STORMWATER MANAGEMENT REPORT

PROPOSED BARN AND PAVILION (OPEN WALLED STRUCUTRE)
NAVAN FAIR
1279 COLONIAL ROAD
NAVAN, CITY OF OTTAWA, ONTARIO

Prepared For:

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1 INTRODUCTION

Kollaard Associates was retained by Luc Picknell of the Cumberland Township Agricultural Society to complete a Stormwater Management Report for the proposed fairgrounds expansion, which will consist of a 372 square metre agricultural building (barn) and 1235 square metre pavilion (open walled structure/roof canopy). The proposed agricultural building will be 100 feet (30.48m) in length by 40 feet (12.20m) in width. The proposed pavilion will be 175 feet (53.34m) in length by 75 feet (22.86m) in width. Neither of the proposed structures will be serviced and will be considered to be “dry facilities.”

The report shall summarize the stormwater management (SWM) design requirements and proposed works that will address stormwater flows arising from the site under post-development conditions from both a quality and quantity perspective. The report will also describe any measures to be taken during construction to minimize erosion and sedimentation.

The site has a total area of 8.075 hectares and is currently occupied by various buildings and pavilions with associated gravel roadways and is in use by the Agricultural society. The development has been limited to a 3.879 hectare area based on the existing drainage patterns.

The subject site is accessed from Colonial Road to the north, Fairgreen Avenue to the west, and Delson Drive to the northeast. The proposed development is limited to the southern portion of the irregularly shaped parcel. Villeroy Crescent is south of the southern property line of the site, but is not accessible to the grounds.

The proposed building and pavilion will be accessed via existing gravel roadways. During events, the structures will primarily be accessed by pedestrian traffic.

2 SITE SERVICING DESIGN

2.1 Domestic Demand

As mentioned above, both of the proposed structures as part of this development are considered to be “dry Facilities” and do not have water service or sanitary services.

2.2 Fire Flow Calculations

The pre-consultation notes provided by the City of Ottawa on May 3, 2024 state that “... determination of the required fir flow, and confirm the size and location of any on-site storage existing or required. It is the responsibility of the owner to ensure that an adequate water supply for firefighting is provided.”

Following further consultation with the City of Ottawa, it was confirmed by Chief Larry Roy of Ottawa Fire Services that additional water storage for firefighting purposes is **NOT** required to facilitate the proposed development. A copy of the communication from the City of Ottawa is included in Appendix D.

3 STORMWATER DESIGN

3.1 Background

The subject property for the proposed development is located along the south side of Colonial Road and the southwest side of Delson Drive. The existing ground surface is in general relatively level with a slightly higher elevation at the west end of development sloping towards the northeast. The catchment area in which the proposed structures are to be located primarily drains towards the northeast towards the roadside ditch of Delson Drive. There are also low areas, which do not drain which were observed when reviewing the topographic survey.

Runoff originating from the development area is directed to the northeast to the roadside ditch along Delson Drive.

Existing drainage patterns outside of the development area drain towards the remaining property lines via overland flow. The runoff towards the south drains to the roadside ditch of Villeroy Crescent. The runoff draining towards the north is intersected by catch basins within the existing storm network. Drainage to the west drains towards the property line via overland flow towards the property line, where it is intersected by catch basins. The existing drainage patterns outside of the development area are to remain unchanged and are outside of the scope of this stormwater management design.

The subsurface conditions at the site consist of a thin veneer of topsoil overlying fine to medium grained sand, transitioning to glacial till at depths of between n 0.50 m and 0.75 m depth. During permeameter testing (to be discussed in section 2.3.12), two test holes were advanced to depths of 0.40 m below the ground surface. Groundwater was not encountered in either test hole on June 25, 2024. The ground surface of the site is primarily covered with mowed grass and various trees. No plant species associated with high groundwater were observed on the site. This would indicate that the soil conditions are well draining and not subject to prolonged periods of ponding.

3.2 Stormwater Management Design Criteria

Design of the storm system was completed in conformance with the City of Ottawa design requirements as well as with the Ministry of Environment (MOE) Stormwater Management Planning and Design Manual (March 2003).

Quantity and quality control criteria were provided by the City of Ottawa.

The criteria are as follows:

- The post-development flow rates from the site are to ensure that runoff during the 100-year post development peak flow rate must less than or equal to the 2-year storm pre-development runoff rates.
- The pre-development runoff coefficient or a maximum equivalent ‘C’ of 0.5, whichever is less
- A calculated time of concentration cannot be less than 10 minutes.
- Runoff volume control should proceed with the following hierarchical order, with each step exhausted before proceeding to the next
    - Retention (infiltration, reuse, or evapotranspiration);
    - Low Impact Development Filtration, and;
    - Conventional Stormwater Management.
- An enhanced level of treatment to be provided for runoff from the site, corresponding to 80% total suspended solids removal.

3.2.1 Low Impact Development (LID)

The MECP LID Guide provides the following guidance with respect to Low Impact Design: Low impact development begins with the application of the principles of ‘better site design’ or best management practices. From a stormwater management perspective, better site design involves considering site-level opportunities and constraints to stormwater management infrastructure from the beginning of the site design process. While not all of the techniques will apply to every development, the goal is to apply as many of them as possible to maximize stormwater reduction benefits before the use of structural LID best management practice.
Better site design techniques applicable to the propose development include:

- Preserving natural areas and natural area conservation;
- Stream and shoreline buffers;
- Disconnecting and distributing runoff;
- Disconnection of surface impervious cover;
- Rooftop disconnection;
- Disconnection of foundation drainage disposal from a municipal stormwater collection system;

- Reduced lot grading;
- Reduced swale slopes and increased swale cross sections where possible;

The BMPs are intended to reduce flow rates and promote filtration and the removal of sediments.

3.3 Stormwater Quantity Control

3.3.1 Methodology

Peak Flow for runoff quantities for the pre-development and post-development stages of the project were calculated using the rational method. The rational method is a common and straightforward calculation, which assumes that the entire drainage area is subject to uniformly distributed rainfall. The formula is:

$$Q = \frac{CiA}{360}$$

Where
Q is the Peak runoff measured in ***m^3/s***
C is the Runoff Coefficient, **Dimensionless**
A is the runoff area in ***hectares***
i is the storm intensity measure in ***mm/hr***

All values for intensity, i, for this project were derived from IDF curves provided by the City of Ottawa for data collected at the Ottawa International airport. For this project three return periods were considered, 2, 5 and 100-year events. The formulas for each are:

2-Year Event

$$i = \frac{732.951}{(t_c + 6.199)^{0.810}}$$

5-Year Event

$$i = \frac{998.071}{(t_c + 6.053)^{0.814}}$$

100-Year Event

$$i = \frac{1735.688}{(t_c + 6.014)^{0.82}}$$

Where
t_c is time of concentration in ***min***
i is the storm intensity measure in ***mm/hr***

3.3.2 Runoff Coefficients

Runoff coefficients for impervious surfaces (roofs, walkways and asphalt) were taken as 0.90, whereas pervious surfaces (grass) were taken as 0.20. Gravel areas have a runoff coefficient of 0.7 due to its highly compacted nature on the gravel roadways.

A 25% increase, to a maximum of 1.0 was used for the post development 100-year runoff coefficients. Refer to Appendix A for pre-development and post-development runoff coefficients.

3.3.3 Pre-development Conditions

As previously indicated the site has a total area of 8.075 hectares. The proposed development has been limited to a single catchment area of 3.879 hectares. The catchment area is currently occupied by four metal sided buildings, as well as 2 existing pavilions. The remaining buildings and pavilions on the site are outside of the catchment area and thus were not included in the stormwater management design. The location of the proposed barn is currently occupied by a concrete pad, which is to be retrofitted/ replaced to facilitate the proposed structure. The area of the proposed open sided pavilion is currently occupied by a compacted gravel pad.

The existing drainage patterns within the catchment area are via overland flow towards the east to the roadside ditch along the west side of Delson Drive with the exception of a small area immediately south of the gravel pad. The ground surface immediately south of the existing gravel pad, to be utilised for the proposed pavilion is currently a low area, which accepts runoff from the immediate areas and has no outlet. Following storm events, short term ponding is assumed to take place in this location and infiltrate into the native sand below. This area will be re-graded as part of the proposed development to ensure that no ponding takes place following the construction of the proposed structures.

Drawing 240297-PRE shows the pre-development conditions.

3.3.4 Pre-development Runoff Coefficient

The existing ground cover consisted of manicured grass, gravel roadways and various existing metal sided buildings. A detailed breakdown of the surface areas and 'C' value calculations can be found in Appendix A. The 5 year pre-development runoff coefficient was calculated to be 0.36 and the 100-year pre-development runoff coefficient is 0.44.

3.3.5 Pre-development Time of Concentration

The pre-development time of concentration was calculated using the airport method. The equation for the airport method is as follows:

$$tc = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S^{0.33}}$$

Where
tc is time of concentration ***min***
C is Rational Method runoff coefficient 0.36
L= is Flow length **m**
S= catchment slope **%**

Using an average slope of 1.2% and a most distant flow path length of 182m, the time of concentration was calculated to be 30.39 min and was rounded to 35 minutes as a conservative estimate.

3.3.6 Pre-development Flow Rate

The storm intensities calculated using a time of concentration of 35 minutes and the IDF curve equations previously provided yielded 36.06 mm/hr for a 2 year storm, 48.52 mm/hr for a 5 year storm and 82.58mm/hr for a 100 year storm. Using the Rational Method with the previously calculated runoff coefficients and these storm intensities, the pre-development runoff rates for the 2-year, 5-year and 100- year storms are as follows:

2 year $_{\text{pre-development}}$= 2.78 x 0.36 x 36.06 x 3.879 = 140.0 L/s
5 year $_{\text{pre-development}}$= 2.78 x 0.36 x 48.52 x 3.879 = 188.4 L/s
100 year $_{\text{pre-development}}$ = 2.78 x 0.44 x 82.58 x 3.879 = 391.8 L/s

3.3.7 Post-development Time of Concentration

A minimum time of concentration of 10 minutes is to be used for the post-development conditions.

3.3.8 Post-Development Site Conditions

For the purposes of this storm water management design, the site has been divided into uncontrolled and controlled areas as outlined in drawing 240297-POST (included in the appendix). Uncontrolled areas consist of areas from which runoff free flows directly off the site without restriction. Controlled areas consist of the areas from which runoff is directed by means of sheet flow, swales and subdrains to the storm water storage area from which discharge is restricted and released at a controlled rate.

The uncontrolled areas of the proposed development consist of thin strips of grassed area adjacent the property lines, as well as the portion of the gravel roadway, which currently drains to the surface of Delson Drive.

The controlled areas of the proposed development consist of the proposed and existing building areas, remaining gravel roadways and parking areas, and the landscaped areas within the catchment area defined in pre-development conditions. Runoff from this area will be directed by either sheet flow, swales, or subdrains to the proposed stormwater storage area located along the east side of the site adjacent the roadside ditch of Delson Drive.

The post-development site conditions are summarised for the site in the following Table 2-1.

Table 2-1 – Summary of Post-Development Site Conditions

POST-DEVELOPMENT

Description	Runoff Coefficient		Area (ha)
	5-year	100-year	
Controlled			3.809
Roof	0.90	1.00	0.426
Asphalt / Sidewalk	0.90	1.00	0
Grass	0.20	0.25	2.779
	0.70	0.88	0.604
Controlled Area Weighted Average C	*0.36*	*0.43*	
Controlled area Impervious including gravel (percentage)	*27%*		
Uncontrolled			0.069
Asphalt	0.90	1.00	0
Grass	0.20	0.25	0.052
Gravel	*0.70*	*0.88*	0.017
Uncontrolled Area Weighted Average C	*0.32*	*0.41*	

3.3.9 Post-Development Uncontrolled Area Runoff Rate

The uncontrolled runoff rate from the site consists of the flow that is directed off site from the areas without control or flow restriction. The time of concentration for the uncontrolled grass surfaced areas was taken as 10 minutes.

A post-development time of concentration of 10 minutes corresponds to a storm intensity of 76.81 mm/hr, 104.19mm/hr and 178.56mm/hr for the 2-year, 5-year and 100-year storm events respectively. Using the rational method and the above calculated runoff coefficients and rainfall intensities; the runoff rate for the combined uncontrolled areas of the site is calculated as follows:

$$Q_{2uncon} = 2.78 \times 0.32 \times 76.81 \times 0.069 = 4.7 \text{ L/sec.}$$
$$Q_{5uncon} = 2.78 \times 0.32 \times 104.19 \times 0.069 = 6.4 \text{ L/sec.}$$
$$Q_{100uncon} = 2.78 \times 0.41 \times 178.56 \times 0.069 = 14.1 \text{ L/sec.}$$

3.3.10 Post-Development Conveyance and Subdrain Design.

The runoff from the proposed barn and pavilion roof's is directed to the gravel surfaced areas adjacent the respective structures. The ground surface adjacent the proposed and existing structures slopes towards subdrained swales as shown on 240297-GR. The grading changes to the existing ground surface are to be minimal, with the major exception being the low area discussed above. This area is to be filled in to ensure that ponding does not occur in post development conditions adjacent the proposed pavilion. The subdrained swales connect to a perforated storm pipe, which will convey stormwater to the stormwater management area while simultaneously allow for infiltration within the native sand on the site.

The proposed subdrains will be placed within a clearstone trench, which has been proposed as a modified version of the City of Ottawa Standard Drawing S9 – "Ditched Pipe Storm Sewer Installation." The subdrains will have a longitudinal slope of 0.2% as an implementation of low impact development techniques.

Since the purpose of the subdrains is both collection and conveyance of a significant quantity of runoff, it is considered that the use of the subdrains at the site is more in keeping with a storm sewer system rather than a rear yard subdrain system. In the proposed facilities, the maintenance holes are located to facility access and maintenance of the perforated storm sewer comprising the subdrain rather than for inlet which is the purpose of the rear yard catch basins utilized in a rear yard subdrain system.

3.3.11 Implementation of LID Techniques for Quantity Control

Following the criteria for the site, and expanded on in section 2.2.1 above, Low Impact Development (LID) techniques have been implemented where possible. While the 0.2% longitudinal slope is less than what is typically proposed for conveyance of stormwater runoff, the proposed trenches in which the subdrains have been proposed are to be comprised entirely of clear stone to allow for additional short term storage areas in the event of high intensity storm events. The shallow longitudinal slope within the subdrains allows for a reduction in grading within the controlled area to promoted infiltration within the native sand, without surface ponding.

The disconnection of roof drainage and reduction in lot grading also allows for runoff from the roof areas of the proposed barn and pavilion to infiltrate in the adjacent ground surface prior to being intercepted by the stormwater management.

3.3.12 Permeability of Native Soils

Permeability testing was completed on the native sand materials within the areas of the proposed clear stone surfaced stormwater storage area along the northeast sides of the development area The test results are included in Appendix A. The test results indicate that the permeability K for the native soils at the site range from 1.74 x 10-^{4} cm/s to 1.86 x 10^{-4} cm/s. The average permeability K was taken as 1.81x 10^{-4} cm/s. The following table obtained from Appendix C of the CVC LID guide indicates the relationship between the Percolation Time, Coefficient of Permeability and Infiltration Rate.

Table C1: Approximate relationships between hydraulic conductivity, percolation time and infiltration rate

Hydraulic Conductivity, K_{fs} (centimetres/second)	Percolation Time, T (minutes/centimetre)	Infiltration Rate, 1/T (millimetres/hour)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

From the above comparison, the native soils at the site would have an estimated infiltration rate of 50 mm/hr and a Percolation Time T of 12 minutes.

It is acknowledged that Low Impact Design Guidelines recommend that a measured infiltration rate should be reduced by some factor prior to its use in infiltration design to account for the potential that the soil could be at capacity.

The permeability testing was completed on June 25th, 2024. Review of historical weather records of the Ottawa area from https://ottawa.weatherstats.ca/charts/precipitation-daily.html provide the following:

Date	*Precipitation (mm)*
June 24th, 2024	*15.4*
June 23rd, 2024	*33.3*
June 22nd, 2024	*3.2*
June 21st, 2024	*0*
June 20th, 2024	*1.4*
June 19th, 2024	*0*
June 18th, 2024	*0*
June 17th, 2024	*1.1*

Based on this historical rainfall data, the permeameter testing was completed immediately following three days of successive rainfall. The rainfall events included a rainfall which exceeded a typical quality storm event. It is considered reasonable to assume that the soils would have been near capacity following the successive rainfall events and would be as close to saturated as they could get. As such, it is considered reasonable to assume that the permeability results obtained during the testing on June 25 represent the infiltration rate of the soil at a near saturated condition.

3.3.13 Allowable Post-Development Discharge Rate from the Controlled Area.

In keeping with the stormwater management criteria, the total post-development runoff rate for storms up to and including the 100-year storm event is to be less than or equal to the pre-development runoff rate for the 2-year storm. The maximum allowable release rate from the controlled areas of the site is therefore equal to the total allowable runoff rate minus the runoff rate from the uncontrolled area of the site for each design storm event. The total allowable post-development runoff rate for the site is equal to the pre-development runoff rate.

The allowable post development runoff rates for the site are as follows:

$\mathbf{Q_{controlled} = Q_{total\ allowable} - Q_{uncontrolled}}$

For the 2-year Storm event

$\mathbf{Q_{controlled}}$= 140.0– 4.7 L/s = 135.3 L/s

For the 5-year Storm event

$\mathbf{Q_{controlled}}$= 140.0– 6.4 L/s = 133.6 L/s

For the 100-year Storm event
$\mathbf{Q_{controlled}}$= 140.0 – 14.1 L/s = 125.9 L/s

The allowable controlled area release rate for the site is summarized in Appendix A.

3.3.14 Post-Development Restricted Flow and Storage

Stormwater storage for the purposes of restricting the post-development runoff rates of storms up to and including the 100-year storm event to the 2-year pre-development storm runoff will be provided within a stormwater storage area located along the east portion of the site. The proposed storage area along the east side of the site has been designed in conjunction with the quality control criteria to ensure that both the quantity and quality control criteria will be met.

It is noted that the bottom of the stormwater storage area will be between 0.70m and 1.30m above the roadside ditch. This means that there will be sufficient outlet from the storage area following storm events.

The stormwater storage area has been designed as follows:

- The stormwater storage area has a flat bottom, with a bottom elevation of 81.90m. The bottom width of the storage area is 20.5m.
- A bottom length of 61.0 metres running parallel with the roadside ditch of Delson Drive.
- The stormwater storage area has been constructed with a depth of 0.45 metres and side slopes of 10H:1V on the northwest and south west side slopes respectively. The southeast side slope of the storage area is 7H:1V. The side slope of the northeast side of the stormwater storage area has side slopes of 4H:1V.
- The bottom and side slopes of the storage area will be sovered with 150mm of sandy topsoil and seeded with a grass mixture such as MTO mix.
- The storage area will range from at or slightly above the existing ground surface to as much as 0.45m below the existing ground surface. Should the proposed subgrade elevation along the northeast portion of the storage area be lower than the existing grade, the sand removed from the southwest portion of the storage area as well as sand removed from the subdrain trenches can be used to fill the northeast side of the storage area or the low area adjacent the subdrained swales..
- A berm has been proposed along the northeast end side of the storage area, to restrict stormwater runoff from discharging directly into the roadside ditch without control or treatment. The berm can be constructed of either imported fill materials or utilize existing native soils on site to build up the grade adjacent the property line. The top of the berm is to have a minimum elevation of 82.35m. The side slopes of the berm were assumed to have a maximum slope of 4 units horizontal to 1 unit vertical

- Discharge from the swale will be controlled by an outlet v-notch weir plate in a concrete weir wall located within the proposed berm. The v-notch weir will have a 120 degree angle and will be set an invert of 82.05 m.
- An overflow channel will be through a rectangular weir portion over top of the v-notch weir plate with a channel width of 1.00m at an invert elevation of 82.20m
- There will be a sand filter constructed within the proposed berm on the northeast side of the stormwater storage area. The filter will have a subgrade elevation of 81.90 metres and the top of the filter elevation is 82.05 m for a total height of 0.15m.
- The above mentioned berm will be constructed overtop of the sand filter. The filter will have a height of 0.15m and top width of about 1.40m. The side slopes of the filter will be 3H:1V to an approximate bottom width of 2.3m. Filter construction will be discussed further in section 3.4.1 below
- The side slopes of the berm within the storage area are to be surface with a minimum of 0.20m thickness rip-rap meeting the gradation requirements for R-10 rip-rap in accordance to OPSS 1004. The remaining side slopes are to be vegetated. The outside face of the berm, between the stormwater storage area and the roadside ditch is to be surfaced with a minimum of 0.20m thickness topsoil and vegetated. The outside face of the berm to a distance of 1.5m of either side of the outlet weir wall is to be surfaced with R-10 rip-rap in accordance to OPSS 1004. The proposed rip-rap will reinforce areas expected to experience concentrated flows and will mitigate any risks of scouring.
- Discharge from the storage area between the elevations of 81.90 m and 82.05m will be controlled by flow through the sand filter until the filter is overtopped. Once the filter is overtopped, it will flow through the proposed weir, and will continue to discharge through the filter.
- The stormwater storage area has been sized to ensure that the entire 15mm storm event is contained any will discharge solely by infiltration or flow through the filter.
- Since there is sufficient outlet following storm events, it is anticipated that the storage area should be able to drain empty between storm events during normal circumstances
- The proposed subdrains, which will outlet into the clear stone storage area are to be constructed in accordance with the City of Ottawa’s standard drawing S9 and modified as following:
    - The trench is to be 1.20m wide
    - The trench is to be 1.00m deep
    - The side slopes of the excavation within the subdrained swales were assumed to be near vertical. If the excavation is sloped, it will provide additional storage area for runoff.
    - A 250mm diameter perforated pipe in filter sock is to be placed 150mm above the bottom of the trench
    - The trench is to be backfilled completely with clear stone to the ground surface

- The longitudinal slope of the subdrain is 0.2% to promote more infiltration of stormwater runoff prior to discharging into the storage facility.
- Rear Yard Catch Basin (RYCB) – ADS Nyloplast drain basins (product number 2812AG) complete with solid cover meeting H-20 (product number 1299) are to be installed in the locations as shown on drawing 240297-GR and are to be utilized for inspection and cleanout ports. Product information can be found in Appendix B.
- The outlet of the perforated subdrains is to the ground surface of the clear stone storage area at an invert of 82.05m
- The clear stone within the trenches has been included in the storage and infiltration calculation for trenches with elevations between 82.05m and 82.35m.
- The area of the trenches has been included for infiltration purposes only above elevations of 82.05
- The portion of the subdrain which intersects the gravel access road is to consist of a solid 250mm diameter HDPE R320 pipe. Details for the gravel structure can be seen on 240297-GR

The physical characteristics of the stormwater storage area and outlet control will result in the stage - storage - discharge relationship as indicated in the following Table 2-2

It is noted that the MECP SWPDM requires a minimum of 1 metres of separation between the groundwater level and the bottom of an infiltration facility when considering either infiltration trenches or basins. The high groundwater level was determined to be greater than 1.0 metres below the bottom of the infiltration basin based on the results of test pits put down at the site and the relative elevation of the infiltration basin in comparison to the adjacent roadside ditch.

Due to the relatively high permeability of the subsurface soils at the site as determined by the permeability testing, significant mounding and deviation of the groundwater level across the site is not expected. As such, the high groundwater level will be controlled by the relatively lower elevations of the adjacent ditch.

The bottom of the infiltration basin has a minimum elevation of 81.90 m. The invert of the roadside ditch varies from 80.65 to 80.31 adjacent the site and continues to decrease to 81.16 m north of the storage area. The bottom of the infiltration elevation of the infiltration trenches increases with distance from the ditch increasing the separation. Since the bottom roadside ditch is more than 1.3 metres below the immediately adjacent proposed infiltration basin, it is expected that the high ground water level will be more than 1.0 metres below the infiltration basin.

It is further noted that the proposed design does not solely rely on infiltration. The infiltration trenches and basin have been designed to lower the impact of the development. The predominate quality control mechanism is by means of filtration by means of horizontal flow through an imported sand filter. As discussed in section 3.4 there is sufficient volume within the infiltration basin to retain the entire volume of runoff from the quality storm event. If there is a short period of time with an elevated ground water level which was to reduce the infiltration rate, the facility as designed would continue to provide stormwater management in terms of quality and quantity control. The impact of the elevated groundwater would be to temporarily reduce the effectiveness of the low impact design techniques implemented. For these reasons, it is considered that the determination that the high ground water level is greater than 1.0 metres below the infiltration trenches and basins is a reasonable and conservative assumption.

Table 2-2 – Elevation, Storage and Discharge Relations

Stage (Elevation) m	Quality Storage Volume m^3	Quantity Storage Volume m^3	Infiltration L/sec	Overflow Channel Flow L/sec	Total Outflow L/sec	Discharge From Site L/sec
82.35		750.2	10.31	203.44	330.3	324.1
82.30		647.2	9.38	110.74	193.9	188.2
82.25		549.2	8.44	39.15	89.8	87.5
82.20		465.1	7.5	0	28.0	23.1
82.15		367.9	6.57		14.0	9.5
82.10		284.5	5.62		7.0	3.0
82.05	205.9	205.9	4.69		3.8	0.6
82.00	131.9	131.9	3.57		3.1	0.3
81.95	64.0	64.0	2.56		2.7	0.2
81.90	0	0	1.63		2.4	0

The modified rational method was utilized to determine the maximum storage requirement within the storage swale based on the above storage discharge relationship. The calculation tables are included in Appendix A.

From the calculation tables provided in Appendix A, the maximum discharge rates and storage requirements and ponding depths for the design storm are as summarized in the following Table 2-3.

Table 2-3 – Summary of Maximum Discharge Rate, Storage Requirement and Ponding Depth

Design Storm Event	Allowable Release Rate L/sec	Total Discharge Rate From Storage area[1] L/sec	Discharge From Site Through Outlet[2] L/sec	Storage Requirement m^3	Maximum Ponding Depth m	Available Storage[3] m^3
2 year	135.3	12	8	350	0.24	750.2
5 year	135.3	18	14	402	0.27	750.2
100 year	135.3	106	102	565	0.36	750.2

[1] Total Outflow from the storage area includes both outlet by infiltration, filter flow and the outlet to the roadside ditch by means of the outlet structure.
[2] Discharge from site includes only the portion of the flow exiting the storage area by means of the outlet structure and sand filter and being directed to the adjacent roadside ditch.
[3] The available storage considers the total storage within the stormwater storage area.

From the Calculations in Appendix A as summarized in the above table, the discharge rate from the storage swale to the adjacent roadside ditch will be less than the allowable release rate for each design storm event.

3.4 Stormwater Quality Control

The City of Ottawa requires an enhanced level of treatment for the site. An enhanced level of treatment corresponds to 80 percent total suspended solids removal.

Stormwater treatment of 80% TSS removal will be provided by a treatment train approach. The treatment train consists of sedimentation during sheet flow over existing vegetated areas towards the subdrained swales and stormwater storage area, followed by sedimentation within the clear stone area of the subdrained swales and settlement within the stormwater storage area, followed by filtration through a sand filter. Pre-treatment will be provided by best management practices. Quality Control will be provided by temporary detention of the entire quality control volume generated in the controlled area. A storage swale will be constructed adjacent to the northeast property line. The native sand below the swale will promote infiltration. A sand filter will be constructed across the northeast side of the clear stone surfaced stormwater storage area below the outlet to provide filtration for all of the quality control volume.

Water from CA1 will travel by sheet flow and subdrains to the storage area along the northeast side of the development area. The storage swale has been designed to outlet the quality storage volume by infiltration through the native sand along the sides and bottom of the storage area and by flow through the sand filter. As a conservative approach, only infiltration through the bottom of the storage area and subdrain trenches was considered for infiltration calculations.

The Ministry of Environment Stormwater Management Planning and Design Manual (March 2003) (MOE Manual) provides guidance on design for stormwater quality control. Quality control design is completed with the fundamental understanding that the majority of sediment and particulate pollutants are washed from the site surfaces during minor (frequent) storm events. Section 3.3.1 of the MOE Manual indicates that in most cases, quality control design storms range from 12.5 mm to 25 mm. The MOE Manual also indicates that an alternate approach to the volumetric sizing of stormwater facilities for quality control has been applied in Ontario. The alternate approach is summarized in Table 3.2 *Water Quality Storage Requirements Based on Receiving Waters.* Table 3.2 of the MOE manual specifies the storage volume required to achieve an enhanced minimum required quality control level of treatment using infiltration or filtration.

In Part 4, the MOE Manual details the design requirements of several types of end of pipe stormwater management facilities. The proposed stormwater management design for quality control will consist of filtration. Design guidance for filtration is provided in Part 4 Section 4.6.7 Filters of the MOE Manual.

Section 4.6.7 provides the design guidance with respect to the use of a filter as summarized in the table below. A column has been added to indicate how the proposed design conforms to the Criteria.

Design Element	Design Objective	Minimum Criteria	Design Conformance
Drainage Area		< 5 hectares	~ 3.81 hectares
Pre-treatment	Longevity	Pre-treatment by means of sedimentation chamber, or forebay, vegetated filter strip, swale or oil/grit separator	-Vegetative filtration on grass within the landscaped areas adjacent the subdrained swales and storage area. –Vegetative filtration and sedimentation across the bottom of the storage area upstream of the filter.
Storage Depth	Avoid Filter Compaction	Subsurface sand and organic filters: 0.5 m Maximum 1.0 m	-Maximum storage depth of 0.15m before filter overflow. Maximum total storage depth of 0.45m.
Filter Media Depth	Filtering	Sand: 0.5 m	-Sand Filter has a minimum thickness of 1.40 m at the top.

Under-drain	Discharge	Minimum 100 mm perforated pipes bedded in 150 – 300 mm of 50 mm gravel	-No Under-drain provided. -Horizontal Discharge.
Land use		any land use, often employed for commercial and industrial	-Agricultural and public use
Volumetric Sizing		Provided in Table 3.2 under infiltration. By-pass flows should not occur below a 4 hr 15 mm design event	-Quality storage volume sufficient to contain entire volume of a 15 mm storm event before by-pass
Filter Size		Determined using the Darcy Equation	-Determined using the Darcy Equation
Filter Lining	prevent clogging	liner to prevent native material from entering filter	-Non-woven geotextile filter cloth used between native sand filter and clearstone
Overflow / by-pass		required	-Overflow is provided above the Quantity storage requirement to ensure a maximum ponding depth
Drawdown time	prevent standing water	from 24 to 48 hours 24 hours preferred	Total drawdown time: -Following 2 year storm to drain empty: 23.8 hours -Following 5-year storm to drain empty: 24.4 hours -Follwing 100-year storm to drain empty: 25.2 hours

3.4.1 Volumetric Sizing and Filter Size.

The water quality storage volume requirement to achieve an enhanced level of treatment using the sand filter was determined from the MOE Manual Table 3.2 under infiltration. As previously calculated, the impervious ratio for the controlled area of the site is 27% when including gravel areas. From MOE Table 3.2, for a 35% impervious ratio at an enhanced level of treatment the storage requirement is 25 m^3/ha.

Catchment area CA1 has an area of 3.809 ha. 3.809 x 25 m^3/ha gives a quality storage requirement of 95.2 m^3.

The MOE Manual in section 4.6.7 under the heading Volumetric Sizing provides the following additional design guidance when using filtration for quality control:

"Water quality volumes to be used in the design are provided in Table 3.2 under the “infiltration” heading. Erosion and quantity control volumes are not applicable to this type of SWMP. The design should be such that at a minimum, the by-pass of flows should not occur below or at the peak runoff from a 4 hour 15 mm design event."

In order to ensure that by-pass would not occur below a 4 hr 15 mm design event, the clear stone surfaced stormwater storage area has been designed to accommodate the runoff volume of a 15 mm rainfall. It is noted that a runoff coefficient of 0.36 indicates that 36% of the rainfall will become runoff.

The MOE Manual indicates that the size of the filter be designed to ensure a specified volume is discharged within a specified time period using the Darcy Equation. The size of the filter and storage volume must be sufficient to ensure that no overflow or by-pass occurs below the 4 hr 15 mm design storm.

Catchment area CA1 has an area of 3.81ha. A 15mm storm event will result in a runoff volume of (3.81ha x 15mm x 0.36) = 205.7 m^3 for CA1. This results in a minimum quality storage requirement of 205.7 m^3. There is a total storage volume available for quality control purposes of 205.9 m^3 below the invert of the outlet weir and can only discharge by means of infiltration and filter flow.

As such the entire quality control volume required by the MOE Manual will be stored below the top of the sand filter and no by-pass or overtopping of the filter will occur below he 15mm storm event

The proposed filter has been sized based on the space available for the filter. The flow rate through the filter was calculated and the drawdown time was determined based on the volume of the quality storage in the catchment.

In CA1 the proposed filter will be constructed across the width of the storage area resulting in a length of between approximately 61 metres. The filter width will vary from 2.30m at the bottom to 1.40m at the top. The length of the filter was assumed to be constant as the water level increases.

The filter will be constructed with imported filter media sand having a percolation rate "T" time of 2 min/cm a coefficient of permeability of k = 0.1 cm/sec. Details for the sand filter have been illustrated on Kollaard Associates Inc. drawing #240297–GR. The surface of the filter will be covered with a 6 ounce/yd^2 non-woven geotextile filter fabric (such as Terrafix 360R or an approved alternative – Included for reference in Appendix B) followed by rip-rap on the northeast side of the filter to protect the filter from erosion. The southwest side of the filter will be backfilled with the required clear stone for the storage area. This fabric offers medium tensile strength at high elongation and good filtration, coupled with high permeability to allow for proper filtration, while holding the filter sand in place as designed.

3.4.2 Discharge Through Filters

The average flow rate through the sand filters was calculated using Darcy's Equation to be:
Q= A K i
A = the cross sectional area of the filter
K= coefficient of permeability
i= hydraulic gradient = head across the filter/ flow path across the filter

Calculations are provided in Appendix A.

Based on the discharge rates through the filter and by infiltration, it is expected that the draw down time in the swale for the quality control volume will be 18.8 hrs assuming the entire 15mm storm event is applied instantaniously.

3.5 Stormwater System Operation and Maintenance

3.5.1 Storm Sewers

As previously indicated, surface runoff from the catchment area will be directed by sheet flow and by subdrains to the stormwater storage area. Catch basins will be used for inspection ports and cleanouts for the subdrains which will consist of 250 mm diameter HDPE double wall pipe with smooth interior pipes. The catch basins should be inspected on a bi-monthly basis to remove debris. Sediment levels should be measured on an annual basis. When sediment builds up more than 0.3 metres it should be removed by hydrovac.

3.5.2 Stormwater Storage Area

The stormwater storage area should be inspected on a weekly basis and after any rain fall event after construction until vegetation is well established across the site to document whether any sediment is migrating towards the storage area. Any areas of erosion or distress to the re-graded should be repaired immediately. Any excessive sediment accumulation within the storage area should be removed and reported to the engineer to determine a remediation strategy.

Once vegetation of the re-graded areas is completed, it is not anticipated that any maintenance will be required for the stormwater storage area.

Inspect the storage area after large storm events and at least monthly for improper water drainage, berm settling, soil erosion, as well as vegetation growing within the clear stone area. Any debris and invasive plants should be removed from the storage if present.

It is anticipated that the grassed area adjacent the proposed stormwater storage area and adjacent the subdrained trenches will be disturbed as a result of the proposed development. Following the completion of construction, these areas should be reseeded to re-establish grass grown within these areas. Always water plants (grass) throughout the first year after planting until the grassed areas are fully established. The grass should be able to tolerate wet and dry conditions on their own afterwards. However, should bare or thin grass areas be encountered, over seeding may be required.

If long term ponding occurs within the storage area, the engineer should be notified. At this point the engineer could make an assessment of the material in the upper portion of the filter. If the assessment indicates that the subdrains have become compromised with sediment, the swale and filter system will require maintenance. Sub-drains would need to be replaced, and surrounding clearstone may need to be replace or washed free of silt and sediment.

4 EROSION AND SEDIMENT CONTROL

An erosion and sediment control plan, appropriate to the site conditions and to the satisfaction of the City of Ottawa, will be implemented by the owner/contractor prior to undertaking any site alterations (filling, grading, removal of vegetation, etc.). The plan will be maintained during all phases of site preparation and construction in accordance with the current best management practices for erosion and sediment control. It is considered to be the owners and/or contractors responsibility to ensure that the erosion control measures are implemented and maintained.

In order to limit the amount of sediment carried in stormwater runoff from the site during construction, it is recommended to install a silt fence along the property line, as shown in Kollaard Associates Inc. Drawing #240297-ESC. The silt fence may be polypropylene, nylon, and polyester or ethylene yarn.

If a standard filter fabric is used, it must be backed by a wire fence supported on posts not over 2.0 m apart. Extra strength filter fabric may be used without a wire fence backing if posts are not over 1.0 m apart. Fabric joints should be lapped at least 150 mm (6") and stapled. The bottom edge of the filter fabric should be anchored in a 300 mm (1 ft) deep trench, to prevent flow under the fence. Sections of fence should be cleaned, if blocked with sediment and replaced if torn.

Construction activities should be timed to minimize the length of time that unprotected ground surfaces are exposed to erosive conditions. The proposed landscaping works should be completed as soon as possible. The proposed asphaltic concrete surfaced areas should be surfaced as soon as possible.

The silt fences should only be removed once the site is stabilized and landscaping is completed.

These measures will reduce the amount of sediment carried from the site during storm events that may occur during construction.

5 CONCLUSIONS

This report addresses stormwater management (SWM) design requirements and proposed works that will address stormwater flows arising from the site under post-development conditions for the proposed agricultural building and proposed pavilion. Based on the analysis provided in this report, the conclusions are as follows:

Stormwater from the structures, gravel and landscaping will be directed by means of sheet flow and subdrains to a stormwater storage area along the northeast side of the development area.

The post-development runoff for storms up to and including the 100-year storm event will be restricted to less than the pre-development runoff for the 2-year storm event by means of infiltration, flow through a sand filter and outlet control through a v-notch weir complete with overflow.

Quality control will be provided within the stormwater storage area by means of filtration through a sand filter, infiltration and vegetative filtration on the grass surfaces adjacent the storage area.

During all construction activities, erosion and sedimentation shall be controlled.

We trust that this report provides sufficient information for your present purposes. If you have any questions concerning this report please do not hesitate to contact our office.

Sincerely,
Kollaard Associates Inc.

Prepared by: Reviewed by:

LICENSED PROFESSIONAL ENGINEER
Aug 29, 2024
S.E. deWit
100079612
PROVINCE OF ONTARIO

Nick Recoskie, P.Eng. Steve deWit, P.Eng.

Appendix A: Storm Design Information

- Pre-Development Flows
- Uncontrolled Flow
- Catchment 1 Post-Development Required Storage and Release
- Catchment 1 Outlet Control Design Sheet
- Figure 1: Catchment 1 Discharge vs. Storage Curve
- Figure 2: Catchment 1 Elevation vs. Storage Curve
- Permeameter Test Results

APPENDIX A: STORMWATER MANAGEMENT MODEL
SHEET 1 - ALLOWABLE RELEASE RATE AND SWM SUMMARY

Client: Cumberland Township Agricultural Society
Job No.: 240297
Location: 1279 Colonial Road, Navan
Date: August 29, 2024

Pre Dev run-off Coefficient "C" — 2,5 year — 100 year

Area	Surface	Ha	C 2,5 year	C_{avg}	C 100 year	C_{avg}
Total	Gravel	0.806	0.70	**0.36**	0.88	**0.44**
3.879	Building	0.290	0.90		1.00	
	Driveway	0.000	0.90		1.00	
	Landscaping	2.783	0.20		0.25	
	Offsite Areas	0.000	0.20		0.25	

PRE DEVELOPMENT FLOW

2,5 Year Event

Pre Dev.	C	Intensity	Area
2 Year	0.36	36.06	3.879
2.78CIA= 139.99		L/s	
5 Year	0.36	48.52	3.879
2.78CIA= 188.35		L/s	

100 Year Event

Pre Dev.	C	Intensity	Area
100 Year	0.44	82.58	3.879
2.78CIA= 391.82		L/s	

**Use a 35 minute time of concentration for pre-development

Total Allowable Runoff Rate 2 year Event: 140.0 L/s — **100 year Event: 391.8 L/s**
5 year Event: 188.4 L/s

Pre Dev Time of Concentration "t_c"

Pre Dev Time of Concentration "t_c" — Airport Formula

$$t_{ca} = \frac{3.26\,x\,(1.1 - C)\,x\,l_c^{\,0.5}}{S^{0.33}}$$

C = Runoff Coefficient	0.36
Ic = length of flow path	182
Elevation Change	2.24
S = Slope of flow path	1.2

***t_c* = 30.39**

t_c **round to** **35** *min*

STORMWATER MANAGEMENT SUMMARY

Sub Area I.D.	Sub Area (ha)	5 year C	100 year C	Outlet Location	2 Year Controlled Release (L/s)	Required 2 year Storage (m^3)	5 Year Controlled Release (L/s)	Required 5 year Storage (m^3)	100 Year Controlled Release (L/s)	Required 100 year Storage (m^3)
Total Allowable Runoff Rate From Site					**140.0**		**140.0**		**140.0**	
Uncontrolled Runoff Rate from Site										
Uncont.	0.069	0.32	0.41		4.7		6.4		14.1	
Allowable Release Rate To Delson Drive										
					135.3		*133.6*		*125.9*	
Disharge Rate From Controlled Area to Delson Drive										
CA1	*3.809*				*8.0*	*350.0*	*14.0*	*402.0*	*102.0*	*565.0*
Summary - Total Post-Development Runoff Rate and Storage Requirement										
TOTAL	**3.878**				**12.7**		**20.4**		**116.1**	

Equations:

Flow Equation
Q = 2.78 x C x I x A
Where:
C is the runoff coefficient
I is the intensity of rainfall, City of Ottawa IDF
A is the total drainage area

Runoff Coefficient Equation
$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{tot}$

APPENDIX A: STORMWATER MANAGEMENT MODEL

Sheet2 - Uncontrolled Area Runoff Rate Calculation

Client: Cumberland Township Agricultural Society
Job No.: 240297
Location: 1279 Colonial Road, Navan
Date: August 29, 2024

UNCONTROLLED AREA DISCHARGE

Post Dev run-off Coefficient "C"

			5 Year Event		100 Year Event	
Area	**Surface**	**Ha**	**"C"**	**C_{avg}**	**"C"**	**C_{avg}**
Total	Gravel	0.0172	0.70	**0.32**	0.88	**0.41**
0.0692	Landscape	0.0520	0.20		0.25	
	Walkway	0.0000	0.90		1.00	

Impervious Area Ratio 0.25

Post Dev Free Flow

2 Year Event

	C	Intensity	Area
5 Year	0.32	76.81	0.0692
2.78CIA=	4.73		
4.7	L/S		

**Use a 10 minute time of concentration

5 Year Event

	C	Intensity	Area
5 Year	0.32	104.19	0.0692
2.78CIA=	6.41		
6.4	L/S		

100 Year Event

	C	Intensity	Area
100 Year	0.41	178.56	0.0692
2.78CIA=	14.08		
14.1	L/S		

**Use a 10 minute time of concentration

Equations:

Flow Equation

Q = 2.78 x C x I x A

Where:

C is the runoff coefficient

I is the intensity of rainfall, City of Ottawa IDF

A is the total drainage area

Runoff Coefficient Equation

$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{tot}$

APPENDIX A: STORMWATER MANAGEMENT MODEL

SHEET 3 - REQUIRED STORAGE VS. RELEASE RATE sheet 1 of 2

Client: Cumberland Township Agricultural Society
Job No.: 240297
Location: 1279 Colonial Road, Navan
Date: August 29, 2024

Post Dev run-off Coefficient "C" - CA1

			2,5 Year Event		100 Year Event	
Area (ha)	Surface	Area (ha)	"C"	C_{avg}	"C" x 1.25	$C_{100\ avg}$
Total	Roof	0.426	0.90	**0.36**	1.00	**0.43**
	Asphalt	0.000	0.90		1.00	
3.809	Gravel	0.604	0.70		0.88	
	Grass/Swale	2.779	0.20		0.25	

Impervious Ratio = 0.27

REQUIRED STORAGE VERSUS RELEASE RATE FOR 2 YEAR STORM

Runoff Coeffcient, C = **0.36** Duration Interval (min) = **20**
Drainage Area (ha) = **3.809** Release Rate Start (L/s) = **0**
Return Period (yrs) = **2** Release Rate Interval (L/s) = **20**

		Release Rate -->	**0**	**20**	**40**	**60**	**80**	**100**	**120**	**140**	**160**	**180**
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	**Storage Required (m^3)**									
0	167.22	637.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	52.03	198.3	238.0	214.0	190.0	166.0	142.0	118.0	94.0	70.0	46.0	22.0
40	32.86	125.3	300.7	252.7	204.7	156.7	108.7	60.7	12.7	-35.3	-83.3	-131.3
60	24.56	93.6	337.0	265.0	193.0	121.0	49.0	-23.0	-95.0	-167.0	-239.0	-311.0
80	19.83	75.6	362.8	266.8	170.8	74.8	-21.2	-117.2	-213.2	-309.2	-405.2	-501.2
100	16.75	63.8	383.0	263.0	143.0	23.0	-97.0	-217.0	-337.0	-457.0	-577.0	-697.0
120	14.56	55.5	399.7	255.7	111.7	-32.3	-176.3	-320.3	-464.3	-608.3	-752.3	-896.3
140	12.93	49.3	413.9	245.9	77.9	-90.1	-258.1	-426.1	-594.1	-762.1	-930.1	-1098.1
160	11.65	44.4	426.4	234.4	42.4	-149.6	-341.6	-533.6	-725.6	-917.6	-1109.6	-1301.6
180	10.63	40.5	437.5	221.5	5.5	-210.5	-426.5	-642.5	-858.5	-1074.5	-1290.5	-1506.5
200	9.78	37.3	447.6	207.6	-32.4	-272.4	-512.4	-752.4	-992.4	-1232.4	-1472.4	-1712.4
220	9.08	34.6	456.7	192.7	-71.3	-335.3	-599.3	-863.3	-1127.3	-1391.3	-1655.3	-1919.3
240	8.47	32.3	465.2	177.2	-110.8	-398.8	-686.8	-974.8	-1262.8	-1550.8	-1838.8	-2126.8
260	7.96	30.3	473.1	161.1	-150.9	-462.9	-774.9	-1086.9	-1398.9	-1710.9	-2022.9	-2334.9
280	7.50	28.6	480.4	144.4	-191.6	-527.6	-863.6	-1199.6	-1535.6	-1871.6	-2207.6	-2543.6
300	7.10	27.1	487.4	127.4	-232.6	-592.6	-952.6	-1312.6	-1672.6	-2032.6	-2392.6	-2752.6
320	6.75	25.7	493.9	109.9	-274.1	-658.1	-1042.1	-1426.1	-1810.1	-2194.1	-2578.1	-2962.1
340	6.43	24.5	500.0	92.0	-316.0	-724.0	-1132.0	-1540.0	-1948.0	-2356.0	-2764.0	-3172.0
360	6.14	23.4	505.9	73.9	-358.1	-790.1	-1222.1	-1654.1	-2086.1	-2518.1	-2950.1	-3382.1
380	5.89	22.4	511.5	55.5	-400.5	-856.5	-1312.5	-1768.5	-2224.5	-2680.5	-3136.5	-3592.5
Max. Storage Requirement =			511.5	266.8	204.7	166.0	142.0	118.0	94.0	70.0	46.0	22.0

REQUIRED STORAGE VERSUS RELEASE RATE FOR 5 YEAR STORM

Runoff Coeffcient, C = **0.36** Duration Interval (min) = **20**
Drainage Area (ha) = **3.809** Release Rate Start (L/s) = **0**
Return Period (yrs) = **5** Release Rate Interval (L/s) = **20**

		Release Rate -->	**0**	**20**	**40**	**60**	**80**	**100**	**120**	**140**	**160**	**180**
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	**Storage Required (m^3)**									
0	230.48	878.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	70.25	267.8	321.4	297.4	273.4	249.4	225.4	201.4	177.4	153.4	129.4	105.4
40	44.18	168.4	404.2	356.2	308.2	260.2	212.2	164.2	116.2	68.2	20.2	-27.8
60	32.94	125.6	452.1	380.1	308.1	236.1	164.1	92.1	20.1	-51.9	-123.9	-195.9
80	26.56	101.3	486.0	390.0	294.0	198.0	102.0	6.0	-90.0	-186.0	-282.0	-378.0
100	22.41	85.4	512.5	392.5	272.5	152.5	32.5	-87.5	-207.5	-327.5	-447.5	-567.5
120	19.47	74.2	534.3	390.3	246.3	102.3	-41.7	-185.7	-329.7	-473.7	-617.7	-761.7
140	17.27	65.8	553.0	385.0	217.0	49.0	-119.0	-287.0	-455.0	-623.0	-791.0	-959.0
160	15.56	59.3	569.3	377.3	185.3	-6.7	-198.7	-390.7	-582.7	-774.7	-966.7	-1158.7
180	14.18	54.1	583.8	367.8	151.8	-64.2	-280.2	-496.2	-712.2	-928.2	-1144.2	-1360.2
200	13.05	49.7	596.9	356.9	116.9	-123.1	-363.1	-603.1	-843.1	-1083.1	-1323.1	-1563.1
220	12.10	46.1	608.9	344.9	80.9	-183.1	-447.1	-711.1	-975.1	-1239.1	-1503.1	-1767.1
240	11.29	43.1	620.0	332.0	44.0	-244.0	-532.0	-820.0	-1108.0	-1396.0	-1684.0	-1972.0
260	10.60	40.4	630.3	318.3	6.3	-305.7	-617.7	-929.7	-1241.7	-1553.7	-1865.7	-2177.7
280	9.99	38.1	639.9	303.9	-32.1	-368.1	-704.1	-1040.1	-1376.1	-1712.1	-2048.1	-2384.1
300	9.46	36.0	648.9	288.9	-71.1	-431.1	-791.1	-1151.1	-1511.1	-1871.1	-2231.1	-2591.1
320	8.98	34.2	657.4	273.4	-110.6	-494.6	-878.6	-1262.6	-1646.6	-2030.6	-2414.6	-2798.6
340	8.56	32.6	665.4	257.4	-150.6	-558.6	-966.6	-1374.6	-1782.6	-2190.6	-2598.6	-3006.6
360	8.17	31.2	673.1	241.1	-190.9	-622.9	-1054.9	-1486.9	-1918.9	-2350.9	-2782.9	-3214.9
380	7.83	29.8	680.3	224.3	-231.7	-687.7	-1143.7	-1599.7	-2055.7	-2511.7	-2967.7	-3423.7
Max. Storage Requirement =			680.3	392.5	308.2	260.2	225.4	201.4	177.4	153.4	129.4	105.4

APPENDIX A: STORMWATER MANAGEMENT MODEL

SHEET 4 - REQUIRED STORAGE VS. RELEASE RATE sheet 2 of 2

Client: **Cumberland Township Agricultural Society**
Job No.: **240297**
Location: **1279 Colonial Road, Navan**
Date: **August 29, 2024**

REQUIRED STORAGE VERSUS RELEASE RATE FOR 100 YEAR STORM

Runoff Coeffcient, C =	**0.43**	Duration Interval (min) =	**20**
Drainage Area (ha) =	**3.809**	Release Rate Start (L/s) =	**0**
Return Period (yrs) =	**100**	Release Rate Interval (L/s) =	**20**

		Release Rate -->	**0**	**20**	**40**	**60**	**80**	**100**	**120**	**140**	**160**	**180**
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	**Storage Required (m^3)**									
0	398.62	1815.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	119.95	546.2	655.4	631.4	607.4	583.4	559.4	535.4	511.4	487.4	463.4	439.4
40	75.15	342.2	821.2	773.2	725.2	677.2	629.2	581.2	533.2	485.2	437.2	389.2
60	55.89	254.5	916.2	844.2	772.2	700.2	628.2	556.2	484.2	412.2	340.2	268.2
80	44.99	204.9	983.3	887.3	791.3	695.3	599.3	503.3	407.3	311.3	215.3	119.3
100	37.90	172.6	1035.5	915.5	795.5	675.5	555.5	435.5	315.5	195.5	75.5	-44.5
120	32.89	149.8	1078.4	934.4	790.4	646.4	502.4	358.4	214.4	70.4	-73.6	-217.6
140	29.15	132.7	1115.0	947.0	779.0	611.0	443.0	275.0	107.0	-61.0	-229.0	-397.0
160	26.24	119.5	1147.0	955.0	763.0	571.0	379.0	187.0	-5.0	-197.0	-389.0	-581.0
180	23.90	108.8	1175.4	959.4	743.4	527.4	311.4	95.4	-120.6	-336.6	-552.6	-768.6
200	21.98	100.1	1201.1	961.1	721.1	481.1	241.1	1.1	-238.9	-478.9	-718.9	-958.9
220	20.37	92.8	1224.6	960.6	696.6	432.6	168.6	-95.4	-359.4	-623.4	-887.4	-1151.4
240	19.01	86.5	1246.2	958.2	670.2	382.2	94.2	-193.8	-481.8	-769.8	-1057.8	-1345.8
260	17.83	81.2	1266.2	954.2	642.2	330.2	18.2	-293.8	-605.8	-917.8	-1229.8	-1541.8
280	16.80	76.5	1284.9	948.9	612.9	276.9	-59.1	-395.1	-731.1	-1067.1	-1403.1	-1739.1
300	15.89	72.4	1302.4	942.4	582.4	222.4	-137.6	-497.6	-857.6	-1217.6	-1577.6	-1937.6
320	15.09	68.7	1319.0	935.0	551.0	167.0	-217.0	-601.0	-985.0	-1369.0	-1753.0	-2137.0
340	14.37	65.4	1334.7	926.7	518.7	110.7	-297.3	-705.3	-1113.3	-1521.3	-1929.3	-2337.3
360	13.72	62.5	1349.5	917.5	485.5	53.5	-378.5	-810.5	-1242.5	-1674.5	-2106.5	-2538.5
380	13.14	59.8	1363.7	907.7	451.7	-4.3	-460.3	-916.3	-1372.3	-1828.3	-2284.3	-2740.3
400	12.60	57.4	1377.2	897.2	417.2	-62.8	-542.8	-1022.8	-1502.8	-1982.8	-2462.8	-2942.8
Max. Storage Requirement =			1377.2	961.1	795.5	700.2	629.2	581.2	533.2	487.4	463.4	439.4

Kollaard Associates
Engineers
210 Prescott Street, Unit 1
P.O. Box 189
Kemptville, Ontario K0G 1J0

APPENDIX A: STORMWATER MANAGEMENT MODEL

SHEET 5 - OUTLET CONTROL DESIGN SHEET - SWALE

Client: **Cumberland Township Agricultural Society**
Job No.: **240297**
Location: **1279 Colonial Road, Navan**
Date: **August 29, 2024**

Infiltration Information			**Filter Information**		
Percolation Time T =	12	min/cm	Percolation Time T =	2	min/cm
Infiltration Rate =	50	mm/hr	Percolation Rate =	3600	mm/hr
Permeability k =	1.82E-06	m/s	Permeability k =	1.0E-03	m/s
Depth of Layer =	1		Thickness of Filter at Top =	1.4	

Overflow Channel	
Bottom Channel Width (m):	1.20
Channel Invert (m):	82.20

Outlet Weir	
	V-Notch
Weir Invert (m):	82.05
Notch Angle	120

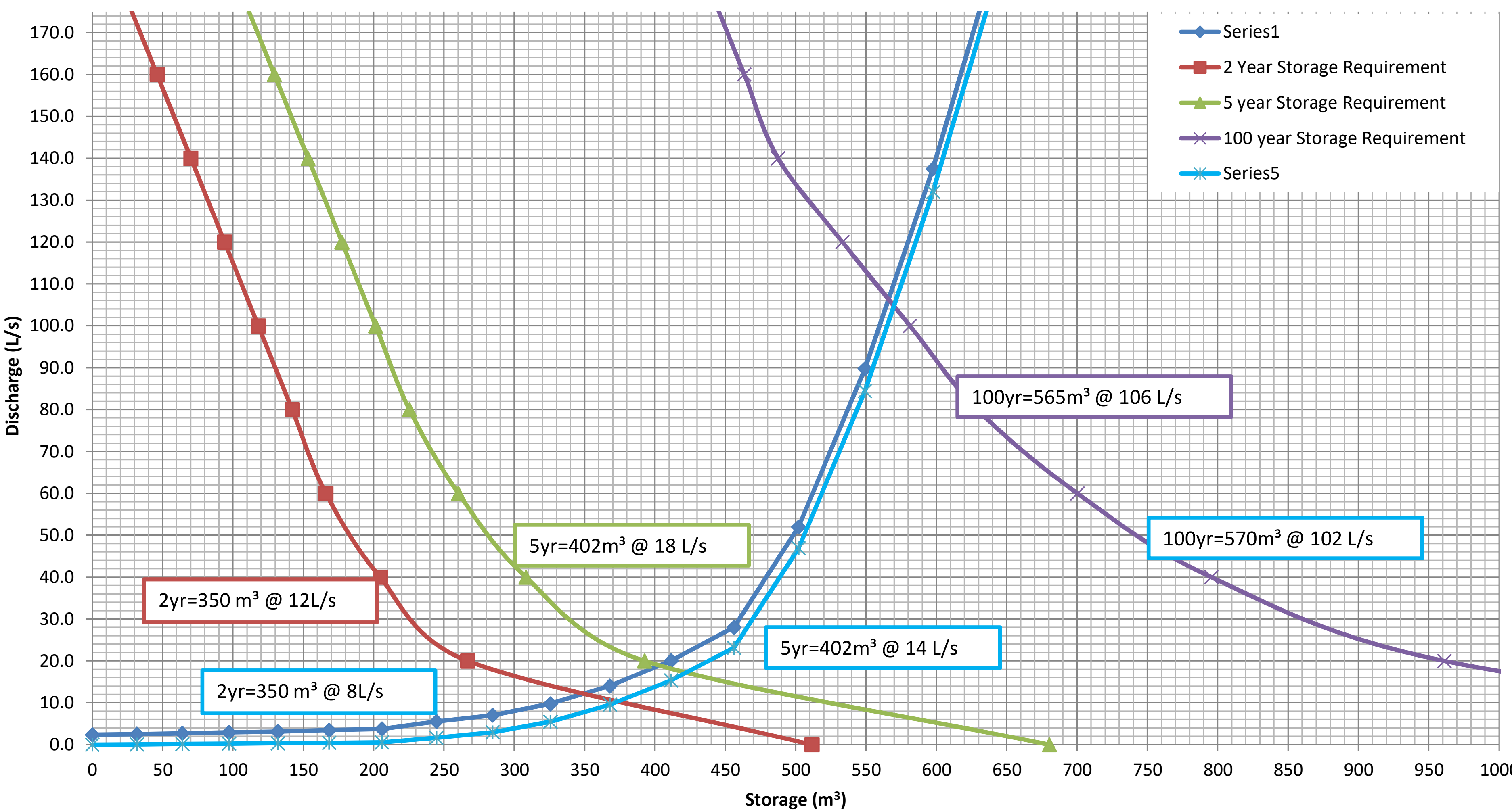
								Infiltration			Filter Flow				Overflow Channel		Weir Flow						
Stage, WSE Elev (m)	Comments	Layer Thickness (m)	Top Layer Area (m²)	Bottom Layer Area (m²)	Volume in Swale (m^3)	Quality Storage Volume (m3)	Quantity Storage (m3)	Head* (m)	Hydraulic Gradient	Infiltration Rate (m^3/sec)	Filter Thickness (m)	Head* (m)	Hydraulic Gradient	Filter Flow (m^3/sec)	Head (m)	Overflow (m^3/sec)	Head (ft)	cw	Weir Flow* (m^3/sec)	Total Outflow (m^3/sec)	Total Outflow (L/sec)	Discharge from Site (L/sec)	Draw Down Time (hrs)
82.35		0.025	2109.8	2059.6	52.1	0.0	750.2	0.45	1.5	0.006215	0.0	0.45	0.39	0.0037	0.15	0.2034	0.98	0.58	0.1170	0.3303	330.3	324.1	0.0
82.33		0.025	2059.6	2009.6	50.9	0.0	698.1	0.43	1.4	0.005977	0.0	0.43	0.37	0.0035	0.13	0.1548	0.90	0.58	0.0941	0.2583	258.3	252.4	0.1
82.30		0.025	2009.6	1959.9	49.6	0.0	647.2	0.40	1.4	0.005746	0.0	0.40	0.35	0.0033	0.10	0.1107	0.82	0.58	0.0742	0.1939	193.9	188.2	0.1
82.28		0.025	1959.9	1910.5	48.4	0.0	597.6	0.38	1.4	0.005519	0.0	0.38	0.33	0.0031	0.08	0.0719	0.74	0.58	0.0570	0.1375	137.5	132.0	0.1
82.25	Approx. 100-yr Storm	0.025	1910.5	1861.5	47.1	0.0	549.2	0.35	1.4	0.005298	0.0	0.35	0.30	0.0029	0.05	0.0392	0.66	0.58	0.0425	0.0898	89.8	84.5	0.1
82.23		0.025	1861.5	1812.7	45.9	0.0	502.1	0.33	1.3	0.005081	0.0	0.33	0.28	0.0026	0.03	0.0138	0.57	0.58	0.0304	0.0520	52.0	46.9	0.2
82.20		0.025	1812.7	1764.2	44.7	0.0	456.1	0.30	1.3	0.004870	0.0	0.30	0.26	0.0024	0.00	0.0000	0.49	0.58	0.0207	0.0280	28.0	23.1	0.4
82.18	Approx. 5-yr Storm	0.025	1764.2	1716.0	43.5	0.0	411.4	0.28	1.3	0.004664	0.0	0.28	0.24	0.0022			0.41	0.58	0.0131	0.0200	20.0	15.4	0.6
82.15	Approx. 2-year	0.025	1716.0	1668.1	42.3	0.0	367.9	0.25	1.3	0.004463	0.0	0.25	0.22	0.0020			0.33	0.58	0.0075	0.0140	14.0	9.5	0.8
82.13		0.025	1668.1	1620.5	41.1	0.0	325.6	0.23	1.2	0.004268	0.0	0.23	0.20	0.0018			0.25	0.58	0.0037	0.0098	9.8	5.5	1.2
82.10		0.025	1620.5	1573.2	39.9	0.0	284.5	0.20	1.2	0.004077	0.0	0.20	0.17	0.0016			0.16	0.58	0.0013	0.0070	7.0	3.0	1.6
82.08		0.025	1573.2	1526.2	38.7	0.0	244.6	0.18	1.2	0.003891	0.0	0.18	0.15	0.0014			0.08	0.58	0.0002	0.0056	5.6	1.7	1.9
82.05		0.025	1526.2	1479.5	37.6	205.9	205.9	0.15	1.2	0.003209	1.40	0.15	0.06	0.0006			0.00	0.58	0.0000	0.0038	3.8	0.6	2.8
82.03		0.025	1479.5	1433.1	36.4	168.3	168.3	0.13	1.1	0.003046	1.55	0.13	0.05	0.0004						0.0035	3.5	0.4	2.9
82.00		0.025	1433.1	1342.0	34.7	131.9	131.9	0.10	1.1	0.002799	1.70	0.10	0.04	0.0003						0.0031	3.1	0.3	3.1
81.98		0.025	1342.0	1311.2	33.2	97.2	97.2	0.08	1.1	0.002677	1.85	0.08	0.03	0.0002						0.0029	2.9	0.2	3.2
81.95		0.025	1311.2	1280.7	32.4	64.0	64.0	0.05	1.1	0.002559	2.00	0.05	0.02	0.0001						0.0027	2.7	0.1	3.3
81.93		0.025	1280.7	1250.5	31.6	31.6	31.6	0.03	1.0	0.002445	2.15	0.03	0.01	0.0001						0.0025	2.5	0.1	3.5
81.90		0.000	1250.5	1250.5	0.0	0.0	0.0	0.00	1.0	0.002388	2.30	0.00	0.00	0.0000						0.0024	2.4	0.0	0.0

Total Draw down time following 2- year storm (hours)	24.3
Total Draw down time following 5- year storm (hours)	24.9
Total Draw down time following 100- year storm (hours)	25.7
Total Draw down time following 15mm quality storm (hours)	18.7

Note: Total Outflow from the storage swale includes both outlet by infiltration as well as outlet to the roadside ditch by means of the outlet structure and sand filter.
Discharge from site includes only the portion of the flow exiting the storm pond by means of the outlet structure and sand filter and being directed to the adjacent roadside ditch.

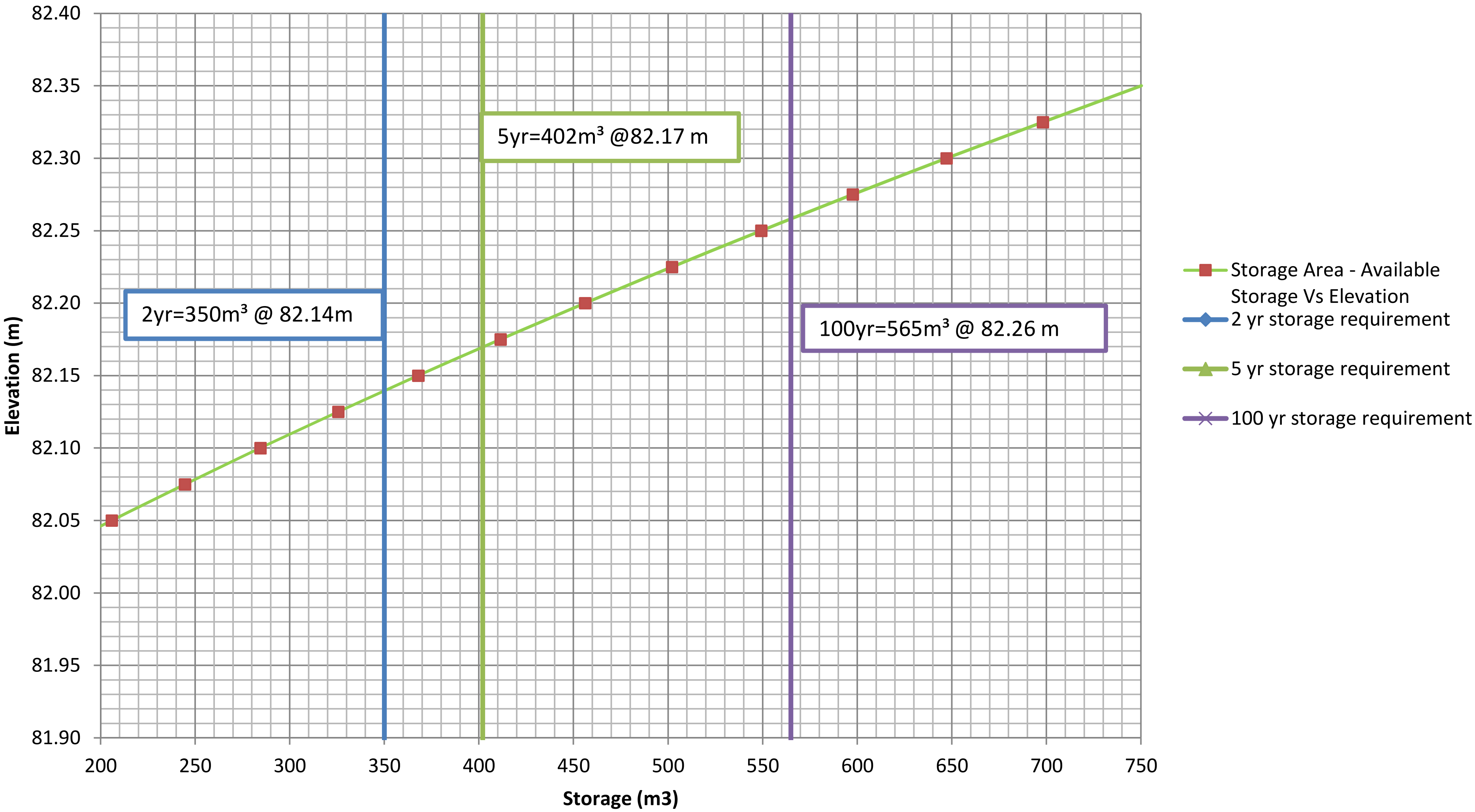
APPENDIX A: STORMWATER MANAGEMENT MODEL
Figure 1 - Discharge Vs Storage Curve

Client: Navan Fair
Job No.: 240297
Location: 1279 Coloniel Road
Date: August 29, 2024



Client: Navan Fair
Job No.: 240297
Location: 1279 Coloniel Road
Date: August 29, 2024

APPENDIX A - STORMWATER MANAGEMENT MODEL
Figure 2 - Elevation Vs Storage Curve



Guelph Permeameter

Input

Result

Single Head Method - AH1

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir):	2.16
Enter water Head Height ("H" in cm):	3
Enter the Borehole Radius ("a" in cm):	6
Enter the soil texture-structure category (enter one of the below numbers):	3

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): 0.1500

* "R" = three values in a row with matching Δh/Δt

Res Type	2.16
H	3
a	6
H/a	0.5
a*	0.12
C0.01	0.376
C0.04	0.383
C0.12	0.336
C0.36	0.336
C	0.336
R	0.150
Q	0.005
pi	3.142

α^* = 0.12 cm^{-1}

C = 0.336426

Q = 0.0054

K_{fs} = 7.22E-06 cm/sec
4.33E-04 cm/min
7.22E-08 m/sec
1.71E-04 inch/min
2.84E-06 inch/sec

Φ_m = 6.02E-05 cm^2/min

Single Head Method - AH2

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir):	2.16
Enter water Head Height ("H" in cm):	20
Enter the Borehole Radius ("a" in cm):	6
Enter the soil texture-structure category (enter one of the below numbers):	2

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): 1.2000

* "R" = three values in a row with matching Δh/Δt

Res Type	2.16
H	20
a	6
H/a	3.33333
a*	0.04
C0.01	1.21841
C0.04	1.29023
C0.12	1.28754
C0.36	1.28754
C	1.29023
R	1.200
Q	0.0432
pi	3.1415

α^* = 0.04 cm^{-1}

C = 1.290234

Q = 0.0432

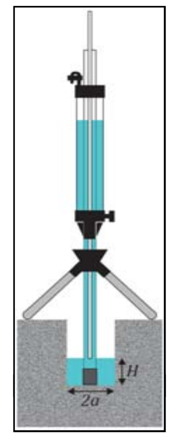
K_{fs} = 9.61E-06 cm/sec
5.77E-04 cm/min
9.61E-08 m/ses
2.27E-04 inch/min
3.78E-06 inch/sec

Φ_m = 2.40E-04 cm^2/min

Average

K_{fs} = 8.41E-06 cm/sec
5.05E-04 cm/min
8.41E-08 m/s
1.99E-04 inch/min
3.31E-06 inch/sec

Φ_m = 1.50E-04 cm^2/min



Calculation formulas related to shape factor (C). Where H_1 is the first water head height (cm), H_2 is the second water head height (cm), a is borehole radius (cm) and α^* is microscopic capillary length factor which is decided according to the soil texture-structure category. For one-head method, only C_1 needs to be calculated while for two-head method, C_1 and C_2 are calculated (Zang et al., 1998).

Soil Texture-Structure Category	$\alpha^*(cm^{-1})$	Shape Factor
Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.	0.01	$C_1 = \left(\frac{H_2/a}{2.081 + 0.121(H_2/a)}\right)^{0.672}$
Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.	0.04	$C_1 = \left(\frac{H_1/a}{1.992 + 0.091(H_1/a)}\right)^{0.683}$ $C_2 = \left(\frac{H_2/a}{1.992 + 0.091(H_2/a)}\right)^{0.683}$
Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.	0.12	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093(H_1/a)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093(H_2/a)}\right)^{0.754}$
Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macro pores, etc.	0.36	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093(H_1/a)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093(H_2/a)}\right)^{0.754}$

Calculation formulas related to one-head and two-head methods. Where $\bar{R}$ is steady-state rate of fall of water in reservoir (cm/s), K_{fs} is Soil saturated hydraulic conductivity (cm/s), Φ_m is Soil matric flux potential (cm^2/s), a^* is Macroscopic capillary length parameter (from Table 2), a is Borehole radius (cm), H_1 is the first head of water established in borehole (cm), H_2 is the second head of water established in borehole (cm) and C is Shape factor (from Table 2).

One Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$	$K_{fs} = \frac{C_1 \times Q_1}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi\left(\frac{H_1}{a^*}\right)}$
One Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$	$\Phi_m = \frac{C_1 \times Q_1}{(2\pi H_1^2 + \pi a^2 C_1)a^* + 2\pi H_1}$
Two Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$ $Q_2 = \bar{R}_2 \times 35.22$	$G_1 = \frac{H_2 C_1}{\pi(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1))}$ $G_2 = \frac{H_1 C_2}{\pi(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1))}$ $K_{fs} = G_2Q_2 - G_1Q_1$
Two Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$ $Q_2 = \bar{R}_2 \times 2.16$	$G_3 = \frac{(2H_2^2 + a^2C_2)C_1}{2\pi(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1))}$ $G_4 = \frac{(2H_1^2 + a^2C_1)C_2}{2\pi(2H_1H_2(H_2 - H_1) + a^2(H_1C_2 - H_2C_1))}$ $\Phi_m = G_3Q_1 - G_4Q_2$

Appendix B: Product Information

- Geotextile
- Nyloplast Catchbasins

Terrafix 360R - Geotextile

Function: Filtration & Drainage.

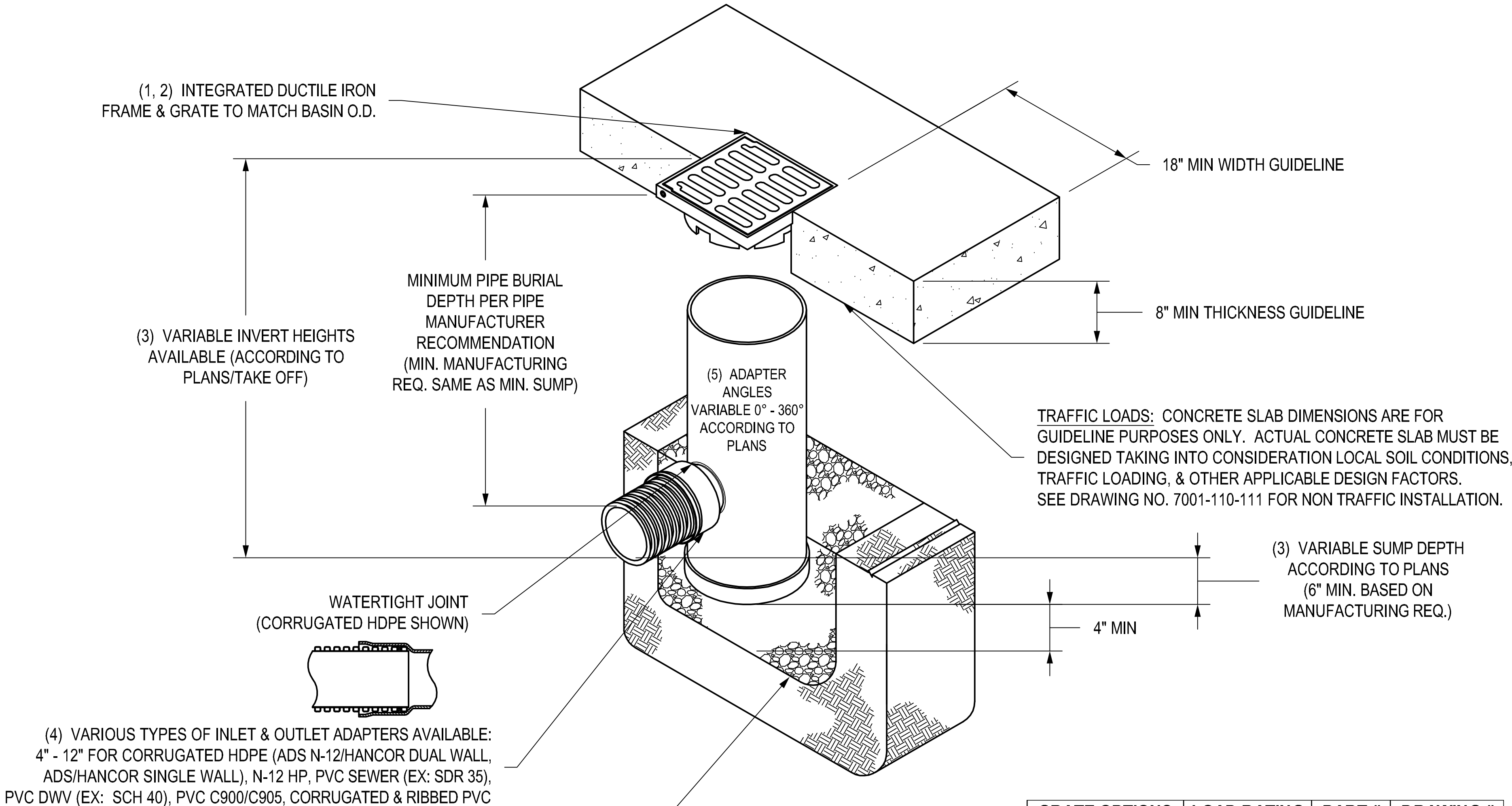
Terrafix 360R is a needle-punched nonwoven geotextile made of 100% virgin polypropylene staple fibers, which are formed into a random network for dimensional stability. Terrafix 360R resists ultraviolet deterioration, rotting, biological degradation, naturally encountered alkalis and acids. Polypropylene is stable within the pH range of 2-13.

<u>Types of applications for 360R are:</u> Subdrains, French Drains, Foundation Drains, Trench Drains, Blanket Drains.

360R used in weaker soil conditions. Used in conjunction with coarser drainage materials.

Property	ASTM Test Method	Value Metric Units
Typical Geotextile Properties		
• Grab Tensile Strength	D 4632	712 N
• Elongation at break	D 4632	50-105%
• Tear Resistance	D 4533	267 N
• Puncture CBR	D 6241	1820 N
• Permittivity	D 4491	1.5 sec^{-1}
• Water Flow	D 4491	4480 $l/min/m^2$
• Apparent Opening Size	D 4751	212 µm
• U.V. Stability	D 4355	70% @ 500hrs
• FOS	CAN 148.1 No.1	70-160 µm

NYLOPLAST 12" DRAIN BASIN: 2812AG _ _ X



THE BACKFILL MATERIAL SHALL BE CRUSHED STONE OR OTHER GRANULAR MATERIAL MEETING THE REQUIREMENTS OF CLASS I, CLASS II, OR CLASS III MATERIAL AS DEFINED IN ASTM D2321. BEDDING & BACKFILL FOR SURFACE DRAINAGE INLETS SHALL BE PLACED & COMPACTED UNIFORMLY IN ACCORDANCE WITH ASTM D2321.

GRATE OPTIONS	LOAD RATING	PART #	DRAWING #
PEDESTRIAN	MEETS H-10	1299CGP	7001-110-202
STANDARD	MEETS H-20	1299CGS	7001-110-203
SOLID COVER	MEETS H-20	1299CGC	7001-110-204
PEDESTRIAN BRONZE	N/A	1299CGPB	7001-110-205
DOME	N/A	1299CGD	7001-110-206
DROP IN GRATE	LIGHT DUTY	1201DI	7001-110-021

1 - GRATES/SOLID COVER SHALL BE DUCTILE IRON PER ASTM A536 GRADE 70-50-05, WITH THE EXCEPTION OF THE BRONZE GRATE.
2 - FRAMES SHALL BE DUCTILE IRON PER ASTM A536 GRADE 70-50-05
3 - DRAIN BASIN TO BE CUSTOM MANUFACTURED ACCORDING TO PLAN DETAILS. RISERS ARE NEEDED FOR BASINS OVER 84" DUE TO SHIPPING RESTRICTIONS. SEE DRAWING NO. 7001-110-065
4 - DRAINAGE CONNECTION STUB JOINT TIGHTNESS SHALL CONFORM TO ASTM D3212 FOR CORRUGATED HDPE (ADS N-12/HANCOR DUAL WALL), N-12 HP, & PVC SEWER.
5 - ADAPTERS CAN BE MOUNTED ON ANY ANGLE 0° TO 360°. TO DETERMINE MINIMUM ANGLE BETWEEN ADAPTERS SEE DRAWING NO. 7001-110-012.

DRAWN BY	EBC	MATERIAL	
DATE	03-29-06		
REVISED BY	NMH	PROJECT NO./NAME	
DATE	03-11-16		
DWG SIZE	A	SCALE 1:20	SHEET 1 OF 1

ADS Nyloplast
3130 VERONA AVE
BUFORD, GA 30518
PHN (770) 932-2443
FAX (770) 932-2490
www.nyloplast-us.com

TITLE
12 IN DRAIN BASIN QUICK SPEC INSTALLATION DETAIL

DWG NO. 7001-110-189 REV E

Appendix C: Drawings

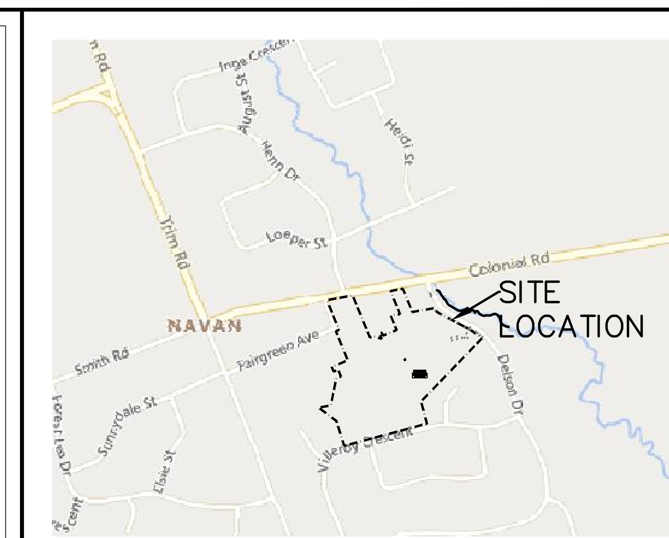
- 240297 – PRE – Pre-Development drawing
- 240297 – POST – Controlled and Uncontrolled Areas
- 240297 – GR – Grading and Drainage Plan
- 240297 – ESC –Erosion and Sediment Control Plan

PREDEVELOPMENT PLAN

SCALE = 1:750m

LEGEND

CONTROLLED	CATCHMENT LABEL
0.39	CATCHMENT AREA (HECTARES)
0.34	RUNOFF COEFFICIENT
	CATCHMENT AREA BOUNDARY
	DIRECTION OF FLOW
	PROPERTY LINE
	SILT FENCE
	DRAINAGE
	CONTROLLED AREA
	UNCONTROLLED AREA



KEY PLAN

NOT TO SCALE

0 25 50

1:750 METRES

NOTES:
1. ALL DIMENSIONS ARE IN METRES, UNLESS OTHERWISE SPECIFIED; ALL ELEVATIONS ARE IN METRES.
2. THIS IS NOT A LEGAL SURVEY.
3. EXISTING SERVICES INFORMATION SHOWN ARE BASED ON BEST CURRENT INFORMATION. CONTRACTOR TO VERIFY EXACT LOCATION AND REPORT ANY DISCREPANCIES TO KOLLAARD ASSOCIATES INC.
4. CLIENT IS RESPONSIBLE FOR ACQUIRING ALL NECESSARY PERMITS.
5. CONTRACTOR TO VERIFY THAT APPROPRIATE PERMITS HAVE BEEN ACQUIRED PRIOR TO ANY CONSTRUCTION.
6. CONTRACTOR IS RESPONSIBLE FOR LOCATION AND PROTECTION OF UTILITIES.
7. ALL DIMENSIONS TO BE VERIFIED ON SITE BY CONTRACTOR PRIOR TO CONSTRUCTION.
8. THIS DRAWING IS NOT FOR CONSTRUCTION UNTIL ALL APPROVALS HAVE BEEN GRANTED.
9. INSPECTION OF ROUGH GRADE BY KOLLAARD ASSOCIATES INC. AND MUNICIPALITY MUST BE CONDUCTED PRIOR TO PLACEMENT OF TOPSOIL OR SOD.
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No.	REVISION	DATE	BY
2.	RESPONSE TO CITY REVIEW COMMENTS	AUG 29,2024	NJR
1.	ISSUED FOR SITE PLAN APPROVAL	JUNE 28,2024	NJR

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DESIGN	NJR
DRAWN	ABD
CHECKED	NJR
APPROVED	SD

STAMP

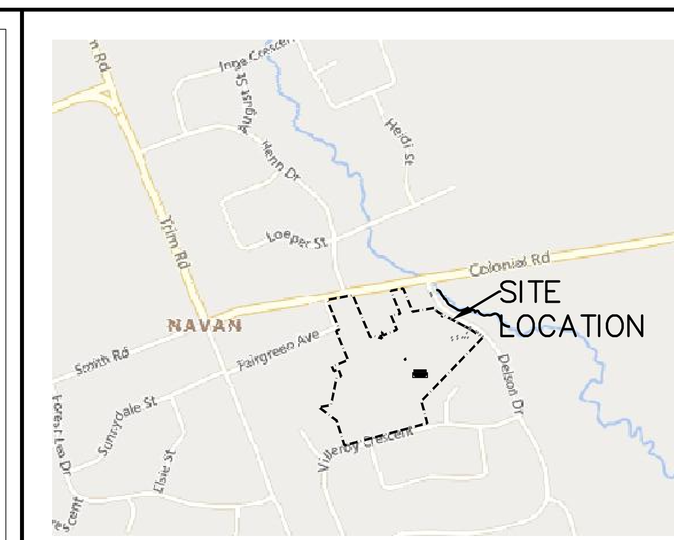
CLIENT NAME	CUMBERLAND TOWNSHIP AGRICULTURE SOCIETY	PROJECT No.	240297
PROJECT NAME	PROPOSED BARN AND PAVILION (OPEN WALLED STRUCTURE)	DATE	JUNE 28, 2024
PROJECT LOCATION	1279 COLONIAL ROAD, OTTAWA, ON, K4B 1N1	SCALE	1:750 m
DRAWING	PREDEVELOPMENT PLAN	DRAWING No.	240297–PRE

D07-12-24-0055

ARCH D

POST–DEVELOPMENT PLAN

SCALE = 1:750m



KEY PLAN

NOT TO SCALE

0 25 50
1:750 METRES

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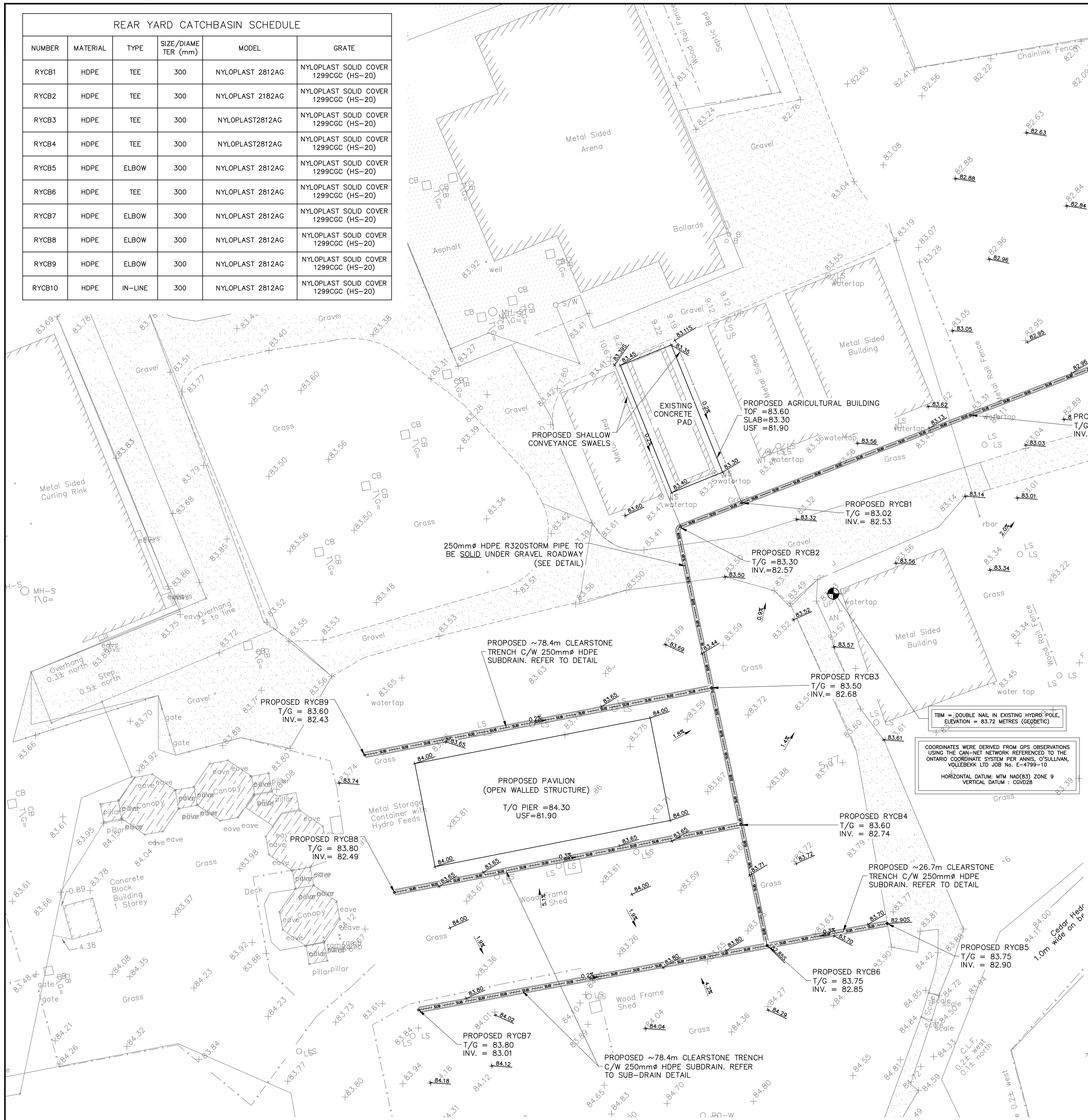
STAMP

CLIENT NAME	CUMBERLAND TOWNSHIP AGRICULTURE SOCIETY
PROJECT NAME	PROPOSED BARN AND PAVILION (OPEN WALLED STRUCTURE)
PROJECT LOCATION	1279 COLONIAL ROAD, OTTAWA, ON, K4B 1N1
DRAWING	POST–DEVELOPMENT CONDITOINS
PROJECT No.	240297
DATE	JUNE 28, 2024
SCALE	1:750 m
DRAWING No.	240297–POST

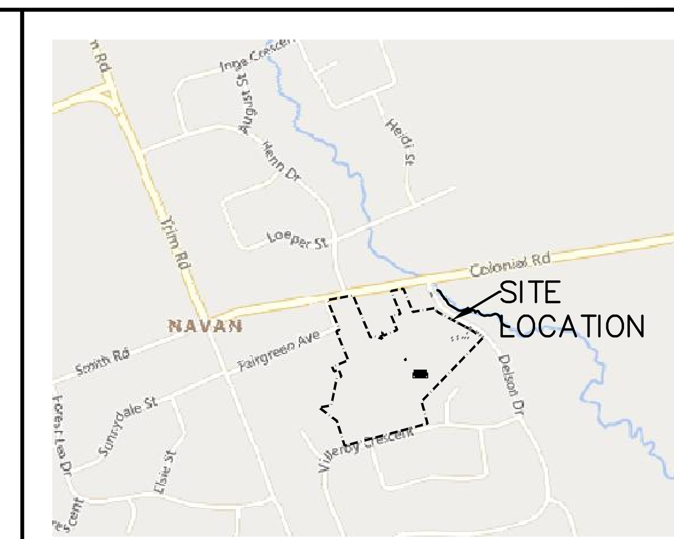
ARCH D

REAR YARD CATCHBASIN SCHEDULE					
NUMBER	MATERIAL	TYPE	SIZE/DIAMETER (mm)	MODEL	GRATE
RYCB1	HDPE	TEE	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB2	HDPE	TEE	300	NYLOPLAST 2182AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB3	HDPE	TEE	300	NYLOPLAST2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB4	HDPE	TEE	300	NYLOPLAST2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB5	HDPE	ELBOW	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB6	HDPE	TEE	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB7	HDPE	ELBOW	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB8	HDPE	ELBOW	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB9	HDPE	ELBOW	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)
RYCB10	HDPE	IN-LINE	300	NYLOPLAST 2812AG	NYLOPLAST SOLID COVER 1299CGC (HS-20)



GRADING PLAN

SCALE = 1:500m



KEY PLAN

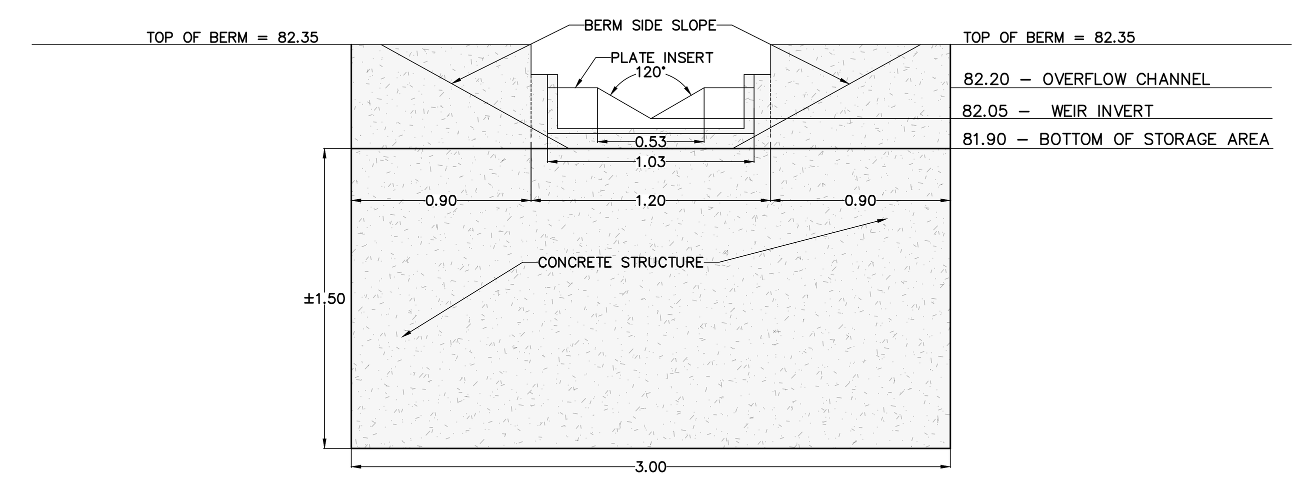
NOT TO SCALE

LEGEND

- PROPOSED ELEVATION
- PROPOSED SUBDRAIN ELEVATION
- AUGERHOLE
- PROPOSED SUBDRAIN
- EXISTING STORM PIPE
- TOP OF SLOPE
- TEMPORARY BENCHMARK

PROPOSED STORM AND SUBDRAIN PIPE

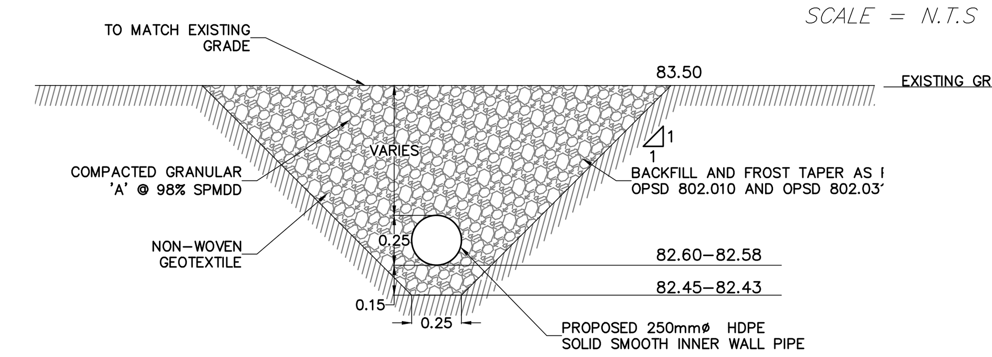
PROPOSED STORM AND SUBDRAIN PIPE TO CONSIST OF 250mmø HDPE 320 kPa SMOOTH INNER WALL PERFORATED PIPE IN A FILTER SOCK



OUTLET WEIR STRUCTURE

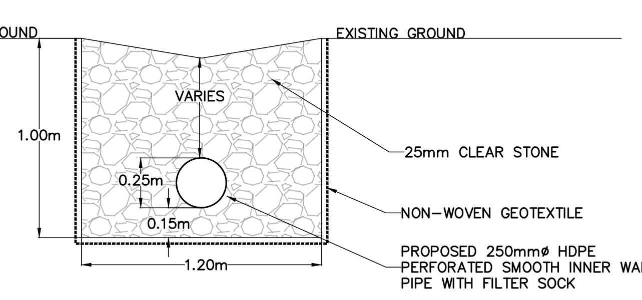
SECTION A–A'

SCALE = N.T.S



STORM PIPE UNDER GRAVEL ROAD

SCALE = N.T.S

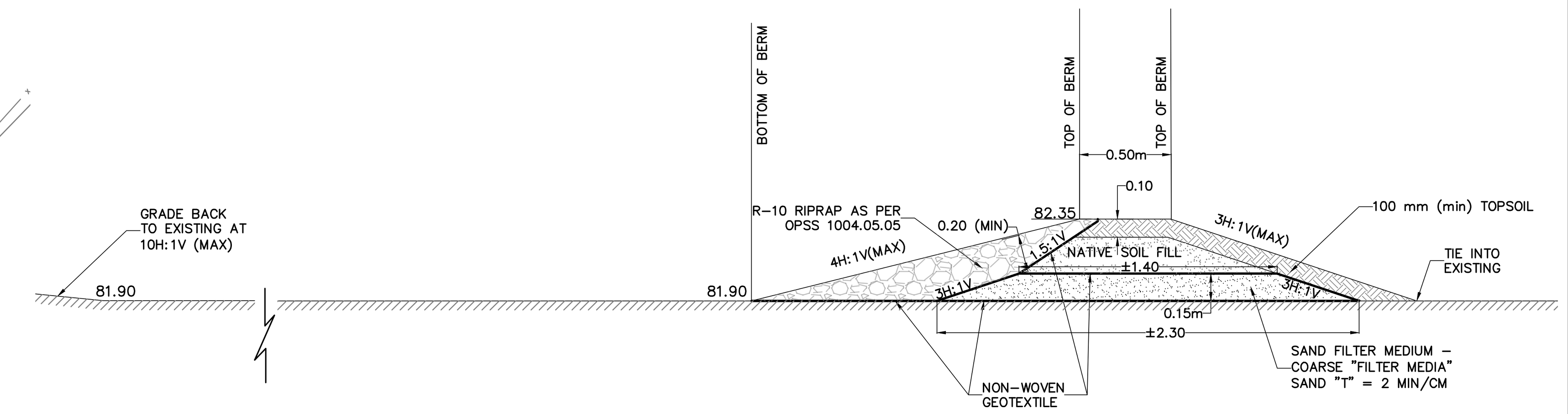


SUB-DRAIN DETAIL

SCALE = N.T.S

CONSTRUCTION PROCEDURES

1. IMPLEMENT SITE EROSION CONTROL MEASURES PER DRAWING 240297-ESC.
2. REMOVE AND STOCKPILE TOPSOIL FROM PROPOSED GRAVEL STORAGE AREA/CLEAR STONE TRENCHES (WHERE APPLICABLE).
3. IMPLEMENT EROSION CONTROL MEASURES. AROUND TOPSOIL STOCKPILE BY INSTALLING A SILT FENCE AROUND THE PERIMETER OF THE TOPSOIL STOCK PILE. THE SILT FENCE SHOULD BE INSTALLED IN ACCORDANCE TO OPSD 209.110.
4. EXCAVATE THE CLEAR STONE STORAGE AREA. SHOULD THE SUBGRADE ELEVATIONS BE LOWER THAN THE PROPOSED UNDERSIDE OF CLEAR STONE AREA, NATIVE SOIL FROM EXCAVATED AREAS CAN BE USED TO RAISE SUBGRADE TO DESIGN ELEVATION.
5. INSTALL GEOTEXTILE AS SHOWN ON THE DETAILS.
6. INSTALL FIRST 0.15m OF CLEAR STONE WITHIN THE GRAVEL STORAGE AREA, AND FIRST 0.10m OF CLEAR STONE WITHIN THE TRENCHES.
7. INSTALL THE PROPOSED GEOWEB WITHIN THE GRAVEL STORAGE AREA AS SHOWN ON THE DETAILS. INSTALL PERFORATED STORM PIPE/ SUBDRAINS AND PROPOSED RYCBS.
8. BACKFILL TRENCHES AND GEOWEB WITH REMAINDER OF CLEAR STONE.
9. FINAL GRADING AND REINSTATE TOPSOIL IN DISTURBED AREAS.
10. ONLY ONCE DISTURBED VEGETATION IS FULLY ESTABLISHED, AND REMAINING FILL PILES HAVE BEEN REMOVED FROM SITE, CAN EROSION CONTROL MEASURES BE REMOVED FROM SITE.



CROSS-SECTION B-B

STORAGE AREA WITH SAND FILTER

SCALE = N.T.S

0 20 40

1:500 METRES

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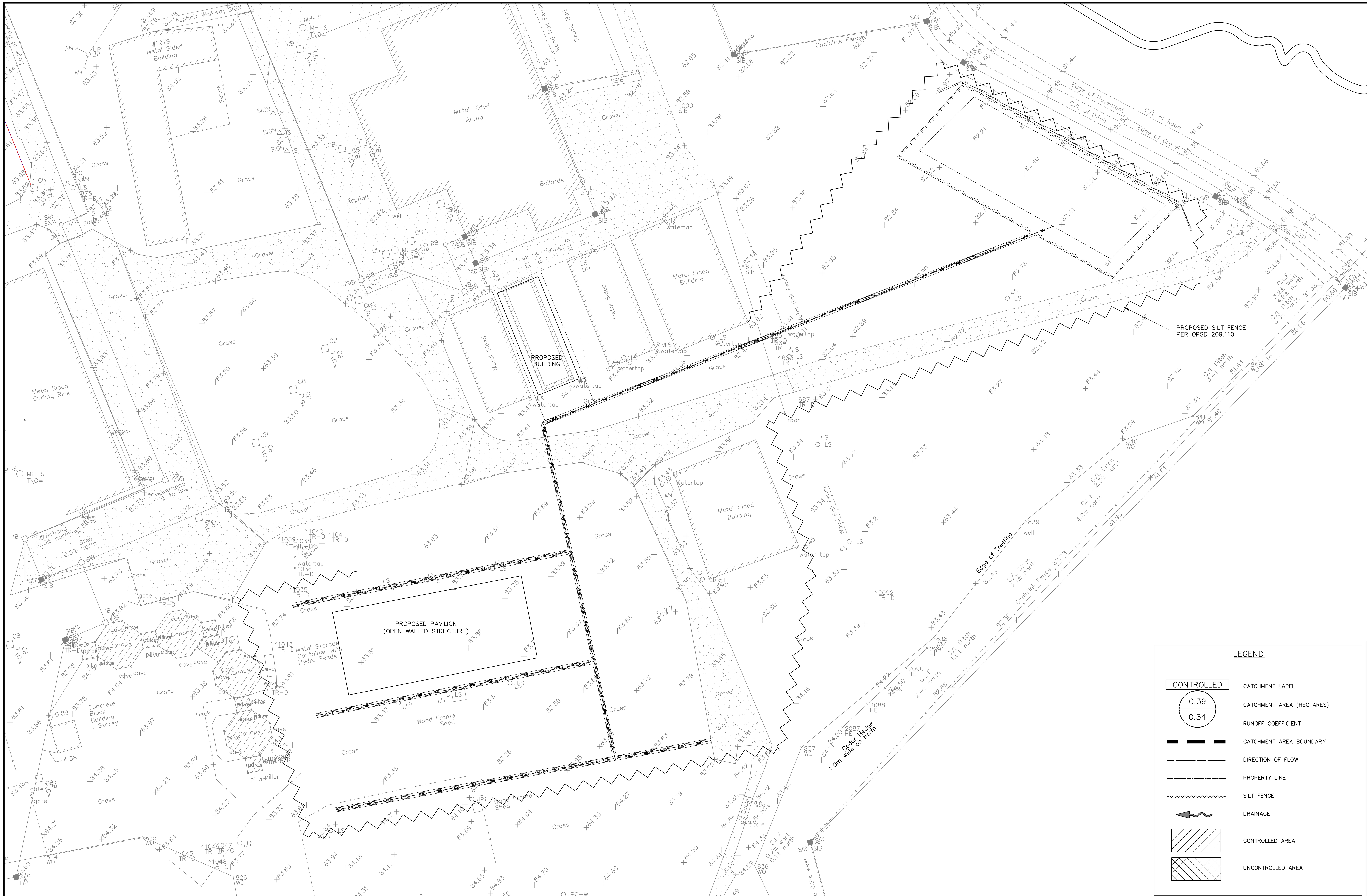
DESIGN NJR | DRAWN ABD | CHECKED NJR | APPROVED SD

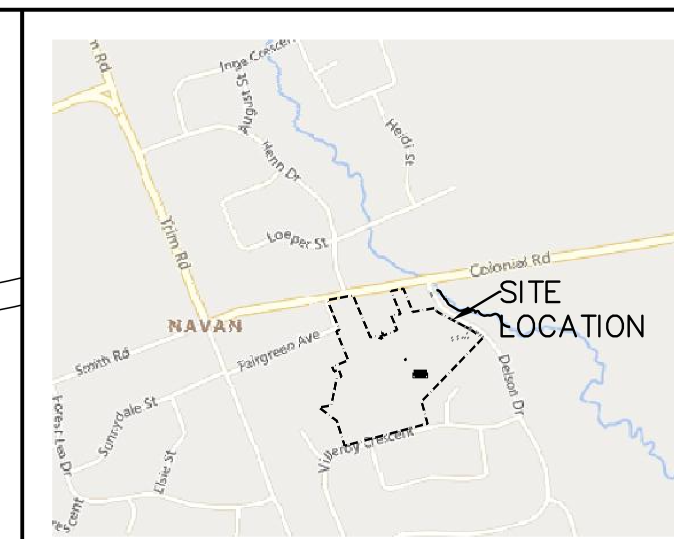
STAMP: LICENSED PROFESSIONAL ENGINEER AUG.29.2024 S.E. deWit 100079612 PROVINCE OF ONTARIO

CLIENT NAME	CUMBERLAND TOWNSHIP AGRICULTURE SOCIETY
PROJECT NAME	PROPOSED BARN AND PAVILION (OPEN WALLED STRUCTURE)
PROJECT LOCATION	1279 COLONIAL ROAD, OTTAWA, ON, K4B 1N1
DRAWING	SITE GRADING PLAN
PROJECT No.	240297
DATE	JUNE 28, 2024
SCALE	AS SHOWN
DRAWING No.	240297-GR



EROSION AND SEDIMENT CONTROL PLAN

SCALE = 1:500m



KEY PLAN

NOT TO SCALE

EROSION AND SEDIMENT CONTROL NOTES:

1. THE CONTRACTOR SHALL IMPLEMENT BEST MANAGEMENT PRACTICES, TO PROVIDE FOR PROTECTION OF THE STORM SEWER SYSTEM, DURING CONSTRUCTION ACTIVITIES. THE CONTRACTOR ACKNOWLEDGES THAT FAILURE TO IMPLEMENT APPROPRIATE EROSION AND SEDIMENT CONTROL MEASURES MAY BE SUBJECT TO PENALTIES IMPOSED BY ANY APPLICABLE REGULATORY AGENCY.
2. THE OWNER (AND/OR CONTRACTOR) AGREES TO PREPARE AND IMPLEMENT AN EROSION AND SEDIMENT CONTROL PLAN AT LEAST EQUAL TO THE STATED MINIMUM REQUIREMENTS AND TO THE SATISFACTION OF THE TOWNSHIP OF NORTH DUNDAS, APPROPRIATE TO THE SITE CONDITIONS, PRIOR TO UNDERTAKING ANY SITE ALTERATIONS (FILLING, GRADING, REMOVAL OF VEGETATION, ETC.) AND DURING ALL PHASES OF SITE PREPARATION AND CONSTRUCTION IN ACCORDANCE WITH THE CURRENT BEST MANAGEMENT PRACTICES FOR EROSION AND SEDIMENT CONTROL.
3. THE CONTRACTOR IS TO ENSURE THAT THE SITE ACCESS POINTS AND ADJACENT STREETS TO THE ACCESS POINTS ARE MAINTAINED AND KEPT CLEAN OF CONSTRUCTION MATERIALS SUCH AS, BUT NOT LIMITED TO MUD, DIRT, CLAY AND GRANULARS ON A DAILY BASIS OR AS NECESSARY, TO THE SATISFACTION OF THE CITY OF OTTAWA.
4. EVERY EFFORT WILL BE MADE TO ENSURE THAT ALL DISTURBED AREAS ARE TOPSOILED AND SEEDED AS SOON AS REASONABLY POSSIBLE.
5. THE SEDIMENT AND EROSION CONTROL PLAN IS A LIVING DOCUMENT WHICH MAY BE AMENDED BY ONSITE REQUIREMENTS AT THE APPROVAL OF THE MUNICIPALITY AND THE CONSERVATION AUTHORITY.
6. ALL SEDIMENT AND ERROSION CONTROL MEASURES TO BE INSPECTED/CLEANED AFTER RAIN EVENTS.
7. MUD MAT OR MECHANICAL TRACKOUT PLATE TO BE INSTALLED AT CONSTRUCTION SITE ENTRANCE.

MINIMUM EROSION AND SEDIMENT CONTROL PLAN REQUIREMENTS:

- TIME THE DEMOLITION AND EXCAVATION ACTIVITIES SO THAT THEY OCCUR NO SOONER THAN IS NECESSARY FOR SUBSEQUENT CONSTRUCTION ACTIVITIES.
- LANDSCAPE THE SITE AS SOON AS PRACTICALLY POSSIBLE.
- USE SILT FENCES AROUND ANY STOCKPILES OF SOIL.
- PRIOR TO CONSTRUCTION, SILT FENCE BARRIERS (OPSD 219.110) WILL BE PLACED ALONG THE PROPERTY LINES AS ON THE DRAWING.
- THE SILT FENCE SHOULD BE REMOVED ONLY WHEN THE SITE IS STABILIZED.
- FILTER SOCKS SHOULD BE INSTALLED ACROSS EXISTING STORM MANHOLE AND CATCH BASIN LIDS. AS WELL, FILTER SOCKS SHOULD BE INSTALLED ACROSS THE PROPOSED CATCH BASIN AND MANHOLE LIDS IMMEDIATELY AFTER THE STRUCTURES ARE PLACED. THE FILTER SOCKS SHOULD ONLY BE REMOVED ONCE THE ASPHALTIC CONCRETE IS INSTALLED AND THE SITE IS CLEANED..

0 20 40
1:500 METRES

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DESIGN	NJR
DRAWN	ABD
CHECKED	NJR
APPROVED	SD

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DRAWING	EROSION AND SEDIMENT CONTROL PLAN
PROJECT No.	240297
DATE	2024 MAY 28
SCALE	AS SHOWN
DRAWING No.	240297-ECS

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D07-12-24-0055