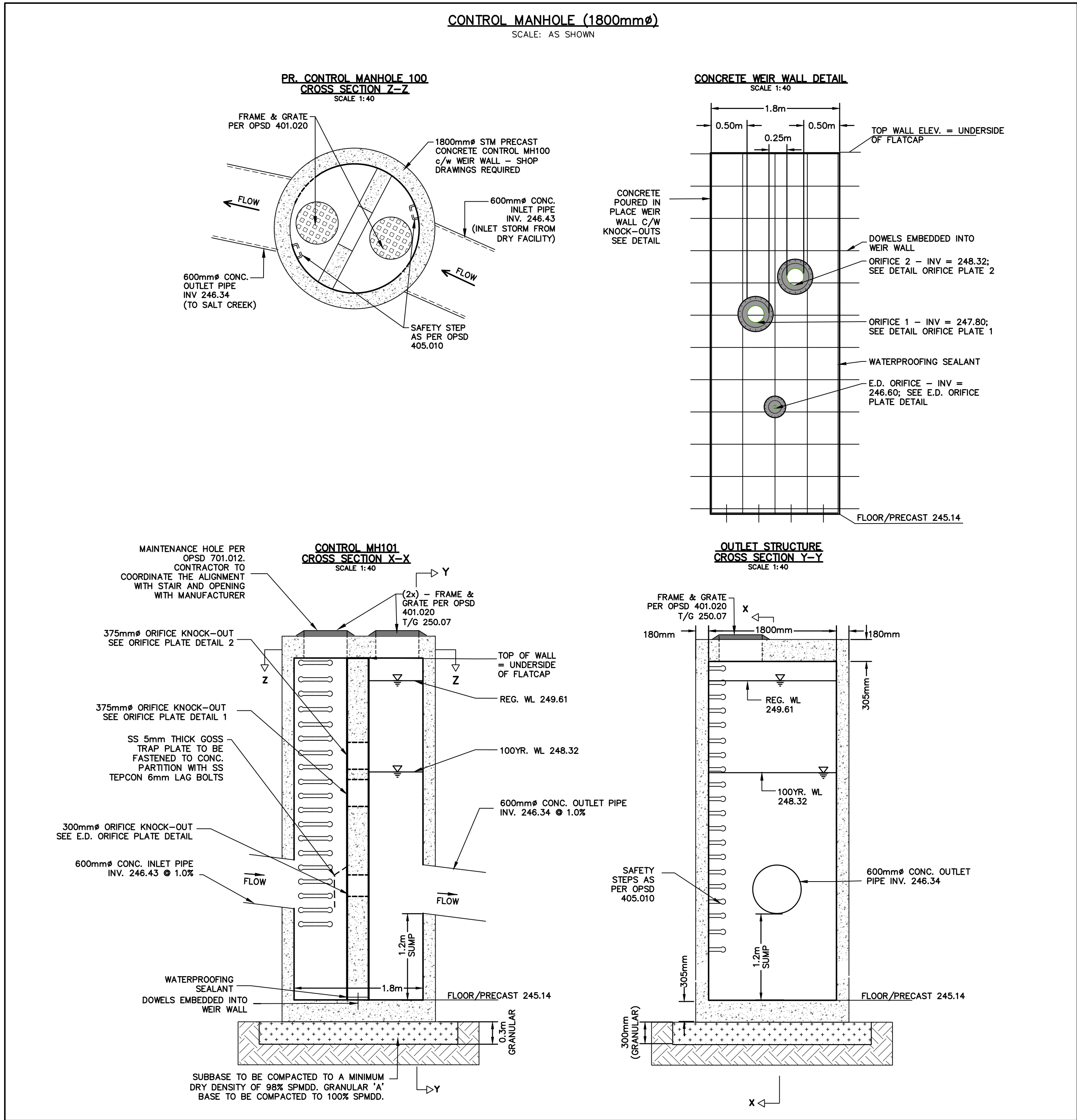
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Project	0 AIRPORT ROAD TOWN OF CALEDON
Drawing	JELLYFISH DETAIL

Drawn By	K.W./V.M.	Design By	K.W./V.M.	Scale	NA	Project	2278-7228
Check By	J.S.	Check By	J.S.	Sheet	12 OF 19	Drawing	C104 C





SHOP DRAWING RESPONSIBILITIES

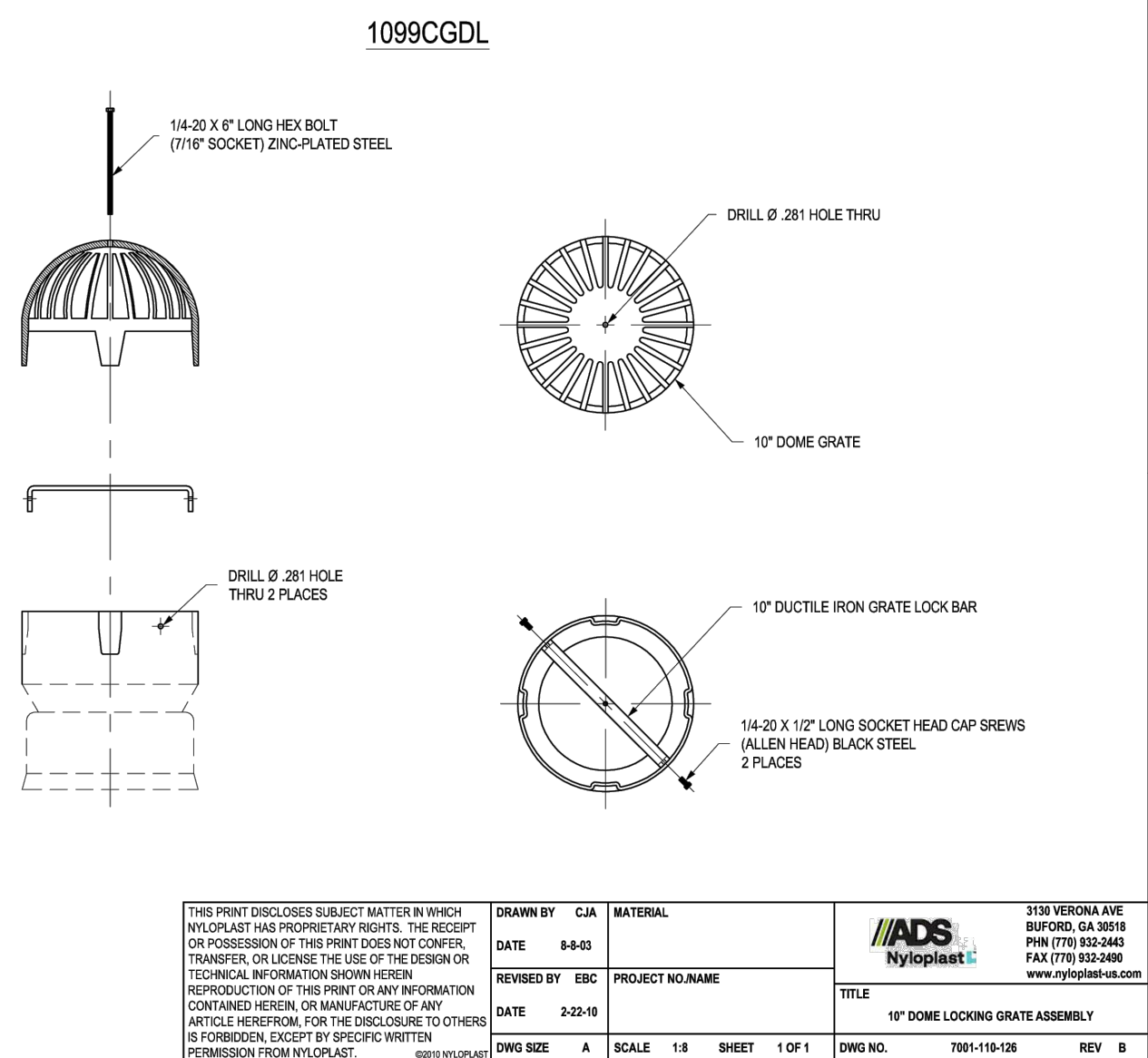
AS PART OF THEIR FIELD SERVICES, THE ENGINEER MAY REVIEW SHOP DRAWINGS PERTAINING TO WORK SHOWN ON THE ENGINEER'S DRAWINGS BY MEANS OF APPROPRIATE RATIONAL SAMPLING PROCEDURES AND COMMENT ON THE ACCURACY WITH WHICH THE CONTRACTOR PREPARED THE DRAWINGS. REVIEW OF SHOP DRAWINGS IS FOR THE SOLE PURPOSE OF ASCERTAINING CONFORMANCE WITH THE GENERAL DESIGN CONCEPT AND IS NOT AN APPROVAL OF THE DETAIL DESIGN INHERENT IN THE SHOP DRAWINGS. RESPONSIBILITY FOR WHICH SHALL REMAIN WITH THE CONTRACTOR SUBMITTING THEM. SUCH REVIEW SHALL NOT RELIEVE THE CONTRACTOR OF HIS RESPONSIBILITY FOR ERRORS AND OMISSIONS IN THE SHOP DRAWINGS OR FOR MEETING ALL REQUIREMENTS OF THE CONTRACT DOCUMENTS.

THE CONTRACTOR IS SOLELY RESPONSIBLE FOR INFORMATION PERTAINING TO THE FABRICATION PROCESS TECHNIQUES OF CONSTRUCTION AND INSTALLATION, AND FOR CO-ORDINATION OF THE WORK OF ALL SUB-TRADES.

- MANUFACTURERS OF ALL STRUCTURAL ELEMENTS SHALL SUBMIT COMPLETE SHOP DRAWINGS, SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN ONTARIO TO THE CONSULTANT FOR REVIEW PRIOR TO THE FABRICATION (3 SETS OR PDF). SUBMIT SHOP DRAWINGS IN ACCORDANCE WITH SPECIFICATIONS AND ALLOW A MINIMUM OF TWO(2) WEEKS FOR REVIEW. THE SUBMISSION NOR THE REVIEW SHALL RELIEVE THE CONTRACTOR OF THEIR RESPONSIBILITY FOR PROVIDING PROPER ENGINEERING DESIGN, METHODS, EQUIPMENT, WORKMANSHIP AND SAFETY PROCEDURES.

- CONTRACTOR TO PROVIDE SHOP DRAWINGS FOR THE FOLLOWING:

- CONCRETE REINFORCING BAR
- CONCRETE MIX DESIGN
- CONCRETE PARTITION WALL
- KNOCK HOLES AND CUT OUT
- PRECAST BOX OUTLET STRUCTURE



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BENCHMARKS

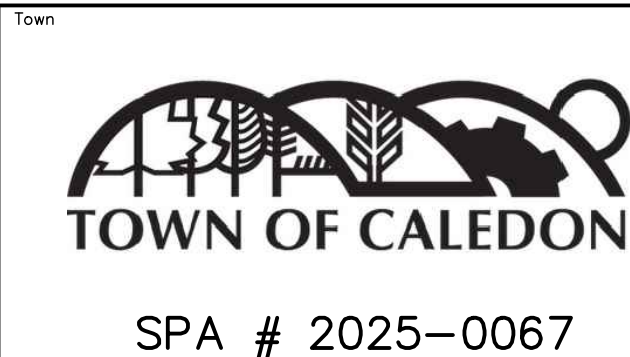
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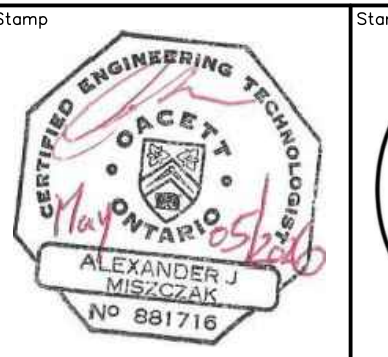
SURVEY COMPLETED BY R-PE SURVEYING LTD. (2022/OCC/12)

SITE PLAN NOTES

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Project

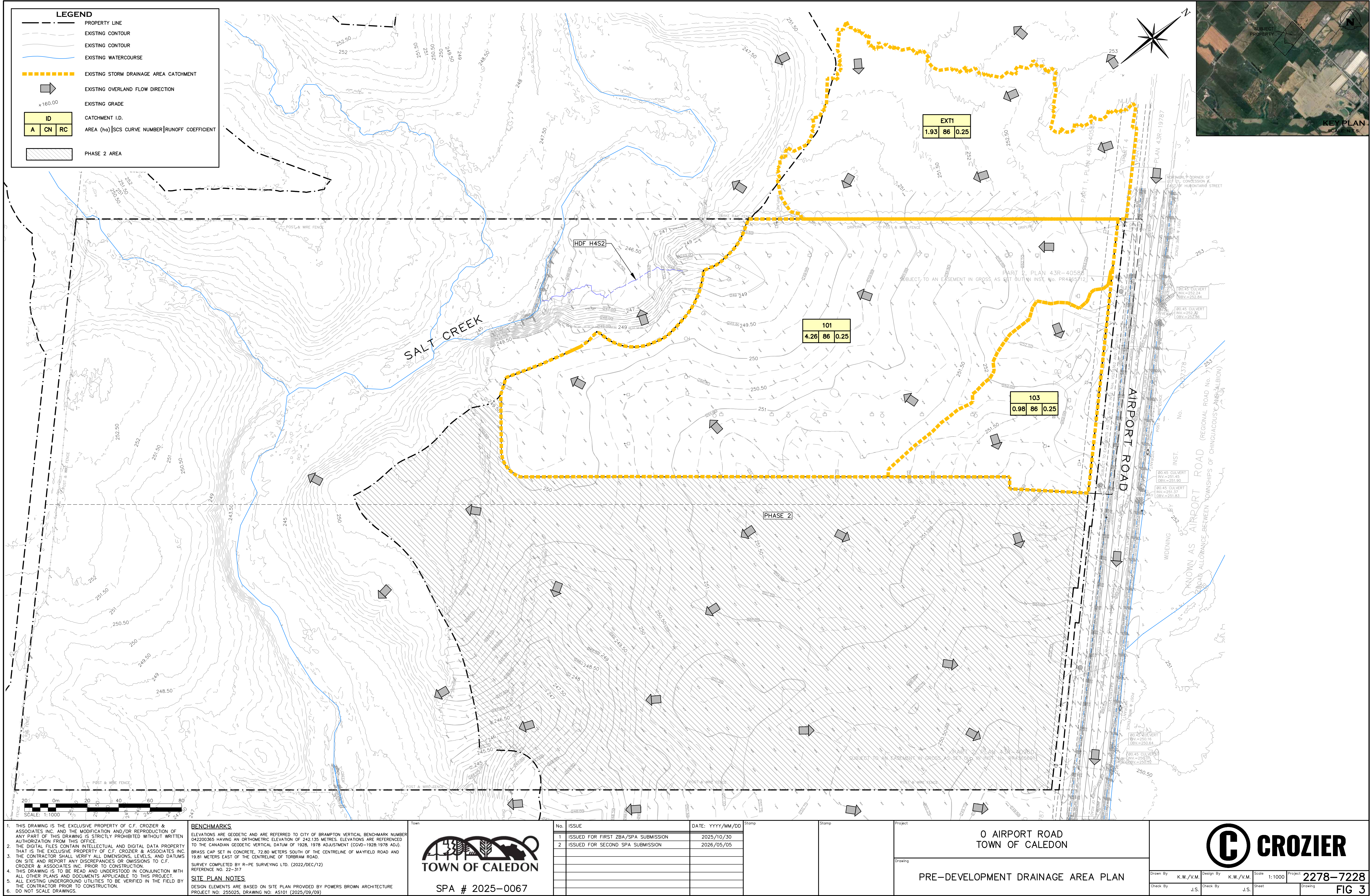
0 AIRPORT ROAD
TOWN OF CALEDON

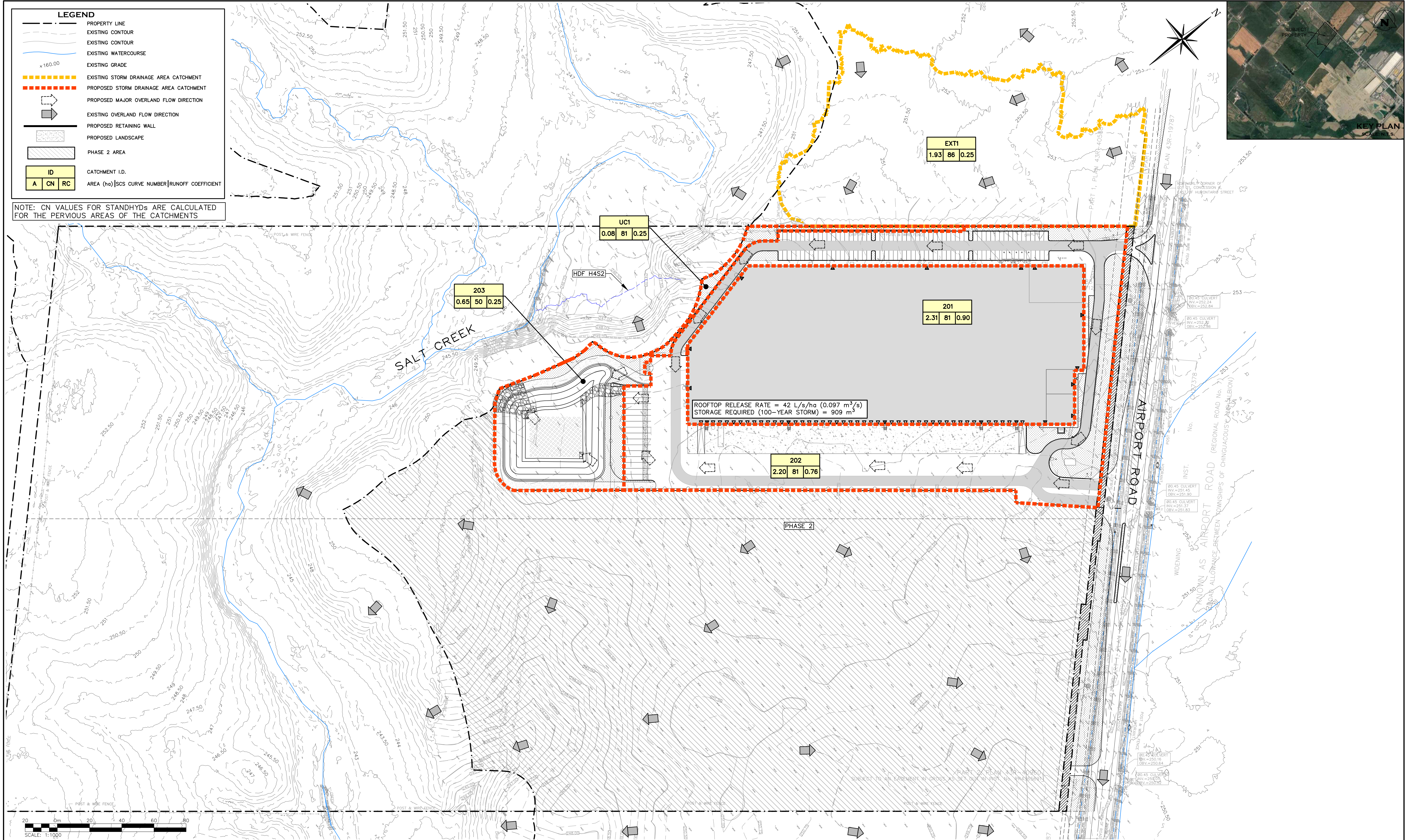
STORMWATER MANAGEMENT
DETAILS

Drawn By: A.J.M. Design By: A.J.M./J.F. Scale: AS SHOWN Project: 2278-7228

Check By: J.S. Check By: J.S. Sheet: 14 OF 18 Drawing: C106A

FIGURES





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Town



TOWN OF CALEDON

SPA # 2025-0067

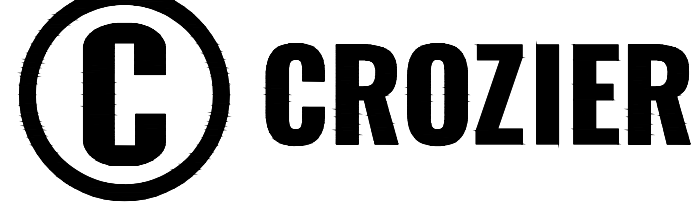
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Project

0 AIRPORT ROAD
TOWN OF CALEDON

Drawing

POST-DEVELOPMENT DRAINAGE AREA PLAN



CROZIER

Drawn By

K.W./V.M.

Design By

K.W./V.M.

Scale

1:1000

Project

2278-7228

Check By

J.S.

Check By

J.S.

Sheet

Drawing

FIG 4