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SITE SERVICING AND STORMWATER MANAGEMENT REPORT

Residential Apartment Building
5574 Rockdale Road, Vars
Ottawa, Ontario.

Prepared For:

Mr. J.P. Bergeron
880 Smith Road
Navan, Ontario
K4B 1N9

PROJECT #: 220863

DISTRIBUTION
City of Ottawa
J.P. Bergeron
Kollaard Associates Inc.

Issued in Response to Engineering Comments with Respect to Previous Submissions by others.

December 21, 2023

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1 INTRODUCTION

Mr. J.P. Bergeron has retained the services of Kollaard Associates Inc to complete the Civil Engineering design submission in support of the site plan control application for the proposed residential development at 5574 Rockdale Road, Vars, Ottawa, Ontario. This Site Servicing and Stormwater Management report will address the serviceability of the proposed residential development with respect to the water and sanitary demands and outline the proposed design to meet these requirements.

The report shall also summarize the stormwater management (SWM) design requirements and proposed works that will address stormwater flows arising from the site under post-development conditions and will identify any stormwater servicing concerns. The report will describe any measures to be taken during construction to minimize erosion and sedimentation for the proposed development. This report will also address the engineering review comments provided for the previous submissions by others.

This report and site servicing plan have been prepared based on the design prepared by A. Dagenais Associates Ltd. and revised by Blanchard Letendre Engineering Ltd. (BLEL). The information contained herein is based on the provided drawings and report and if there is any discrepancy with the survey or site plan, Kollaard should be informed in order to verify the information and complete the changes if required.

1.1 Background and Project History

A Dagenais & Assoc. Inc., was previously retained by JP Bergeron to provide revised site development drawings and a storm water management report for the proposed residential project at 5574 Rockdale Road, Vars. As A. Dagenais & Assoc. Inc. was subsequently acquired by BLEL, BLEL was then retained by JP Bergeron to finalize the site servicing and stormwater management design.

It is understood that BLEL advised Mr. J.P. Bergeron that they no longer wished to continue with the project following which Mr. J.P. Bergeron retained Kollaard to finalize the Civil Engineering design submission.

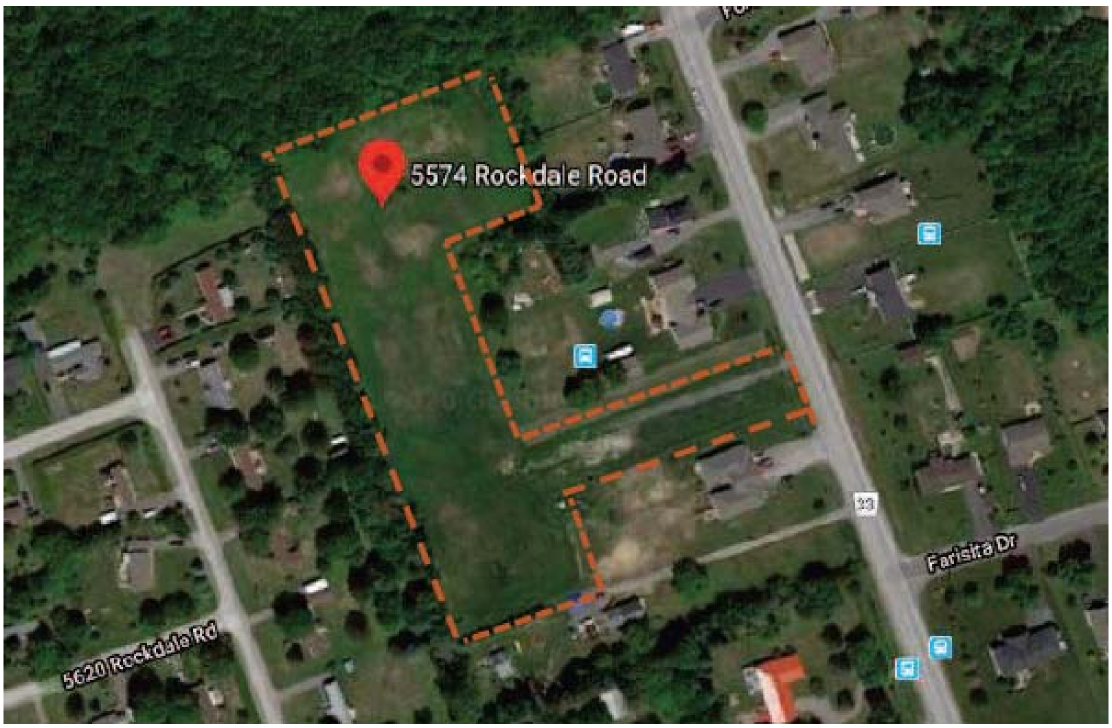
Previous submissions completed by A. Dagenais & Assoc. Inc are dated: April 30, 2019; December 8, 2017; September 9th, 2015; July 29, 2015; September 17, 2014; April 10th, 2014; November 11th, 2013

Previous submissions completed by Blanchard Letendre Engineering Ltd. are dated November 29, 2021.

1.2 Existing Site

For the purposes of this report, Rockdale Road is considered to be oriented along a north south axis. The proposed development site is located west of Rockdale Road in Vars, Ottawa, Ontario. As shown in the aerial photo in Figure 1 below, the majority of the site is located behind the existing residential development along Rockdale Road and is accessed by means of an about 30 metre wide lot.

Figure 1 – Existing Site at 5574 Rockdale Road.



The proposed development site has a total area of approximately 1.777 hectares, is undeveloped and is surfaced with unmaintained grasses. There are no watercourses or easements effecting development on the proposed site. The site is located within the Ruisseau Shaw's Creek subwatershed. Stormwater runoff from the site current is directed by sheet flow to the adjacent properties and to the ditch along Rockdale Road. This ditch outlets by means of an existing storm sewer to the Ruisseau Shaw's Creek.

The topographic survey completed on the site defines a high point/ridge on the northern portion of the site, which separates the drainage pattern of the north side of the side from the remainder of the site. Since the proposed development will be entirely contained within the portion of the site south of the high point/ridge, a limit of development for the proposed site will be defined occurring along the top of the ridge. The northern portion of the site is beyond the limit of development and will not be considered in the analysis. The limit of development is illustrated on Kollaard Associates Drawing No. 220863-PRE and 220863-POST. As such, the northern portion has been deemed as a “no-development zone” and has been omitted from the stormwater management calculations. The “no development zone” has an area of 484 m^2, which reduces the development area to be 1.777 ha – 0.484 ha = 1.293 ha

2 STORMWATER DESIGN

2.1 Stormwater Management Design Criteria

Design of the storm sewer system was completed in conformance with the City of Ottawa Design Guidelines. (October 2012). Section 5 “Storm and Combined Sewer Design”.

2.1.1 Quantity Control Design Criteria

The quantity control design criteria provided for the site consisted of:

The Post-development runoff rate from the site during all storm events up to and including a 100 year design storm must be controlled to pre-development levels for the 5 year design storm event assuming a runoff coefficient equal to the lesser of existing conditions or C = 0.5.

The proposed development should not result in additional runoff onto the adjacent properties.

2.1.2 Additional System Design Criteria

All proposed storm sewers should be sized to convey the unrestricted flow occurring during a 5 year storm event under gravity or open channel conditions.

2.1.3 Quality Control Design Criteria

The water quality objective was provided by the South Nation Conservation Authority. The water quality control objective is an enhanced level of protection which corresponds to 80% Total Suspended Solids removal.

2.2 Storm Analysis Variables

2.2.1 Runoff Coefficients

The runoff coefficients for a 5 year return period for the following surfaces are:

<u>Impervious</u>

Roofs – C = 9.0

Asphalt and Sidewalk – C = 0.9

<u>Pervious surfaces</u>

Grasses and Soft Landscaping – C = 0.30.

A 25% increase for the 100-year runoff coefficients was used as per City of Ottawa guidelines. Refer to Appendix A for pre-development and post development runoff coefficients. Note a maximum of C = 1.0 was used for impervious surfaces.

2.2.2 Impervious Ratio

The impervious ratio for the site is the total impervious area divided by the total site area and is equal to 0.257 / 1.777 = 14.5 percent.

The impervious area for the portion of the site within the limit of development is equal to

0.257/1.293 = 19.9 percent.

The impervious ratio for the controlled area Post-C1 contributing to the stormwater management facilities in Post-C1 is equal to 0.222 / 0.648 = 34.3 percent.

The impervious ratio for the controlled area Post-C2 contributing to the stormwater management facilities in Post-C2 is equal to 0.001 / 0.361 = 0 percent.

2.2.3 Time of Concentration

2.2.3.1 Pre-Development

The time of concentration for pre-development conditions was calculated using the Velocity method. The velocity method assumes that the time of concentration is the sum of travel times for segments along the hydraulically most distant flow path. The segments used in the velocity method may be of three types: sheet flow T_s, shallow concentrated flow T_{sc}, and open channel flow T_c. Since the area of consideration for the stormwater analysis consists of a single site, open channel flow will not be present and is not considered.

Travel time for sheet flow:

Sheet flow is defined as flow over plane surfaces. Sheet flow usually occurs at the upper end of each individual catchment and typically occurs for no more than 30 metres before transitioning to shallow concentrated flow. The Manning's roughness coefficients used for sheet flow only apply for flow depth of less than 3 cm and vary from those used in shallow concentrated flow and open channel flow.

The Manning's roughness coefficient for sheet flow includes the effects of roughness as well as the effects of raindrop impact, drag over the surface, obstacles such as litter and rocks and transportation of sediment. The Manning's n for sheet flow ranges from 0.8 for Woods (with dense underbrush and no clearing of deadfall and other detritus) to 0.018 for bare hard packed earth. The Manning's n for grass ranges from 0.15 (short grass prairie or lawn grass with bare patches), 0.24 (dense prairie grasses (long stems grasses)) to 0.41 (Bermuda grass or dense lush lawn grass).

Since the sheet flow will occur over unmaintained grasses, a Manning's n of 0.24 was used.

The time of Concentration for Pre-C2 was calculated as follows:

The travel time for sheet flow is calculated using the simplified Manning's Kinematic solution as follows:

$$T_s = \frac{0.091(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

Where T_s = travel time, h
n = Manning's roughness coefficient sheet flow = 0.24
l = sheet flow length, 30 m
P_2 = 2-year 24-hour rainfall, = 48.5 mm
S = Slope of land surface m/m = 0.03

T_s = 0.26 hours

Travel time for shallow concentrated flow:

The flow velocity used to calculate the time of travel for shallow concentrated flow was determined using Figure 15-4 of Chapter 15 of the USDA handbook (Included in Appendix A of this Report). This figure can be used to determine the velocity when the slope and ground cover is known. The ground cover to be used in reading Figure 15-4 was determined as follows: Minimum tillage cultivation - Manning's n for concentrated flow = 0.101. From Figure 15-4 of the USDA Handbook using a slope of 0.97 %, the velocity is estimated at 0.15 m/s (0.5 ft/s).

$$T_{sc} = \frac{l}{3600\,V}$$

Where T_{sc} = travel time, h
l = distance of shallow concentrated flow = 103 m
V = average velocity = 0.15 m/s
T_{sc} = 0.19 hrs

Total time of concentration for PRE-CA2 is equal to T_s = 0.26 hours + T_{sc} = 0.19 hrs = 0.45 hrs = 27 min.

The time of concentration for each pre-development area is summarized in the following Table 2-1

Table 2-1 Pre-Development Times of Concentration

Catchment Area	Area of Catchment	Time of Sheet Flow	Time of Shallow Concentrated	Total time of Concentration
	m^2	hrs	hrs	Min
Pre-C1	904	0.30	0.00	18.0
Pre-C2	8034	0.26	0.19	27.0
Pre-C3	3988	0.26	0.10	21.6
OS1	1351	0.31	0.06	22.2
OS2	3601	0.32	0.06	22.2

Since PRE-CA1 is relatively small when compared to PRE-CA2 and PRE-CA3, the pre-development time of concentration was taken as the average time of concentration for catchment areas Pre-C2 and Pre-C3 (27+21.6)/2 is equal to 24.3 min.

2.2.3.2 Post-Development

During post-development conditions, runoff will be directed to the proposed stormwater management facility or be allowed to flow offsite following existing drainage patterns. In keeping with City of Ottawa sewer design guidelines, a post development time of concentration of 10 minutes was used.

2.3 Stormwater Quantity Control

2.3.1 Peak Flow Calculation Method

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 two return periods were considered; the 5 and 100-year events. The formulae for each are:

5-Year Event

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

100-Year Event

$$i = \frac{1735.688}{(t_c + 6.014)^{0.820}}$$

Where t_c is time of concentration

Using the previously calculated pre-development t_c =24.3 min, the above formulas provide the following intensities:

5-Year Event

i = 65.04 mm/hr

100-Year Event

i = 105.81 mm/hr

2.3.2 Pre-development Site Conditions

The site is currently undeveloped and is zoned as Village Residential Third Density Zone (V3E). The existing ground surface covering consists of overgrown and unmaintained grasses. With lack of maintenance, some shrub and bush growth is starting to occur. The existing ground surface is sloped from the east corner of the southernmost side of the lot towards the north and west. The first about 30 metres of the site, from Rockdale Road, which will contain the site access has a slight slope towards Rockdale Road and a cross slope towards the north.

2.3.2.1 Pre-development Off-Site Drainage Patterns

The existing residential properties between the subject site and Rockdale Road (east of the subject site and north of the proposed site access) are developed. Runoff from the front portion of each of these properties, to about the middle of each dwelling, is directed east towards Rockdale Road. Runoff from the remaining portion of these properties is directed by uncontrolled sheet flow onto the subject property.

As the topography at the southeast corner of the subject site represents a local high point in the area, runoff from the residential properties east and south of the site access and south of the site is directed away from the subject site. There will be no offsite runoff from these lots. Runoff from the residential trailer park property west of the site is directed by sheet flow and stormwater swales within the trailer park west towards the Ruisseau Shaw's Creek.

The land north of the subject site, currently zoned Rural Countryside (RU) is undeveloped and tree covered. The general slope towards the north and west continues across this land. As such there will be no offsite runoff from the lands north of the site directed onto the subject site.

2.3.2.2 Pre-development Catchment Areas

The pre-development catchment areas are indicated on the attached Pre-Development Catchment Areas drawing 220863-PRE. The pre-development catchment areas have been divided into three onsite catchments labelled PRE-C1, PRE-C2 and PRE-C3 and two off-site catchments labelled OS1 and OS2.

2.3.2.3 Pre-Development Runoff Coefficients

The pre-development runoff coefficient for each catchment was calculated using weighted average based on the existing ground surface conditions as follows:

$$C = \frac{\left(A_{imp}\ x\ 0.9 + A_{grass}\ x\ 0.30\right)}{A_{total}}$$

The calculations were completed on the spread sheet, Sheet 1 – Allowable Release Rate and SWM Summary, which is included in Appendix A and summarized in the following Table 2-2:

Table 2-2 Pre-Development Runoff Coefficients

Catchment Label	Pre-C1	Pre-C2	Pre-C3	OS1	OS2
5-Year C	0.30	0.30	0.30	0.37	0.39
100-Year C	0.38	0.38	0.38	0.45	0.47

2.3.2.4 Pre-Development Runoff Rate

Using the Rational Method with the above calculated areas, runoff coefficients and storm intensities, the pre-development runoff rates for the 5-year and 100-year storms are:

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

Onsite:

5 year = 0.30 x 62.04 x 1.293 / 360 = 0.0668 m^3/s
100 year = 0.38 x 105.81 x 1.293 / 360 = 0.144 m^3/s

Offsite:

5 $year_{OS1}$ = 0.37 x 62.04 x 0.135 / 360 = 0.0086 m^3/s
100 $year_{OS1}$ = 0.45 x 105.81 x 0.135 / 360 = 0.0179 m^3/s

5 $year_{OS2}$ = 0.39 x 62.04 x 0.360 / 360 = 0.0242 m^3/s
100 $year_{OS2}$ = 0.47 x 105.81 x 0.360 / 360 = 0.0498 m^3/s

5 $year_{tot}$= 5 $year_{OS1+}$5 $year_{OS2}$ = 0.0086 m^3/s + 0.0242m^3/s = 0.0328 m^3/s

100 $year_{tot}$ = 100 $year_{OS1+}$100 $year_{OS2}$ = 0.0179 m^3/s + 0.0498 m^3/s = 0.0677 m^3/s

It is noted that the total pre-development runoff rate directed to the neighbouring properties to the west consists of the runoff from catchment areas Pre-CA2, Pre-CA3, OS1 and OS2 and is equal to 0.100 m^3/sec and 0.212 m3/sec for the 5 year and 100 year events respectively.

2.3.3 Total Allowable Runoff Rate

Based on the stormwater management criteria, the total stormwater runoff from the site during post-development conditions for 100 year storm event must be less than or equal to the stormwater runoff from the site during pre-development conditions for a 5 year storm event.

The total allowable runoff rate was established using the rational method. Calculations are provided in Appendix A – Sheet 1 Allowable Release Rate and SWM Summary. Since each of the pre-development catchment areas has a different time of concentration, the peak runoff rate from each catchment area will be non-coincident and as such would not directly add together. As such, the average time of concentration for the two larger onsite pre-development catchment areas was used for the purposes of the calculation. A storm event with 24.3 min duration yields intensity of 62.04 mm/hr for a 5-year storm event and of 105.81 mm/hr for a 100-year storm event.

The total allowable runoff for the site based on the pre-development runoff conditions includes both the onsite and offsite contributing areas is equal to:

For the 5-year Storm Event

5-year onsite+5-year offsite

=0.0669+0.0328
= 0.0997 m^3/sec

For the 100-year Storm event
= 5 year pre-development
= 0.0997 m^3/sec

2.3.4 Post Development Site Conditions

The proposed development will consist of a 599.5 square metre residential building (700 sqm of roof area including overhang), 110 square metres of sidewalk and walkways and 1756 square metres of parking and roadway areas. All remaining areas will be grassed/landscaped areas. The proposed building will be serviced by a Class 4 onsite septic system and a private water service connected to the water main on Rockdale Road.

The proposed building will be constructed with a basement level parking garage which will be accessed by means an access ramp which slopes downward from the main parking level. The front of the building will be designed with an accessible entrance ramp. The site will be accessed from Rockdale Road by means of an about 110 m long by 6.7 m wide roadway which terminates at the surface parking area.

The stormwater management facility designed to provide quality and quantity control for the portion of the site including the building and proposed access roadway will consist of shallow low sloped swales on either side of the roadway and parking area. Grading immediately adjacent the building will be designed to direct the runoff from the building and the developed area around the building into the low-sloped swales. Landscaped areas towards the north of the proposed building will provide quantity control for the site by means of an infiltration bio-swale.

Runoff from the remaining portions of the site will flow to the west uncontrolled.

2.3.4.1 Post Development Catchment Areas

For the purposes of the stormwater management design, the proposed development has been divided into two controlled area catchments and one uncontrolled area catchments as shown on drawing 220863-POST.

The controlled area Post-C1 consists of the building, surface parking area, access roadway and adjacent parking. Runoff from this area will be directed to the shallow swales along the sides of the access roadway. The controlled area Post-C2 will consist of the remainder of the parking area neglecting the basement access ramp area, and the developed area to the north of the basement access ramp. Runoff this area will be directed to a infiltration bio-swale.

A relatively narrow strip of the landscaped area along the west side of the site will remain uncontrolled. Additionally, the hard surface area from the basement access ramp will drain via sheet flow towards the west. That is runoff will flow from this area without restriction along the existing pre-development flow patterns. The “No-Development Zone” to the north portion of the site will also remain uncontrolled. As previously indicated, since the “No-Development Zone” to the north is not subject to development, it has been omitted from the stormwater management calculations.

There were two offsite catchment areas identified in Section 2.3.2.2 above. Based on the existing drainage patterns, offsite catchment area OS1 will contribute runoff to the stormwater management facility in the POST-C1 catchment area. The second offsite catchment area OS2 will direct runoff via sheet flow and conveyance swale into the stormwater management facility in POST-C2. Any additional off site runoff from the parcels to the east of the subject site will continue to direct runoff into the “No-Development Zone” will not be disrupted by the proposed development.

2.3.5 Post-Development Time of Concentration.

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

2.3.5.1 Post Development Runoff Coefficient

The post-development runoff coefficients were calculated for POST-CA1 and POST-CA2 with the results of the calculation summarized in Table 2-3 below. Since the offsite areas OS1 and OS2 discharge into POST-CA1 and POST-CA2 respectively, the post-development runoff coefficient were also calculated for the post development conditions inclusive of the offsite areas. Since the offsite areas remain unchanged as a result of the development, the runoff coefficient for the offsite area does not change. It is noted that the inclusion of the offsite areas in the runoff coefficient increases the weighted average runoff coefficient for each of POST-CA1 and POST-CA2.

Table 2-3 Post-Development Runoff Coefficients

Catchment Label	Post-C1 (inclusive of OS1)	Post-C2 (inclusive of OS2)	Post-C1 (independent of OS1)	Post-C2 (independent of OS2)	Post-UC	OS1	OS2
Catchment Area	0.784	0.370	0.648	0.361	0.154	0.239	0.453
5-Year C	0.53	0.32	0.51	0.30	0.43	0.37	0.39
100-Year C	0.63	0.40	0.59	0.38	0.51	0.45	0.47

2.3.6 Uncontrolled Area Runoff

Flow from the uncontrolled area will be directed without restriction towards the west and northwest property line of the site via sheet flow, which matches the pre-development runoff conditions. The maximum allowable release rate for the controlled areas equals the allowable post-development runoff rate minus the 100-year runoff rate from the uncontrolled portion of the site. The runoff from the uncontrolled areas was determined using the rational method for a time of concentration of 10 minutes using the above calculated runoff coefficients. A time of concentration of 10 minutes corresponds to storm intensities of 104.19mm/hr and 178.56 mm/hr during the 5-year and 100-year design storm events respectively. The runoff rate from the uncontrolled areas was calculated using the Rational Method.

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

The uncontrolled runoff for the 5 year and 100 year design storm events are as follows (calculations are provided in Appendix A):

The uncontrolled runoff is:

5-year = 0.43 x 104.19 x 0.154 / 360 = 19.2 L/s
100-year = 0.51 x 178.56 x 0.154 / 360 = 39.1 L/s

2.3.7 Allowable Release Rate

The City of Ottawa requires that post-development stormwater runoff rate during a 100 year design storm event be limited to be less than or equal to the pre-development runoff rate, calculated assuming a maximum runoff coefficient of C=0.5, during a 5 year design storm event. To control runoff from the site it will be necessary to limit post-development flows, from the controlled areas, for all design storm events up to and including the 100-year event using onsite inlet controls.

The allowable release rate from the controlled areas of the site is equal to the total allowable runoff rate from the site less the runoff rate from the uncontrolled areas.

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

For the 5-year Storm event
$Q_{controlled}$= 99.7 – 19.2 L/s = 80.5 L/s

For the 100-year Storm event
$Q_{controlled}$= 99.7 – 39.1 L/s = 60.6 L/s

2.3.8 Consideration for Post-development Runoff from Off-Site.

As previously indicated the existing residential properties northeast of the site have been developed with single family dwellings and associated accessory buildings. Runoff from the first about 30 metres of each property is direct by sheet flow towards the roadside ditch on Rockdale Road. The remainder of the parcels discharge runoff uncontrolled via sheet flow towards the subject site towards sub-catchments POST-CA1 and POST-CA2. The existing runoff that discharges towards the “No-Development Zone” will continue to do so in post-development conditions unchanged from pre-development conditions following the completion of the development.

Since a portion of the adjacent properties direct runoff onto the subject site, the proposed grading of the subject site has to accommodate this runoff. The development of the subject site cannot negatively impact the adjacent properties.

The proposed grading plan was designed in keeping with this constant.

2.3.9 Description of Storage and Outlet Control

In order to meet the stormwater quantity control restriction, the post development runoff rate from the controlled areas of the site cannot exceed the allowable release rate to the adjacent respective ditches. Runoff in excess of the allowable release rate will be detained and temporarily stored on the site to be released a controlled rate during and following a storm event.

The stormwater management calculation sheets included in Appendix A were generated to determine the maximum storage requirement for each catchment area:

- On the required storage vs release rate calculation sheets: For catchment area POST-CA1, the storage requirement for a series of design storms was determined as a function of the release rate from the catchment area for each return period. As an Example: For the purposes of this sheet, each time duration of the 5 year storm event is considered to be an individual design storm. When considering a storm event with a 5 year return period for catchment POST-CA1, the maximum storage requirement for a release rate of 10 L/s will occur for a design storm with duration of 100 minutes and for a release rate of 25 L/s will occur for a design storm with duration of 40 minutes.
- On the outlet control design sheet: The available cumulative storage volume in the north and south storage swale is calculated with respect to the ponding level elevation in the swale. Since the discharge rate from the site through the combined swales is a function of the head on the outlet control device, the discharge rate from the swales is also calculated with respect to the ponding elevation.
- The storage discharge curve chart was generated to overlay the maximum storage requirement vs discharge rate curve for each return period (calculated on the required

storage vs release rate sheet) on the available storage volume vs discharge rate curve (calculated on the outlet control design sheet). The point where the curves cross provides the maximum storage volume and discharge rate for each return period considered.

Stormwater storage for the purposes of restricting the post-development runoff rate to the pre-development runoff rate for each storm event will be provided for catchment area POST-CA1 within the stormwater storage swales along both sides of the driveway and for catchment area POST-CA2 within a bio-swale located north of the proposed building. The proposed storage swales and bioswale have been designed in conjunction with the quality control criteria to ensure that both the quantity and quality control criteria will be met.

The stormwater storage and conveyance swales in both catchment areas have been designed based on the following:

- A field investigation completed by Kollaard Associates Inc, determined that the underlying soils consist mostly of fine to medium sand with trace to some silt. Silty clay fill was observed overtop of the native soils within the upper 0.30 metres at various locations within the site. Below the topsoil and/or silty clay fill (where encountered), the soils consisted of a red brown fine to medium sand with trace to some silt transitioning to a grey brown fine to medium sand with trace to some silt. Tactile examination of the samples taken indicated that the sand soils were non-plastic.
- The BLEL report indicates that “the native sand has a percolation time of T=6 min/cm (The percolation rate of the native soil "T=6" was obtained from the geotechnical report by Morey Assoc. Ltd. for this site, dated Sept. 2013, file # 013300).”
- Kollaard Associates obtained sand samples from 3 locations on the site, one each from the conveyance swale locations, bioswale location and proposed septic field area. The results of the grain size analysis completed on the sand samples in combination with the results of the tactile examination indicate that the native sand in these areas would have an estimated percolation time of 10 min/cm.
- Comparison of the results of the particle size analysis with the grain size distribution charts provided in the Ontario Building Code Supplementary Standards SB-6 indicates that the sand classification ranges from SP – poorly graded sand to SM – silty sand / sand silt mixture in accordance with the Unified Soils Classification System. SP soils are expected to have a percolation time of T of 2 to 8 min/cm and a coefficient of permeability of K = 10-1 to 10-3 cm/sec and SM soils are expected to have a percolation time T of 8 to 20 min/cm and a coefficient of permeability of K = 10-3 to 10-5 cm/sec depending on clay content and plasticity.
- For the purposes of stormwater management modelling, the native sands and silty sands were assumed to have a percolation time of 10 min/cm and a coefficient of permeability in the order of k = 10-3 cm/sec = 10-5m/sec.

- The swales will be constructed by excavation into the native sand soils at the site. Any imported material used in the swale construction should have a similar gradation to the native soils. Any fill material encountered within the foot print of the swales that does not have a gradation similar to the native soils should be removed from the vicinity of the swales.

2.3.9.1 Storage and Outlet Control – CA1

The runoff from the front half of the building roof is being directed towards the surface of the asphaltic surfaced parking and roadway, which is continuously sloped from the building to the curb along the outside edge of the pavement area. The slope from the building to the edge of the pavement area is continuously sloped at between 1.4 and 4.4 percent. The roadway is also sloped towards the strategically placed low areas along the south side of the access roadway.

The remaining portion of the roof is being directed towards a conveyance swale, which will be directed towards the stormwater storage swale.

As a result of the proposed grading, stormwater will be conveyed towards curb cuts within the parking lot surface and in the proposed low areas along the access road, which allows stormwater runoff to enter stormwater storage swales. Because of the low slope of the storage swales, a sub drain has been proposed along the swales, complete with catch basins comprised of NYLOPLAST T-fitting and Standard-H20 Drain Grates (product codes 1565AN and 1501DI) included in Appendix E. The sub drain will consist of HDPE pipe with a diameter of 250mm. The sub drain will be connected to the catch basins by standard HDPE fittings. The sub drain network shall be constructed as per City Standard Drawing S30 and S31.

As a conservative approach, the storage capacity of the sub drain pipe has been omitted from the stormwater storage calculations. It is noted that the sub drain is not intended to convey the flow resulting from the design events without surcharge. The sub drain is intended to facilitate the low slope of the swales and reduce the duration of surface ponding following a storm event. From the attached storm sewer design sheets, there is sufficient capacity for surface storage within the swales to temporarily detain stormwater runoff and release at a controlled rate for all events up to an including the 100-year storm event, without including the capacity of the sub drain storage. The stormwater storage swales have been designed as follows:

- The storage swale along the south side of the site will have a flat bottom with a width of 0.5 metres.
    - The side slopes of the storage swale will extend down to the bottom of the swale from the existing ground surface along the property line and from the edge of the parking lot/driveway pavement structure.

  - The side slopes will be covered with a topsoil layer having a minimum thickness of 0.1 metres and will be seeded with grass.
  - The longitudinal slope of the storage swale is 0.3%. Due to the shallow swale conditions, a subdrain has been proposed within the swale to prevent ponding water.
- The storage swale along the north side of the access road will have a flat bottom with a width of 0.5 metres
  - The side slopes of the storage swale will extend down to the bottom of the swale from the existing ground surface along the property line and from the edge of the driveway pavement structure of the access road.
  - The side slope will be covered with at topsoil layer having a minimum thickness of 0.1 metres and will be seeded with grass
  - The longitudinal slope of the storage swale is 0.3%. Due to the shallow swale conditions, a subdrain has been proposed within the swale to prevent ponding water.
- The subdrain for the above mentioned swales will be constructed according to City of Ottawa Standard Drawing S29 for reference and as follows:
  - The top of the clear stone layer will be below 300mm of native sand backfill and 100mm of topsoil and is to be seeded with grass.
  - The subdrain will extend a total of 1.0 metres below the topsoil and will be comprised of 0.50 metres width of clearstone within the native sand.
  - A 250 mm diameter HDPE perforated pipe will be located in the 0.5 metre width of clearstone.
  - The bottom of the clearstone will extend 0.15 metres below the perforated pipe.
  - The clear stone will be wrapped with a 6 ounce per square yard non-woven geotextile fabric.
- The south and north swales are connected by means of a 300 mm HDPE storm pipe located about 15 m west of Rockdale Road.
- The depth of infiltration is only considered for the upper metre of soil.
- The flow rate through the bottom of the subdrained swale was calculated as:

  $Q = A\,k\,i$

  Where A = combined surface area of the swales at the ponding level increment

  k =coefficient of permeability = 1.67×10^{-5} m/s

  i = (h+d)/d where d is the upper 1.0 m of soil below the storage area and h is the ponding depth at the ponding level increment. Calculations are provided in Appendix A.
- Discharge from the storage swales will be controlled by an outlet structure located at the east end of the south swale. The outlet structure will consist of Storm Manhole STM-MH1 complete with a 4 m long 400mm diameter inlet pipe and a 4 m long 400 mm diameter outlet pipe. The inlet pipe will have an invert of 77.30 m, has been designed with no slope

and will have a debris grate at the inlet. The outlet pipe will be fitted with an plug type orifice plate. The orifice is 150mm in diameter with an invert of 77.56m. The outlet pipe discharges to the roadside ditch along Rockdale Road at an elevation of 77.53 m.

- Discharge from the swale below an elevation of 77.56 m is by infiltration only. As such this volume was not included in the calculation for available storage volume for quantity control purposes.
- Overflow from the swales will be by means of an overflow channel at the east end of the north swale which will discharge to the roadside ditch along Rockdale Road at an elevation of 77.85m.

2.3.9.2 Storage and Outlet Control – CA2

The runoff from the developed portion of the site within sub-catchment Post-CA2 is directed via sheet flow and a conveyance swale into an infiltration bio-swale along the northwest portion of the sub-catchment, south of the highpoint/ridge separating the no-development area. The bio-swale has been designed to temporarily store runoff generating during all storage events up to and including the 100-year storm event, with no runoff being discharged off of the site. In the event that a storm-event exceeds the 100-year storm event, an overflow weir has been proposed to allow for excess runoff to spill over into the "No-Development Zone" and will either infiltrate or runoff uncontrolled to the northwest.

The bio-swale will provide both quantity control and quality control. Since all of the runoff generated on Catchment POST-CA2 for all storm events up to and including the 100-year storm event being stored in the bio-swale with no discharge from the site, quality control is automatically achieved and no additional quality control calculations have been provided.

The bio-swale has been designed as follows:

- The bio-swale will have a flat bottom with a width of 15.0 and length of 28.0 metres, and an elevation of 77.00m.
- The side slopes of the bio-swale will extend down to the bottom of the swale from the existing ground surface to allow for sheet flow and runoff from the proposed conveyance swale to enter the bio-swale.
- The side slopes vary between 4H:1V to 5H:1V
- The side slopes will be covered with a topsoil layer having a minimum thickness of 0.1 metres and will be seeded with grass.
- Discharge from the swale will be by means of infiltration through the bottom of the bio-swale.
- The depth of infiltration is only considered for the upper metre of soil below the swale.
- Overflow from the bio-swale will be by means of an overflow channel, which has an invert elevation of 77.40m.

2.3.10 Summary of Post Development Flow Rate and Storage Requirements

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 2-4

Table 2-4 – Summary of Runoff Rates, Storage Requirement and Ponding Depth

Catchment Area ID.	Area	5 – year design Storm			100-year design Storm		
		Release Rate (L/s)	Required Storage (m^3)	Available Storage (m^3)	Release Rate (L/s)	Required Storage (m^3)	Available Storage (m^3)
POST-CA1	0.784*	12.3	226	329.3	13.2	258	329.3
POST-CA2	0.850**	0	112	244.4	0	146	244.4
Total Actual Controlled Area Release Rate							
		12.3			13.2		
Total Allowable Controlled Area Release Rate							
		80.5			60.6		
Uncontrolled Area Release Rate							
UC1	0.154	19.2	N/A	N/A	39.1	N/A	N/A
Total Post Development Runoff From Site (controlled and uncontrolled)							
	1.788	**31.5**			**53.2**		
Total Allowable Runoff From Site							
		99.7			**99.7**		

* The areas for the post development catchment area POST-CA1 is inclusive of the offsite area OS-1.
** The areas for the post development catchment area POST-CA2 is inclusive of the offsite area OS-2.

2.4 Stormwater Quality Control

Stormwater treatment of 80% TSS removal will be provided by a treatment train approach. Pre-treatment will be provided by best management practices and by vegetative filtration and sedimentation within the conveyance swales. Quality Control will be provided by detention and infiltration of the entire quality control volume generated on the controlled areas within the storage swales for POST-CA1 and within the Bio-swale for POST-CA2.

In 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* which provides the required quality control volume as a function of protection level, SWMP type and impervious level.

2.4.1 Quality Control for catchment area POST-CA1

As previously indicated, runoff from the building, parking area, roadway and landscaped ground surface areas in catchment area POST-CA1 will be directed to the subdrained storage swales extending along both sides of the driveway. The subdrained swales provide quality control storage and discharge to the roadside ditch at the front of the site. The quality storage swales have been designed to outlet the quality storage volume vertically through infiltration.

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	~ 0.784 hectares (includes treating the area from OS1
Pre-treatment	Longevity	Pre-treatment by means of sedimentation chamber, or forebay, vegetated filter strip, swale or oil/grit separator	Pre-treatment by vegetated filteration on grassed side slope and grassed bottom of swale.
Storage Depth	Avoid Filter Compaction	Subsurface sand and organic filters: 0.5 m Maximum 1.0 m	Maximum storage depth of ~0.25 m for quality storm. Maximum storage depth of ~0.52m for quantity storm
Filter Media Depth	Filtering	Sand: 0.5 m	Native Sand: >1.0m
Under-drain	Discharge	Minimum 100 mm perforated pipes bedded in 150 – 300 mm of 50 mm gravel	250 mm perforated pipe in minimum 600 mm of 25-50 mm clear stone.
Land use		any land use, often employed for commercial and industrial	Residential
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 for the catchment area of each swale
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 material and filter and between filter and clearstone
Overflow / by-pass		required	overflow is provided above the quality storage requirement
Drawdown time	prevent standing water	maximum from 24 to 48 hours 24 hours preferred	Design drawdown time of about 7.6 hours following a quality storm event

2.4.1.1 Volumetric Sizing and Filter Size

From Table 3.2 under infiltration it was determined that the water quality storage requirement for a 35 percent impervious ratio at an enhanced level of treatment is 25 cubic metres per hectare. The catchment area POST-CA1 has an impervious ratio of 28% when including the uncontrolled runoff from catchment OS1. Since the minimum impervious area on Table 3.2 is 35%, the quality control sizing was determined to be based off of the 35% impervious area criteria. Based on a quality storage requirement of 25 cubic metres per hectare and the surface area of the site, the total water quality storage requirement is 19.6 cubic metres.

The manual however requires that by-pass does not occur below a 4 hr 15 mm design event. In order to ensure that by-pass would not occur below a 4 hr 15 mm design event, the subdrained storage swales were designed to accommodate the entire volume of a 15 mm rainfall originating on catchment area POST-CA1 in the storage swale below the outlet invert. It is noted that a runoff coefficient of 0.53 indicates that only 53% of the rainfall will result in runoff.

The total area contributing to the subdrained storage swales is about 7840 square metres including the offsite area. Since the swales on the north and south side of the access roadway are interconnected with a culvert, the storage for both swales were considered to be a single swale for storage calculation purposes. A 15 mm storm event will result in a runoff volume of:

$V = 7840\ m^2 \times 15mm \times 0.53 = 62.3\ m^3$

This represents a conservative estimate, as the 62.3 m^3 is representative of an instantaneous 15mm storm event, and the swales becoming filled with the 53% calculated runoff with no infiltration occuring.

Quality assurance will be provided by filtration through the sides and bottom of the south and north conveyance swales in catchment POST-CA1. As originally indicated, the outlet structure was designed to discharge 0.28m above the bottom of subdrained storage swales. This results in a storage volume of 67.5 m^3 that will outlet by infiltration only.

2.4.1.2 Draw Down Time – Subdrained Storage Swales

As previously indicated, the native fine to medium sand with trace to some silt were determined to have a coefficient of permeability of 1.7×10^{-5} m/s.

The flow rate through the bottom of the subdrained storage swale was calculated using the derivative of Darcy's Equation provided in Section 4.6.7 of the MOE Manual.

$$A = \frac{1000\ V\ d}{k(h+d)t}$$

Where A = Average surface area of ponded water below outlet = 589.1 m^2
V = design volume (volume below outlet) = 67.5 m^3
d = depth of controlling filter medium (native sand to 1 m below storage swale) = 1m
h = operating head of water (varies between 0 and 0.28) = 0.14 m
k = coefficient of permeability = 60 mm/hr
t = draw down time in hours

Rearranging the above equation provides the following:

$$t = \frac{1000\,V\,d}{k(h+d)A} = (1000 * 67.5 * 1) / (60*(1+0.14)*589.1) = 1.68 \text{ hrs.}$$

2.4.2 Quality Control for catchment area POST-CA2

As previously indicated, runoff generated in catchment area POST-CA2 are comprised of primarily landscaped areas, with the exception of one small portion of the access road, which terminates at the border of the catchment area. Since discharge which enters the swale discharges solely by infiltration and is adequate to store runoff for storm events up to an including the 100-year storm event, it can be concluded that the infiltration pond meets the quality criteria for the site.

2.5 Maintenance

2.5.1 Storage Swale Subdrains

As previously indicated, surface runoff from the parking area will be directed by sheet flow and by swales to the stormwater management swale. Catchbasins will be used for inspection/clean out of the subdrain, which consist of a 250mm diameter HDPE perforated pipe. The catch basins should be inspected on a monthly basis to remove debris. Sediment levels should be measured on an annual basis. When sediment builds up more than 0.15 metres it should be removed by hydrovac.

2.5.2 Storage Swales amd Bio-swale

The swales should be inspected on a weekly basis and after any rain fall event after construction until vegetation is well established. Any areas of erosion or distress should be repaired immediately.

Once plants mature, the maintenance of the swales will be very low.

Inspect the swales after large storm events and at least monthly for improper water drainage, berm settling, soil erosion, as well as invasive plants. Any debris and invasive plants should be removed from the swale if present.

Always water plants throughout the first year after planting until they are established. Species should be able to tolerate dry conditions on their own afterward. However, if plants become distressed additional hand watering may be required.

The grassed side slopes of the swales should be subjected to the same maintenance schedule as the remainder of the grass covered landscaped "lawn" surfaces. That is, the grass should be mowed and cared for as required to maintain a normal healthy appearance. Minimum recommended grass height in the swales is 75 mm.

Cut away excessive dead plant material from living plants and for species that have died or may need replacement if they are not thriving. Shrubs and trees may need occasional pruning depending on desired aesthetics. Some plants may need to be divided if plantings become too crowded.

Removal of accumulated sediment from the swales should be conducted when the accumulation of the sediment begins to significantly affect the quality of the plant growth and/or the drainage patterns along the grassed surfaces. Sediment should be removed in relatively narrow sections such that plants can be replanted in a cleaned section of the swale in order to preserve the vegetation.

If long term ponding occurs within the storage, the engineer should be notified. At this point the engineer could make an assessment of the material in the first metre below the bottom of the swales. If the assessment indicates that the native soil and/or subdrains have become compromised with sediment, the swale and subdrains will require maintenance.

2.5.3 Winter Operation

The MOE Manual indicates that filters suffer in performance during winter operation due to freezing of the filter medium. As previously indicated, Filters receive runoff from parking areas and roads which are subject to sanding and salting.

During winter operation, the predominant sediment load on the storage area will result from sand placed during de-icing salting/sanding of the parking and gravel surfaces of the site and from sand carried onto the site from vehicles. During spring melt, the sediment will be transported towards the storage area. The runoff will be directed over the grasses side slopes of the swales. Sedimentation within the grassed side slopes will provide pre-treatment reducing the sediment load on the storage swale.

The sediment and particulate matter resulting from these sanding and salting operations tend to be coarser in nature and are more prone to sedimentation within the grass surfaces immediately adjacent to swales. As such, during winter operation, the primary quality control mechanism will be storage and sedimentation as opposed to filtration.

3 WATER DEMAND

The site is to be serviced by a 200mm diameter water service to be connected to the 203mm diameter PVC watermain on Rockdale Road. The service will be connected by means of a "T" connection as indicated on the site servicing drawing 220863-SER. The water demand consists of two parts which include: domestic water consumption and fire flow requirements. The proposed service will reduce to two 50mm diameter water services by means of a "T" connection on the south side of the access roadway immediately east of the asphalt parking area. The first 50mm diameter water service is for the proposed building. The second 50mm diameter service is to be capped for potential future development north of the site.

3.1 Water Demand – Domestic Water Demand

Referencing the previous report prepared by BLEL, the proposed development will consists of 12 units. 6 of the units are to be 1-bedroom, and 6 units are to be 2-bedroom. The water demand for the proposed development was based on the City of Ottawa Water Distribution Design Guidelines (as amended) as follows:

Residential

6 Units @ 1.4 PPU = 8.4 People
6 Units @ 2.1 PPU = 12.6 People

Total = 21 People

It is noted that the septic design assumption, and the previous design was completed conservatively assuming a population of 36 people (2 people per bedroom). As such, the population of the building was assumed to be 36 people.

Average Daily Demand (ADD) = 36 People x 280 L/c/dayx(1/86,400 sec/day) = 0.117 L/s
= 0.12L/s

City of Ottawa calculates the Maximum Daily Demand (MDD) demand to be 2.5 x ADD

MHD = 2.5 x ADD
= 2.5 x 0.117 litres/seconds
= 0.29 litres/seconds

Peak Hourly Demand (PHD) (factor of 2.2) = 0.29L/s x 2.2 = 0.64 L/s

3.2 Water Demand – Fire Fighting Supply and Storage

Based on the previous submission from BLEL, it is the understanding of Kollaard Associates that the proposed development will not have a fire suppression sprinkler system. As such, no sprinkler demand is required for the site.

3.2.1 Building Construction and Construction Type Consideration

The proposed building is 2 storeys and assumed to be of wood frame construction with a foot print of 599.5 m^2. The footprint of the upper storey is assumed to be the same as the footprint of the lower storey. This results in a total building area of 1196 m^2. The interior walls between units will be constructed having a minimum rating of 1 hour. Each floor within the building will also be constructed with a minimum fire rating of 1 hour. The exterior wall will be covered with non combustible cladding, have non combustible insulation and will be covered on the interior with 16 mm Type X drywall. This results in an exterior wall assembly having a minimum fire rating of at least 1 hour.

3.2.2 Fire Flow and Hose Stream Demand

The fire flow requirement was calculated for the proposed buildings to ensure that there is adequate flow available to put out a fire within the proposed building should it occur. The fire flow calculation determines the minimum water flow or volume required to be available for firefighting purposes to be used by firefighters. In accordance to the City of Ottawa Technical Bulletin ISTB-2021-03, the fire flow requirement calculation for private property is to first consider the Ontario Building Code (OBC). If the fire protection requirement from the OBC yields a fire flow greater than 9000L/min, then the Fire Underwriters Survey (FUS) shall be used to determine Fire Flow Demand.

Technical Bulletin ISTB-2021-03 provides the following direction with respect to the calculation of the fire flow requirements:

> "The requirements for levels of fire protection on private property in urban areas are covered in section 7.2.11 of the Ontario Building Code. If this approach yields a fire flow greater than 9,000 L/min then the Fire Underwriters Survey method shall be used to determine these requirements instead."

The fire flow demand calculation has been included in Appendix B. Based on the OBC Calculations with the above mentioned building construction, the fire flow requirements for the proposed building is 4,500 L/min or 75.0 L/s. The OBC requires a fire flow duration of 30 min per A-3.5.2.7(3)(a). It is noted that when referencing the Fire Underwriters Survey 2020, for a fire flow of 4500L/min (75 L/s) would results in a required fire flow duration of 1.625 hours.

3.3 Boundary Conditions

The boundary conditions were previously obtained from the City of Ottawa by BLEL. The boundary conditions have been attached in Appendix D of this report. The provided boundary conditions area as follows:

- Basic Day Average = 115.4m
- Minimum Pressure During Basic Day = 108.4m
- Peak Hour on Max Day = 119.3m
- Amount of fire flow = 95L/s, Max Day + Fire = 93.6 m, (~21 psi)
- Amount of fire flow = 90L/s, Max Day + Fire = 98.3 m, (~28 psi)

A 3 hour fire flow of 95 L/s at max day would drop the pump station clear wells to 30%, assuming a starting point of 75%.

3.4 Combined Total Flow Demand

The Max hourly demand is 0.64 /s
The fire flow demand is 75.00 L/s

Total Water Demand is 75.64 L/s.

3.5 Water Service Requirements and Pressure Loss Calculations

The maximum and minimum pressures were determined for both the mechanical room (water entry point), assumed to be in the basement parking garage level of the building and the second floor using seven water demand scenarios.

During the first scenario, only the residential water demand was considered. The minimum pressure was determined using the minimum HGL and the average daily water demand of 0.12 L/s. The scenario analyzed the pressure along the 200mm diameter water service from the main on Rockdale Road to the private fire hydrant. The scenario then analyzed the pressure along the 50mm service from the hydrant to the mechanical room assumed to be in the basement parking garage level of the structure.

During the second scenario, only the residential water demand was considered. The minimum pressure was determined using the minimum HGL and the peak hourly water demand of 0.80 L/s. The scenario analyzed the pressure along the 200mm diameter water service from the main on Rockdale Road to the private fire hydrant. The scenario then analyzed the pressure along the 50mm service from the hydrant to the mechanical room assumed to be in the basement parking garage level of the structure.

During the third scenario, only the residential water demand was considered. The maximum pressure was determined using the Peak Hour on Max Day HGL and the peak hourly water demand of 0.64 L/s. The scenario analyzed the pressure along the 200mm diameter water service from the main on Rockdale Road to the private fire hydrant. The scenario then analyzed the pressure along the 50mm service from the hydrant to the mechanical room assumed to be in the basement parking garage level of the structure.

Scenario 4-6 repeats the analysis completed in Scenario 1-3 from the watermain on Rockdale Road to the second floor of the structure.

During the seventh scenario, the fire flow was considered in addition to the residential water demand. The minimum pressure was determined using the Max Day + Fire Flow HGL, which were provided as 98.3 m at 90L/s. Since the required fire flow of the building is 75L/s, the HGL was assumed to be 99.0 m and the average daily water demand of 0.15 L/s. The scenario analyzed the pressure along the 200mm diameter water service from the main on Rockdale Road to the private fire hydrant. The scenario then analyzed the pressure along the 50mm service from the hydrant to the mechanical room assumed to be in the basement parking garage level of the structure.

The pressure loss to the mechanical room and to the second floor of the proposed building was calculated using Bernoulli's Equation in Combination with the Darcy – Weisbach Equation and the Colebrook Equation. The equations are shown below.

$$H_P + Z_1 - Z_2 + \frac{P_1 - P_2}{S} + \frac{V_1^2 - V_2^2}{2g} = h_f + h_m \quad \text{where:}$$

$$h_m = K_m \frac{V^2}{2g} \qquad \text{Re} = \frac{VD}{\nu} \qquad Q = VA \qquad A = \frac{\pi}{4} D^2$$

$$\text{Darcy-Weisbach Equation: } h_f = f \frac{L}{D} \frac{V^2}{2g} \quad \text{where:}$$

$$\text{If laminar flow} \left(\text{Re} < 4000 \text{ and any } \frac{e}{D} \right), \quad f = \frac{64}{\text{Re}}$$

$$\text{If turbulent flow} \left(4000 \le \text{Re} \le 10^8 \text{ and } 0 \le \frac{e}{D} < 0.05 \right), \text{ then}$$

$$\text{Colebrook Equation: } \frac{1}{\sqrt{f}} = -2.0 \log \left(\frac{e/D}{3.7} + \frac{2.51}{\text{Re}\sqrt{f}} \right)$$

In general conformance with the MOE Guidelines, and City of Ottawa Technical Bulletin ISD-2010-2, the desired range in pressure should be approximately 350KPa (50psi) to 480KPa (70psi) during normal operating conditions. Additionally the distribution system shall be sized so that under maximum hourly demand conditions the pressures are not less than 276 kPa (40 psi.). As per the Ontario Building Code, the maximum pressure should not exceed 552KPa (80psi).

Based on the results of the analysis as presented in the above tables, when using 200 mm diameter service between the main and the fire hydrant, and then reducing to a 50mm diameter service lateral to the building, the above minimum and maximum HGL provide a water pressure of between 277 KPa and 400 kPa in the mechanical room and between 228 kPa and 335 kPa on the second floor of the proposed building. The minimum residual pressure, calculated at the mechanical room, when there is fire flow demand would be 152 kPa or above the minimum of 137 kPa. Since the pressure at the building is above the minimum pressure of 137 kPa during fire flow and 277kPa during maximum hourly demand conditions, the water system meets the requirements for minimum pressure. Since the water pressure on the second floor is below 276 kPa during normal conditions, a booster pump may be required to achieve satisfactory pressure on the upstairs fixtures.

The City Boundary Conditions are provided based on computer modeling of the water network. During construction, a pressure check is to be completed to determine that the pressure in the system at the building does not exceed 552 KPa. If the pressure does exceed 552 Kpa a pressure reducing valve would have to be installed downstream of the isolation valve and water meter in the building.

4 SANITARY SERVICE

No municipal sanitary services are available at this site.

As per Ontario Building Code (OBC) table 8.2.1.3.A, the daily design sanitary sewage flow for the proposed occupancy is 9,900 litres/day. Sanitary sewage will be disposed of in an onsite Class 4 sewage system with a level IV treatment unit. The onsite system will include a partially raised Pressurized Shallow Buried Trench disposal field preceded by a Waterloo Biofilter Basket Biofilter Model AD-BA100. A sewage system application has been prepared for approval through the Rideau Valley Conservation Authority (RVCA) (formerly the Ottawa Septic System Office).

The septic system design has been submitted to the RVCA for Permit. It is noted that the permit lapses 12 months following the date of issue.

4.1 Contribution to the Sanitary Sewer Demand from Floor Drains

If floor drains are to be installed in the basement level parking garage, the drains are not to be connected to the septic system. A 3200L storage tank has been proposed to collect runoff from the floor drains in the basement parking area of the proposed development. The storage tank should be inspected bi-monthly and pumped once full by a licensed service provider. The storage tank will discharge by pumping only. Effluent removed from the storage tank should be disposed of as contaminated water similar to that removed from an oil and grit separator.

5 EROSION AND SEDIMENT CONTROL

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 City of Ottawa, 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. 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, as shown in Kollaard Associates Inc. Drawing #220863-ESC Erosion and Sediment Control Plan. 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.

The proposed landscaping works should be completed as soon as possible. The proposed granular and 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.

6 CONCLUSIONS

Based on the analysis provided in this report, the conclusions are as follows:

Quantity Control measures have been implemented to ensure that post-development runoff during the 100-year storm event do not exceed the runoff during the 5-year storm event during pre-development conditions. An enhanced level of Quality Control will be achieved by means of vegetative filtration followed by infiltration.

The daily design sanitary sewage flow rate from the proposed development will be 9,900 litres/day. Sanitary sewage will be disposed of in an onsite Class 4 sewage system with a level IV treatment unit.

The facility is to be serviced by a 50mm diameter water service extending from a proposed 200 mm diameter water service, which will connect to the existing 203mm diameter water main along Rockdale Road.

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 or if we can be of any further assistance to you on this project, please do not hesitate to contact our office.

Sincerely,
Kollaard Associates Inc.

Prepared by:

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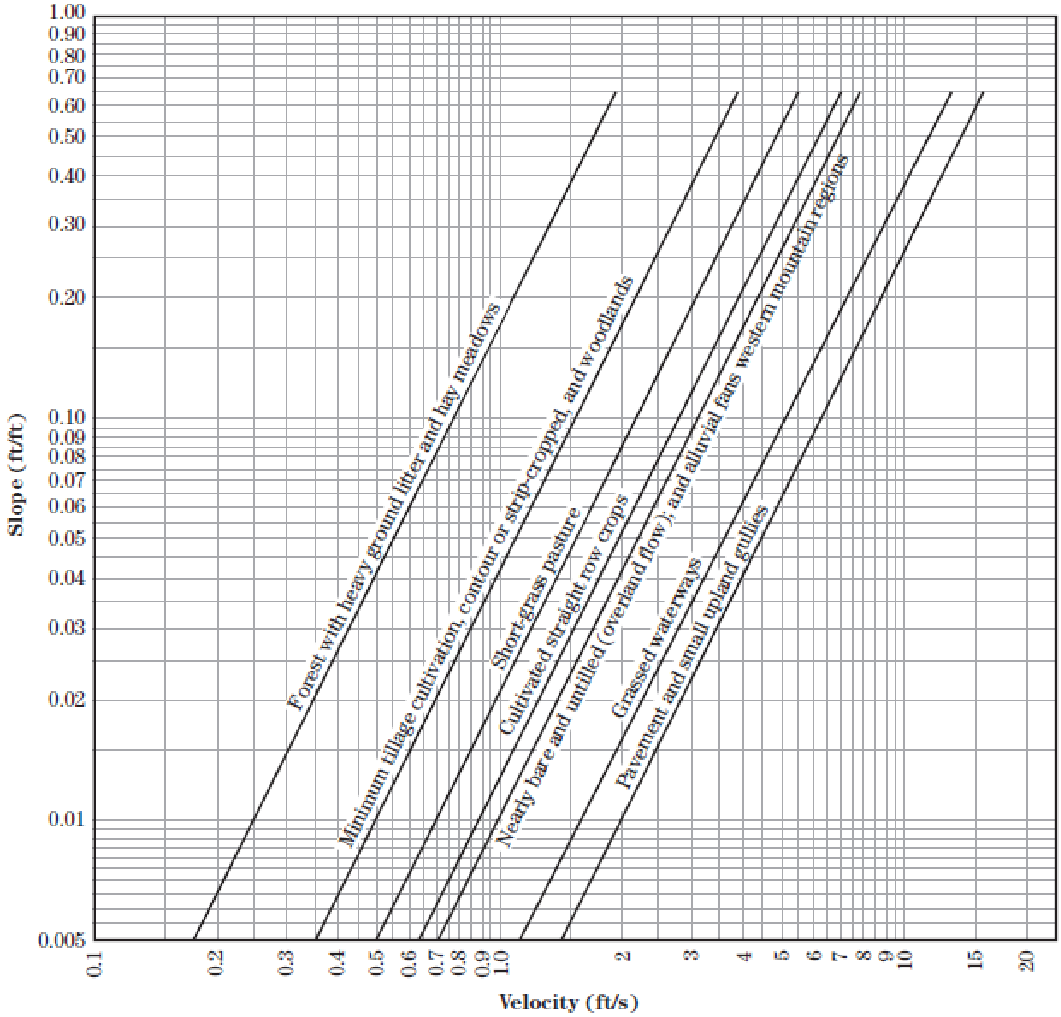
Appendix A: Storm Design Information

- Figure 15-4 of Chapter 15 of the USDA handbook
- Sheet 1 – Allowable Release Rate and SWM Summary
- Sheet 2 – Uncontrolled Area Runoff Rate Calculation
- Sheet 3 – Required Storage Vs. Release Rate (POST-CA1)
- Sheet 4 – Outlet Control Design Sheet – Swale (POST-CA1)
- Figure 1 – Discharge Vs. Storage Curve (POST-CA1)
- Figure 2 – Elevation Vs. Storage Curve (POST-CA1)
- Sheet 5 – Required Storage Vs. Release Rate (POST-CA2)
- Sheet 6 – Outlet Control Design Sheet – Swale (POST-CA2)
- Figure 3 – Discharge Vs. Storage Curve (POST-CA2)
- Figure 4 – Elevation Vs. Storage Curve (POST-CA2)

Figure 15–4 Velocity versus slope for shallow concentrated flow



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

Client: J.P.Bergeron
Job No.: 220863
Location: 5574 Rockdale Road, Vars
Date: December 21, 2023

Pre Dev run-off Coefficient "C"

Catchment	Area ha Total	Area - C 0.3	Area - C 0.9	Weighted Ave. C 5 year	100 year
Pre-C1	0.090	0.0904	0	0.3	0.38
Pre-C2	0.8034	0.8034	0	0.3	0.38
Pre-C3	0.3988	0.3988	0	0.3	0.38
Total	1.2926				
OS1	0.135	0.119	0.016	0.37	0.45
OS2	0.360	0.304	0.056	0.39	0.47
Total	0.495				

Catchment Area	**SHEET FLOW** Sheet Flow Surface Part 630 Table-15-1	Manningс n	length Sheet Flow (m)	2 yr 24 Hr Rainfall (mm)	Slope (m/m)	Time of Sheet Flow (hrs)	
PRE-C1	DENSE GRASSES	0.24	30	48.5	0.02	**0.30**	
PRE-C2	DENSE GRASSES	0.24	30	48.5	0.03	**0.26**	
PRE-C3	DENSE GRASSES	0.24	30	48.5	0.03	**0.26**	
OS1	DENSE GRASSES	0.24	30	48.5	0.02	**0.31**	
OS2	DENSE GRASSES	0.24	30	48.5	0.02	**0.32**	

	SHALLOW CONCENTRATED FLOW Flow Length to Major Channel (m)	Slope (m/m)	sqrt (slope)	Flow type Table 15-4	Depth (ft)	Mannings n	Velocity (m/s)	Time Shallow Conc. Flow (hr)	Total Tc (min)
PRE-C1	0.6	0.001	0.032	Overgrown / Un	0.2	0.101	0.05	**0.00**	**18.00**
PRE-C2	103	0.010	0.100	Overgrown / Un	0.2	0.101	0.15	**0.19**	**27.00**
PRE-C3	55	0.010	0.100	Overgrown / Un	0.2	0.101	0.15	**0.10**	**21.60**
OS1	47	0.019	0.137	Overgrown / Un	0.2	0.101	0.21	**0.06**	**22.20**
OS2	42	0.019	0.137	Overgrown / Un	0.2	0.101	0.21	**0.06**	**22.20**
								Average Pre-C2 & C3	24.30

	Tc used in Calc.	Intensity (mm/hr)		Flow (L/sec)	
	min	5 yr	100 yr	5 yr	100 yr
Pre-C1	24.30	62.04	105.81	4.7	10.1
Pre-C2	24.30	62.04	105.81	41.6	89.8
Pre-C3	24.30	62.04	105.81	20.6	44.6
Subtotal				66.9	144.5
OS1	24.30	62.04	105.81	8.6	17.9
OS2	24.30	62.04	105.81	24.2	49.8
Subtotal				32.8	67.6
			Total	**99.71**	**212.12**

STORMWATER MANAGEMENT SUMMARY

Sub Area I.D.	Sub Area (ha)	5 year C	100 year C	Outlet Location	5 Year (L/s)	Required 5 year Storage (m^3)	100 Year (L/s)	Required 100 year Storage (m^3)
Total Allowable Runoff Rate From Site					**99.7**		**99.7**	
Uncontrolled Runoff Rate from Site								
Uncont.	0.154	0.43	0.51		19.2		39.1	
Allowable Release Rate To Rockdale Road								
					80.5		*60.6*	
Disharge Rate From Swale to Rockdale Road								
CA1	*0.649*			*Rockdale Road*				
CA2	*0.361*			*Infiltration Area*	*0.0*		*0.0*	
Summary - Total Post-Development Runoff Rate and Storage Requirement								
TOTAL	**0.803**				**19.2**		**39.1**	

APPENDIX A: STORMWATER MANAGEMENT MODEL

Sheet2 - Uncontrolled Area Runoff Rate Calculation

Client: J.P.Bergeron
Job No.: 220863
Location: 5574 Rockdale Road, Vars
Date: December 21, 2023

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	Asphalt/ Ret. Wall	0.034	0.90	**0.43**	1.00	**0.51**
0.154	Landscape	0.120	0.30		0.38	
	Walkway	0.000	0.90		1.00	

Impervious Area Ratio 0.22

Post Dev Free Flow
5 Year Event

	C	Intensity	Area
5 Year	0.43	104.19	0.1544
2.78CIA=	19.23		
19.2	L/S		

**Use a 10 minute time of concentration

100 Year Event

	C	Intensity	Area
100 Year	0.51	178.56	0.1544
2.78CIA=	39.09		
39.1	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 (POST-CA1)

Client: J.P.Bergeron
Job No.: 220863
Location: 5574 Rockdale Road, Vars
Date: December 21, 2023

Post Dev run-off Coefficient "C" - CA1

			5 Year Event		100 Year Event	
Area (ha)	**Surface**	**Area (ha)**	**"C"**	**C_{avg}**	**"C" x 1.25**	**$C_{100\ avg}$**
Total	Roof	0.070	0.90	**0.53**	1.00	**0.63**
	Asphalt	0.141	0.90		1.00	
0.784	Sidewalk	0.011	0.90		1.00	
	Grass/Swale	0.562	0.30		0.38	
	Grass (OS-1)	0.136	0.37		0.46	

Impervious Ratio = 0.28

REQUIRED STORAGE VERSUS RELEASE RATE FOR 5 YEAR STORM

Runoff Coeffcient, C = **0.53**
Drainage Area (ha) = **0.784**
Return Period (yrs) = **5**
Duration Interval (min) = **10**
Release Rate Start (L/s) = **0**
Release Rate Interval (L/s) = **5**

		Release Rate -->	**0**	**5**	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	**Storage Required (m^3)**									
0	230.48	266.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	178.56	206.3	123.8	120.8	117.8	114.8	111.8	108.8	105.8	102.8	99.8	96.8
20	119.95	138.6	166.3	160.3	154.3	148.3	142.3	136.3	130.3	124.3	118.3	112.3
30	91.87	106.1	191.0	182.0	173.0	164.0	155.0	146.0	137.0	128.0	119.0	110.0
40	75.15	86.8	208.3	196.3	184.3	172.3	160.3	148.3	136.3	124.3	112.3	100.3
50	63.95	73.9	221.6	206.6	191.6	176.6	161.6	146.6	131.6	116.6	101.6	86.6
60	55.89	64.6	232.4	214.4	196.4	178.4	160.4	142.4	124.4	106.4	88.4	70.4
70	49.79	57.5	241.6	220.6	199.6	178.6	157.6	136.6	115.6	94.6	73.6	52.6
80	44.99	52.0	249.5	225.5	201.5	177.5	153.5	129.5	105.5	81.5	57.5	33.5
90	41.11	47.5	256.4	229.4	202.4	175.4	148.4	121.4	94.4	67.4	40.4	13.4
100	37.90	43.8	262.7	232.7	202.7	172.7	142.7	112.7	82.7	52.7	22.7	-7.3
110	35.20	40.7	268.4	235.4	202.4	169.4	136.4	103.4	70.4	37.4	4.4	-28.6
120	32.89	38.0	273.6	237.6	201.6	165.6	129.6	93.6	57.6	21.6	-14.4	-50.4
130	30.90	35.7	278.4	239.4	200.4	161.4	122.4	83.4	44.4	5.4	-33.6	-72.6
140	29.15	33.7	282.9	240.9	198.9	156.9	114.9	72.9	30.9	-11.1	-53.1	-95.1
150	27.61	31.9	287.0	242.0	197.0	152.0	107.0	62.0	17.0	-28.0	-73.0	-118.0
160	26.24	30.3	291.0	243.0	195.0	147.0	99.0	51.0	3.0	-45.0	-93.0	-141.0
170	25.01	28.9	294.7	243.7	192.7	141.7	90.7	39.7	-11.3	-62.3	-113.3	-164.3
180	23.90	27.6	298.2	244.2	190.2	136.2	82.2	28.2	-25.8	-79.8	-133.8	-187.8
190	22.90	26.5	301.5	244.5	187.5	130.5	73.5	16.5	-40.5	-97.5	-154.5	-211.5
Max. Storage Requirement =			301.5	244.5	202.7	178.6	161.6	148.3	137.0	128.0	119.0	112.3

REQUIRED STORAGE VERSUS RELEASE RATE FOR 100 YEAR STORM

Runoff Coeffcient, C = **0.63**
Drainage Area (ha) = **0.784**
Return Period (yrs) = **100**
Duration Interval (min) = **10**
Release Rate Start (L/s) = **0**
Release Rate Interval (L/s) = **5**

		Release Rate -->	**0**	**5**	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**
Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	**Storage Required (m^3)**									
0	398.62	547.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	178.56	245.2	147.1	144.1	141.1	138.1	135.1	132.1	129.1	126.1	123.1	120.1
20	119.95	164.7	197.6	191.6	185.6	179.6	173.6	167.6	161.6	155.6	149.6	143.6
30	91.87	126.1	227.1	218.1	209.1	200.1	191.1	182.1	173.1	164.1	155.1	146.1
40	75.15	103.2	247.6	235.6	223.6	211.6	199.6	187.6	175.6	163.6	151.6	139.6
50	63.95	87.8	263.4	248.4	233.4	218.4	203.4	188.4	173.4	158.4	143.4	128.4
60	55.89	76.7	276.3	258.3	240.3	222.3	204.3	186.3	168.3	150.3	132.3	114.3
70	49.79	68.4	287.1	266.1	245.1	224.1	203.1	182.1	161.1	140.1	119.1	98.1
80	44.99	61.8	296.5	272.5	248.5	224.5	200.5	176.5	152.5	128.5	104.5	80.5
90	41.11	56.4	304.8	277.8	250.8	223.8	196.8	169.8	142.8	115.8	88.8	61.8
100	37.90	52.0	312.3	282.3	252.3	222.3	192.3	162.3	132.3	102.3	72.3	42.3
110	35.20	48.3	319.0	286.0	253.0	220.0	187.0	154.0	121.0	88.0	55.0	22.0
120	32.89	45.2	325.2	289.2	253.2	217.2	181.2	145.2	109.2	73.2	37.2	1.2
130	30.90	42.4	330.9	291.9	252.9	213.9	174.9	135.9	96.9	57.9	18.9	-20.1
140	29.15	40.0	336.2	294.2	252.2	210.2	168.2	126.2	84.2	42.2	0.2	-41.8
150	27.61	37.9	341.2	296.2	251.2	206.2	161.2	116.2	71.2	26.2	-18.8	-63.8
160	26.24	36.0	345.9	297.9	249.9	201.9	153.9	105.9	57.9	9.9	-38.1	-86.1
170	25.01	34.3	350.3	299.3	248.3	197.3	146.3	95.3	44.3	-6.7	-57.7	-108.7
180	23.90	32.8	354.5	300.5	246.5	192.5	138.5	84.5	30.5	-23.5	-77.5	-131.5
190	22.90	31.4	358.4	301.4	244.4	187.4	130.4	73.4	16.4	-40.6	-97.6	-154.6
Max. Storage Requirement =			358.4	301.4	253.2	224.5	204.3	188.4	175.6	164.1	155.1	146.1

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

APPENDIX A: STORMWATER MANAGEMENT MODEL

SHEET 4 - OUTLET CONTROL DESIGN SHEET - SWALE (POST-CA1)

Client: J.P.Bergeron
Job No.: 220863
Location: 5574 Rockdale Road, Vars
Date: December 21, 2023

Infiltration Information

Percolation Time T = 10 min/cm
Percolation Rate = 60 mm/hr
Permeability k = 1.67E-05 m/s
Depth of Layer = 1

Overflow Channel
Bottom Channel Width (m): 1.00
Channel Invert (m): 77.85

Orifice 1 Information	
Dia (m):	0.150
Area (mm.):	0.0177
Orifice Coeff, C:	0.60
Weir Coeff, C:	0.62
Orifice Top (m):	77.71
Orifice Cen (m):	77.64
Orifice Inv (m):	77.56

									Infiltration			Weir		Orifice 1 Flow						
Stage, WSE Elev (m)	Comments	Layer Thickness (m)	Top Layer Area (m²)	Bottom Layer Area (m²)	Volume in Swale (m^3)	Total Volume in Swale (m^3)	Quality Storage Volume (m3)	Quantity Storage (m3)	Head* (m)	Hydraulic Gradient	Infiltration Rate (m^3/sec)	Head (m)	Weir Flow (m^3/sec)	Depth of Flow Orifice (m)	n	Orifice Flow* (m^3/sec)	Total Outflow (m^3/sec)	Total Outflow (L/sec)	Discharge from Site (L/sec)	Draw Down Time (hrs)
77.86		0.020	1515.2	1470.2	29.9	416.1	0.0	348.7	0.58	1.6	0.0387	0.01	0.0029	0.3000	2.0000	0.0228	0.0644	64.4	25.7	0.0
77.84		0.020	1470.2	1425.2	29.0	386.3	0.0	318.8	0.56	1.6	0.0371	0.00	0.0000	0.2800	1.8667	0.0218	0.0588	58.8	21.8	0.0
77.82		0.020	1425.2	1380.4	28.1	357.3	0.0	289.9	0.54	1.5	0.0354	0.00	0.0000	0.2600	1.7333	0.0206	0.0561	56.1	20.6	0.1
77.80		0.020	1380.4	1335.7	27.2	329.3	0.0	261.8	0.52	1.5	0.0338	0.00	0.0000	0.2400	1.6000	0.0194	0.0533	53.3	19.4	0.1
77.78	Approx. 100-Year Storm	0.020	1335.7	1291.1	26.3	302.1	0.0	234.7	0.50	1.5	0.0323	0.00	0.0000	0.2200	1.4667	0.0181	0.0504	50.4	18.1	0.1
77.76		0.020	1291.1	1246.7	25.4	275.9	0.0	208.4	0.48	1.5	0.0308	0.00	0.0000	0.2000	1.3333	0.0167	0.0474	47.4	16.7	0.1
77.74	Approx. 5-Year Storm	0.020	1246.7	1202.4	24.5	250.5	0.0	183.0	0.46	1.5	0.0293	0.00	0.0000	0.1800	1.2000	0.0150	0.0443	44.3	15.0	0.2
77.72		0.020	1202.4	1158.2	23.6	226.0	0.0	158.5	0.44	1.4	0.0278	0.00	0.0000	0.1600	1.0667	0.0132	0.0410	41.0	13.2	0.2
77.70		0.020	1158.2	1114.1	22.7	202.4	0.0	134.9	0.42	1.4	0.0264	0.00	0.0000	0.1400	0.9333	0.0112	0.0376	37.6	11.2	0.2
77.68		0.020	1114.1	1070.2	21.8	179.7	0.0	112.2	0.40	1.4	0.0250	0.00	0.0000	0.1200	0.8000	0.0091	0.0340	34.0	9.1	0.2
77.66		0.020	1070.2	1026.4	21.0	157.8	0.0	90.3	0.38	1.4	0.0236	0.00	0.0000	0.1000	0.6667	0.0068	0.0304	30.4	6.8	0.2
77.64		0.020	1026.4	953.6	19.8	136.9	0.0	69.4	0.36	1.4	0.0216	0.00	0.0000	0.0800	0.5333	0.0046	0.0263	26.3	4.6	0.2
77.62		0.020	953.6	884.1	18.4	117.1	0.0	49.6	0.34	1.3	0.0197	0.00	0.0000	0.0600	0.4000	0.0027	0.0225	22.5	2.7	0.2
77.60		0.020	884.1	779.3	16.6	98.7	0.0	31.2	0.32	1.3	0.0171	0.00	0.0000	0.0400	0.2667	0.0012	0.0184	18.4	1.2	0.3
77.58		0.020	779.3	681.0	14.6	82.1	0.0	14.6	0.30	1.3	0.0148	0.00	0.0000	0.0200	0.1333	0.0003	0.0151	15.1	0.3	0.3
77.56	Quality Storm Event	0.020	681.0	589.1	12.7	67.5	67.5	0.0	0.28	1.3	0.0126	0.00	0.0000	0.0000	0.0000	0.0000	0.0126	12.6	0.0	0.3
77.54		0.020	589.1	503.7	10.9	54.8	54.8	0.0	0.26	1.3	0.0106	0.00	0.0000	-0.0200	0.0000	0.0000	0.0106	10.6	0.0	0.3
77.52		0.020	503.7	424.8	9.3	43.9	43.9	0.0	0.24	1.2	0.0088	0.00	0.0000	-0.0400	0.0000	0.0000	0.0088	8.8	0.0	0.3
77.50		0.020	424.8	352.3	7.8	34.6	34.6	0.0	0.22	1.2	0.0072	0.00	0.0000	-0.0600	0.0000	0.0000	0.0072	7.2	0.0	0.3
77.48		0.020	352.3	286.3	6.4	26.8	26.8	0.0	0.20	1.2	0.0057	0.00	0.0000	-0.0800	0.0000	0.0000	0.0057	5.7	0.0	0.3
77.46		0.020	286.3	226.8	5.1	20.5	20.5	0.0	0.18	1.2	0.0045	0.00	0.0000	-0.1000	0.0000	0.0000	0.0045	4.5	0.0	0.3
77.44		0.020	226.8	186.5	4.1	15.3	15.3	0.0	0.16	1.2	0.0036	0.00	0.0000	-0.1200	0.0000	0.0000	0.0036	3.6	0.0	0.3
77.42		0.020	186.5	150.3	3.4	11.2	11.2	0.0	0.14	1.1	0.0029	0.00	0.0000	-0.1400	0.0000	0.0000	0.0029	2.9	0.0	0.3
77.40		0.020	150.3	117.3	2.7	7.9	7.9	0.0	0.12	1.1	0.0022	0.00	0.0000	-0.1600	0.0000	0.0000	0.0022	2.2	0.0	0.3
77.38		0.020	117.3	87.5	2.0	5.2	5.2	0.0	0.10	1.1	0.0016	0.00	0.0000	-0.1800	0.0000	0.0000	0.0016	1.6	0.0	0.4
77.36		0.020	87.5	61.0	1.5	3.1	3.1	0.0	0.08	1.1	0.0011	0.00	0.0000	-0.2000	0.0000	0.0000	0.0011	1.1	0.0	0.4
77.34		0.020	61.0	37.7	1.0	1.7	1.7	0.0	0.06	1.1	0.0007	0.00	0.0000	-0.2200	0.0000	0.0000	0.0007	0.7	0.0	0.4
77.32		0.020	37.7	17.6	0.5	0.7	0.7	0.0	0.04	1.0	0.0003	0.00	0.0000	-0.2400	0.0000	0.0000	0.0003	0.3	0.0	0.5
77.30		0.020	17.6	0.8	0.1	0.1	0.1	0.0	0.02	1.0	0.0000	0.00	0.0000	-0.2600	0.0000	0.0000	0.0000	0.0	0.0	3.2
77.28		0.000	0.8	0.8	0.0	0.0	0.0	0.0	0.00	1.0	0.0000	0.00	0.0000	-0.2800	0.0000	0.0000	0.0000	0.0	0.0	0.0

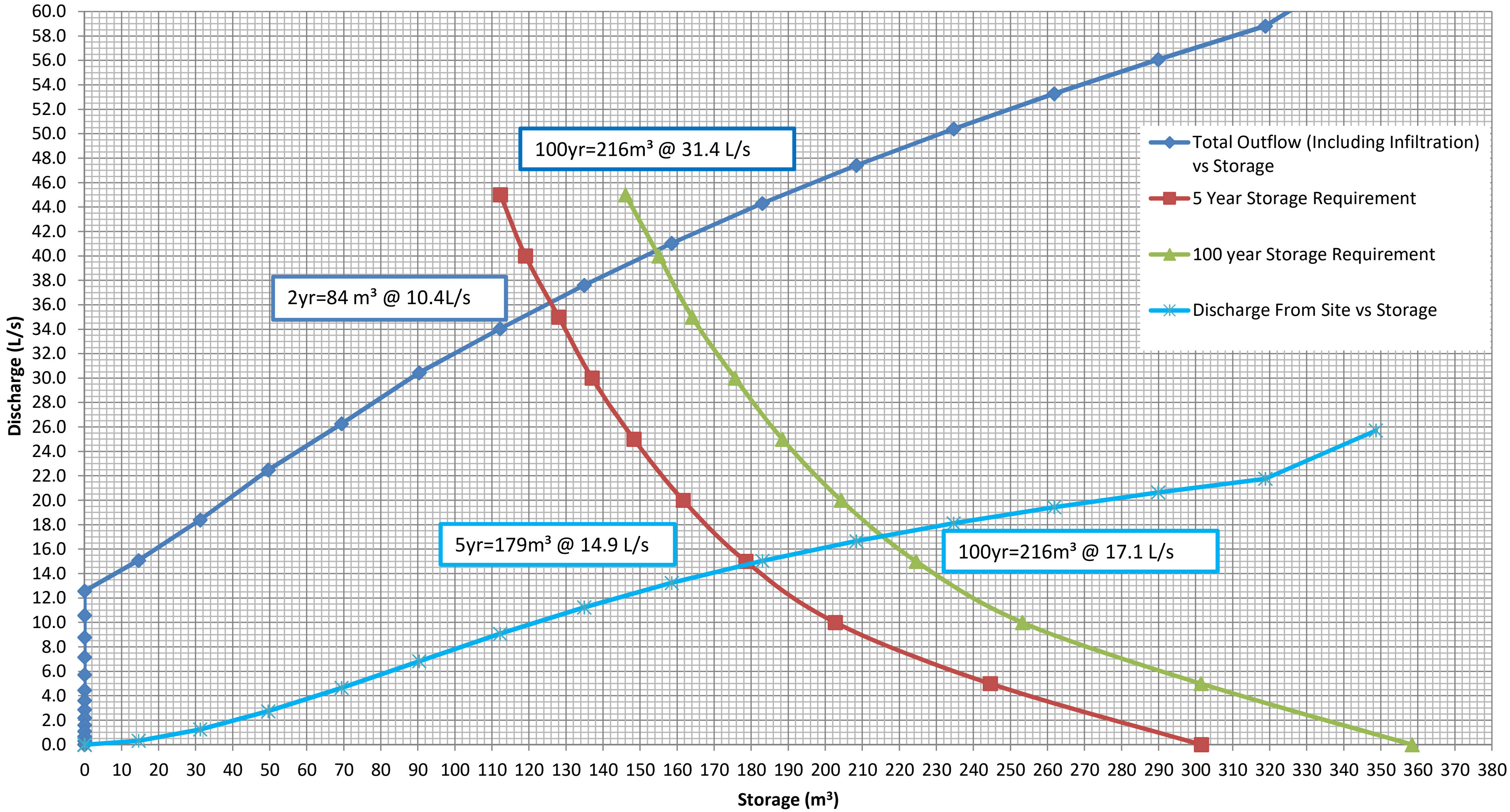
Note: Total Outflow from the storage swale includes both outlet by infiltration as well as outlet to the roadside ditch by means of the outlet structure.
Discharge from site includes only the portion of the flow exiting the swale by means of the outlet structure and being directed to the adjacent roadside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

Total Draw down time following 100-year storm (hours) 9.7
Draw down time below Filter (hours) 7.6

APPENDIX A: STORMWATER MANAGEMENT MODEL

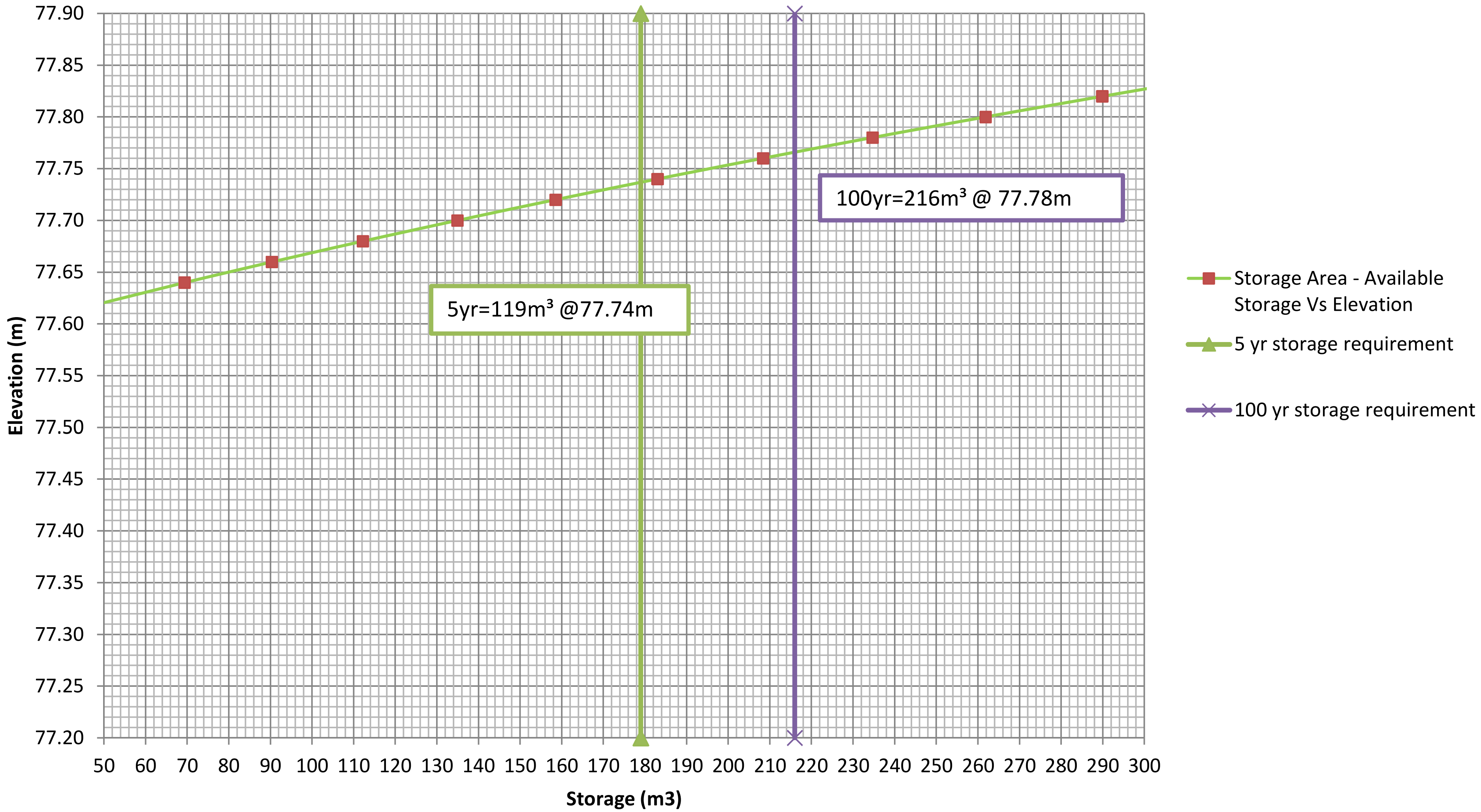
Figure 1 - Discharge Vs Storage Curve (POST-CA1)

Client: Bergeron Construction
Job No.: 220863
Location: 5574 Rockdale Road, City of Ottawa, ON
Date: December 21, 2023



Client: Bergeron Construction
Job No.: 220863
Location: 5574 Rockdale Road, City of Ottawa, ON
Date: December 21, 2023

APPENDIX A - STORMWATER MANAGEMENT MODEL
Figure 2 - Elevation Vs Storage Curve (POST-CA1)



APPENDIX A: STORMWATER MANAGEMENT MODEL
SHEET 5 - REQUIRED STORAGE VS. RELEASE RATE (POST-CA2)

Client: J.P.Bergeron
Job No.: 220863
Location: 5574 Rockdale Road, Vars
Date: December 21, 2023

Post Dev run-off Coefficient "C" - Post-C2

Area (ha)	Surface	Area (ha)	5 Year Event "C"	5 Year Event C_{avg}	100 Year Event "C" x 1.25	100 Year Event $C_{100\ avg}$
Total	Roof	0.000	0.90	0.32	1.00	0.40
	Asphalt	0.001	0.90		1.00	
0.850	Off-site Grassed Area	0.360	0.35		0.44	
	Grass/Swale	0.489	0.30		0.38	

Impervious Ratio = 0.00

REQUIRED STORAGE VERSUS RELEASE RATE FOR 2 YEAR STORM

Runoff Coeffcient, C = **0.32**
Drainage Area (ha) = **0.850**
Return Period (yrs) = **5**
Duration Interval (min) = **10**
Release Rate Start (L/s) = **0**
Release Rate Interval (L/s) = **5**

Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	0	5	10	15	20	25	30	35	40	45
		Release Rate -->	Storage Required (m^3)									
0	398.62	301.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	178.56	135.0	81.0	78.0	75.0	72.0	69.0	66.0	63.0	60.0	57.0	54.0
20	119.95	90.7	108.8	102.8	96.8	90.8	84.8	78.8	72.8	66.8	60.8	54.8
30	91.87	69.5	125.0	116.0	107.0	98.0	89.0	80.0	71.0	62.0	53.0	44.0
40	75.15	56.8	136.4	124.4	112.4	100.4	88.4	76.4	64.4	52.4	40.4	28.4
50	63.95	48.4	145.1	130.1	115.1	100.1	85.1	70.1	55.1	40.1	25.1	10.1
60	55.89	42.3	152.2	134.2	116.2	98.2	80.2	62.2	44.2	26.2	8.2	-9.8
70	49.79	37.6	158.1	137.1	116.1	95.1	74.1	53.1	32.1	11.1	-9.9	-30.9
80	44.99	34.0	163.3	139.3	115.3	91.3	67.3	43.3	19.3	-4.7	-28.7	-52.7
90	41.11	31.1	167.9	140.9	113.9	86.9	59.9	32.9	5.9	-21.1	-48.1	-75.1
100	37.90	28.7	172.0	142.0	112.0	82.0	52.0	22.0	-8.0	-38.0	-68.0	-98.0
110	35.20	26.6	175.7	142.7	109.7	76.7	43.7	10.7	-22.3	-55.3	-88.3	-121.3
120	32.89	24.9	179.1	143.1	107.1	71.1	35.1	-0.9	-36.9	-72.9	-108.9	-144.9
130	30.90	23.4	182.2	143.2	104.2	65.2	26.2	-12.8	-51.8	-90.8	-129.8	-168.8
140	29.15	22.0	185.2	143.2	101.2	59.2	17.2	-24.8	-66.8	-108.8	-150.8	-192.8
150	27.61	20.9	187.9	142.9	97.9	52.9	7.9	-37.1	-82.1	-127.1	-172.1	-217.1
160	26.24	19.8	190.5	142.5	94.5	46.5	-1.5	-49.5	-97.5	-145.5	-193.5	-241.5
170	25.01	18.9	192.9	141.9	90.9	39.9	-11.1	-62.1	-113.1	-164.1	-215.1	-266.1
180	23.90	18.1	195.2	141.2	87.2	33.2	-20.8	-74.8	-128.8	-182.8	-236.8	-290.8
190	22.90	17.3	197.4	140.4	83.4	26.4	-30.6	-87.6	-144.6	-201.6	-258.6	-315.6
Max. Storage Requirement =			197.4	143.2	116.2	100.4	89.0	80.0	72.8	66.8	60.8	54.8

REQUIRED STORAGE VERSUS RELEASE RATE FOR 5 YEAR STORM

Runoff Coeffcient, C = **0.40**
Drainage Area (ha) = **0.850**
Return Period (yrs) = **100**
Duration Interval (min) = **10**
Release Rate Start (L/s) = **0**
Release Rate Interval (L/s) = **5**

Duration (min)	Rainfall Intensity (mm/hr)	Peak Flow (L/sec)	0	5	10	15	20	25	30	35	40	45
		Release Rate -->	Storage Required (m^3)									
0	398.62	376.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	178.56	168.8	101.3	98.3	95.3	92.3	89.3	86.3	83.3	80.3	77.3	74.3
20	119.95	113.4	136.1	130.1	124.1	118.1	112.1	106.1	100.1	94.1	88.1	82.1
30	91.87	86.8	156.3	147.3	138.3	129.3	120.3	111.3	102.3	93.3	84.3	75.3
40	75.15	71.0	170.5	158.5	146.5	134.5	122.5	110.5	98.5	86.5	74.5	62.5
50	63.95	60.4	181.3	166.3	151.3	136.3	121.3	106.3	91.3	76.3	61.3	46.3
60	55.89	52.8	190.2	172.2	154.2	136.2	118.2	100.2	82.2	64.2	46.2	28.2
70	49.79	47.1	197.7	176.7	155.7	134.7	113.7	92.7	71.7	50.7	29.7	8.7
80	44.99	42.5	204.1	180.1	156.1	132.1	108.1	84.1	60.1	36.1	12.1	-11.9
90	41.11	38.9	209.8	182.8	155.8	128.8	101.8	74.8	47.8	20.8	-6.2	-33.2
100	37.90	35.8	215.0	185.0	155.0	125.0	95.0	65.0	35.0	5.0	-25.0	-55.0
110	35.20	33.3	219.6	186.6	153.6	120.6	87.6	54.6	21.6	-11.4	-44.4	-77.4
120	32.89	31.1	223.9	187.9	151.9	115.9	79.9	43.9	7.9	-28.1	-64.1	-100.1
130	30.90	29.2	227.8	188.8	149.8	110.8	71.8	32.8	-6.2	-45.2	-84.2	-123.2
140	29.15	27.6	231.5	189.5	147.5	105.5	63.5	21.5	-20.5	-62.5	-104.5	-146.5
150	27.61	26.1	234.9	189.9	144.9	99.9	54.9	9.9	-35.1	-80.1	-125.1	-170.1
160	26.24	24.8	238.1	190.1	142.1	94.1	46.1	-1.9	-49.9	-97.9	-145.9	-193.9
170	25.01	23.6	241.1	190.1	139.1	88.1	37.1	-13.9	-64.9	-115.9	-166.9	-217.9
180	23.90	22.6	244.0	190.0	136.0	82.0	28.0	-26.0	-80.0	-134.0	-188.0	-242.0
190	22.90	21.6	246.7	189.7	132.7	75.7	18.7	-38.3	-95.3	-152.3	-209.3	-266.3
Max. Storage Requirement =			246.7	190.1	156.1	136.3	122.5	111.3	102.3	94.1	88.1	82.1

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

APPENDIX A: STORMWATER MANAGEMENT MODEL

SHEET 6 - OUTLET CONTROL DESIGN SHEET - SWALE (POST-CA2)

Client: **J.P.Bergeron**
Job No.: **220863**
Location: **5574 Rockdale Road, Vars**
Date: **December 21, 2023**

Infiltration Information		**Filter Information**		Overflow Channel	
Percolation Time T =	10 min/cm	Percolation Time T =	10 min/cm		
Percolation Rate =	60 mm/hr	Percolation Rate =	60 mm/hr	Bottom Channel Width (m):	0.50
Permeability k =	1.7E-05 m/s	Permeability k =	1.7E-05	Channel Invert (m):	77.40
Depth of Layer =	1	Depth of Layer =	0.5		

								Infiltration			Filter Flow			Weir					
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)	Head* (m)	Hydraulic Gradient	Filter Flow (m^3/sec)	Head (m)	Weir Flow (m^3/sec)	Total Infiltration (m^3/sec)	Total Outflow (L/sec)	Discharge from Site (L/sec)	Draw Down Time (hrs)
77.55		0.025	830.0	830.0	16.8	0.0	245.6	0.55	1.6	0.0214	0.55	0.9	0.0000	0.15	0.0848	0.0214	21.4	84.8	0.2
77.53		0.025	830.0	714.0	16.8	0.0	245.6	0.53	1.5	0.0181	0.53	0.9	0.0000	0.13	0.0645	0.0181	18.1	64.5	0.3
77.50		0.025	714.0	697.6	16.8	0.0	245.6	0.50	1.5	0.0174	0.50	0.8	0.0000	0.10	0.0461	0.0174	17.4	46.1	0.3
77.48		0.025	697.6	681.4	16.8	0.0	245.6	0.48	1.5	0.0168	0.48	0.8	0.0000	0.08	0.0300	0.0168	16.8	30.0	0.3
77.45		0.025	681.4	665.3	16.8	0.0	245.6	0.45	1.5	0.0161	0.45	0.8	0.0000	0.05	0.0163	0.0161	16.1	16.3	0.3
77.43		0.025	665.3	649.4	16.4	0.0	228.8	0.43	1.4	0.0154	0.43	0.7	0.0000	0.03	0.0058	0.0154	15.4	5.8	0.3
77.40		0.025	649.4	633.8	16.0	0.0	212.4	0.40	1.4	0.0148	0.40	0.7	0.0000	0.00	0.0000	0.0148	14.8	0.0	0.3
77.38		0.025	633.8	618.2	15.6	0.0	196.3	0.38	1.4	0.0142	0.38	0.6	0.0000	0.00	0.0000	0.0142	14.2	0.0	0.3
77.35		0.025	618.2	602.9	15.3	0.0	180.7	0.35	1.4	0.0136	0.35	0.6	0.0000	0.00	0.0000	0.0136	13.6	0.0	0.3
77.33		0.025	602.9	587.8	14.9	0.0	165.4	0.33	1.3	0.0130	0.33	0.5	0.0000	0.00	0.0000	0.0130	13.0	0.0	0.3
77.30	Approx. 100-year	0.025	587.8	572.8	14.5	0.0	150.5	0.30	1.3	0.0124	0.30	0.5	0.0000	0.00	0.0000	0.0124	12.4	0.0	0.3
77.28		0.025	572.8	558.0	14.1	0.0	136.0	0.28	1.3	0.0119	0.28	0.5	0.0000	0.00	0.0000	0.0119	11.9	0.0	0.3
77.25	Approx. 5-year	0.025	558.0	543.4	13.8	0.0	121.9	0.25	1.3	0.0113	0.25	0.4	0.0000	0.00	0.0000	0.0113	11.3	0.0	0.3
77.23		0.025	543.4	529.0	13.4	0.0	108.1	0.23	1.2	0.0108	0.23	0.4	0.0000	0.00	0.0000	0.0108	10.8	0.0	0.3
77.20		0.025	529.0	514.7	13.0	0.0	94.7	0.20	1.2	0.0103	0.20	0.3	0.0000	0.00	0.0000	0.0103	10.3	0.0	0.4
77.18		0.025	514.7	500.6	12.7	0.0	81.7	0.18	1.2	0.0098	0.18	0.3	0.0000	0.00	0.0000	0.0098	9.8	0.0	0.4
77.15		0.025	500.6	486.8	12.3	0.0	69.0	0.15	1.2	0.0093	0.15	0.3	0.0000	0.00	0.0000	0.0093	9.3	0.0	0.4
77.13		0.025	486.8	473.0	12.0	0.0	56.6	0.13	1.1	0.0089	0.13	0.2	0.0000	0.00	0.0000	0.0089	8.9	0.0	0.4
77.10		0.025	473.0	459.5	11.7	0.0	44.6	0.10	1.1	0.0084	0.10	0.2	0.0000	0.00	0.0000	0.0084	8.4	0.0	0.4
77.08		0.025	459.5	446.2	11.3	0.0	33.0	0.08	1.1	0.0080	0.08	0.1	0.0000	0.00	0.0000	0.0080	8.0	0.0	0.4
77.05		0.025	446.2	433.0	11.0	0.0	21.7	0.05	1.1	0.0076	0.05	0.1	0.0000	0.00	0.0000	0.0076	7.6	0.0	0.4
77.03		0.025	433.0	420.0	10.7	0.0	10.7	0.03	1.0	0.0072	0.03	0.0	0.0000	0.00	0.0000	0.0072	7.2	0.0	0.4
77.00		0.000	420.0	250.0	0.0	0.0	0.0	0.00	1.0	0.0042	0.00	0.0	0.0000	0.00	0.0000	0.0042	4.2	0.0	0.0

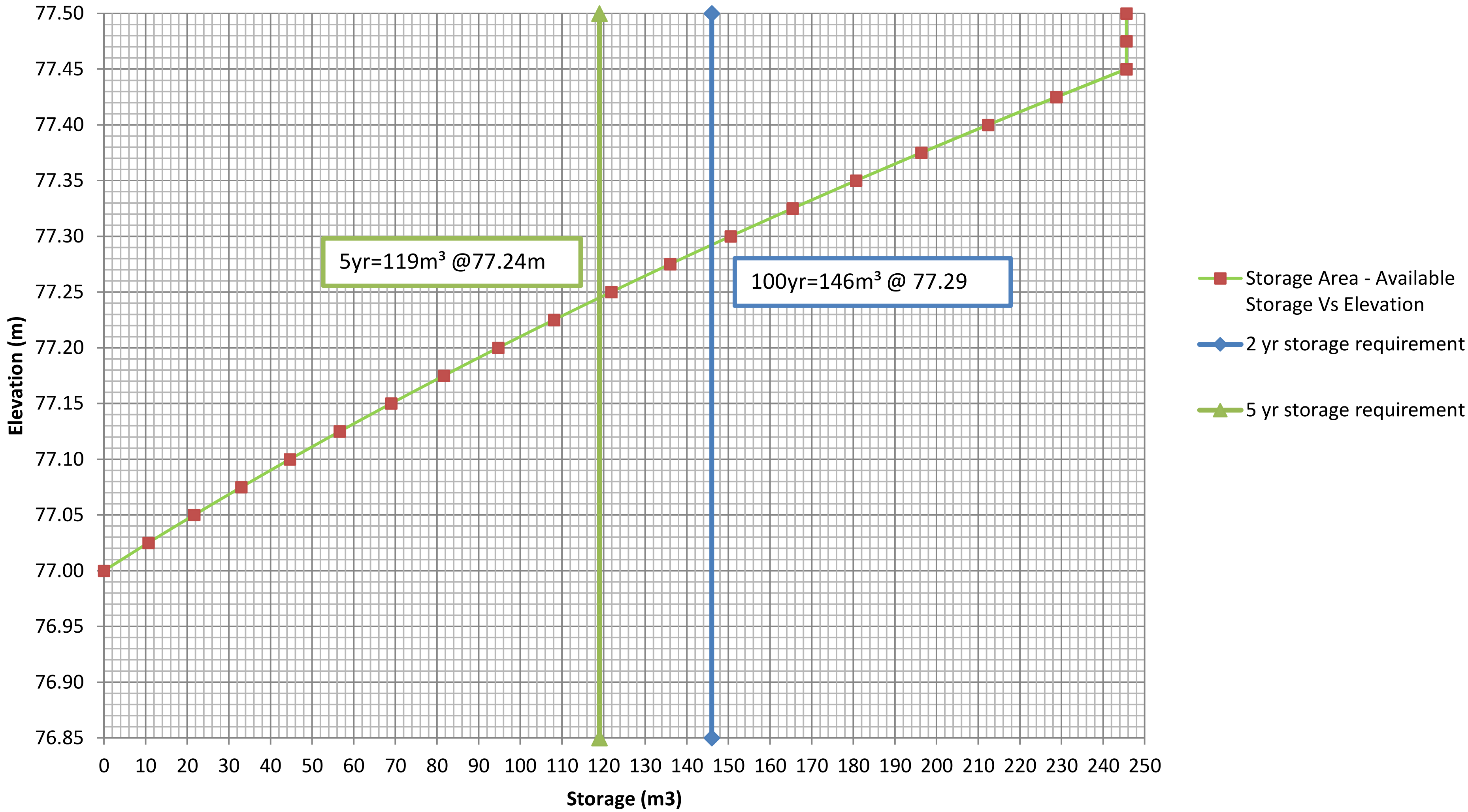
Note: Total Outflow from the bio-swale is by infiltration only.

Total Draw down time (hours) 4.4

Client: Bergeron Construction
Job No.: 220863
Location: 5574 Rockdale Road, City of Ottawa, ON
Date: December 21, 2023

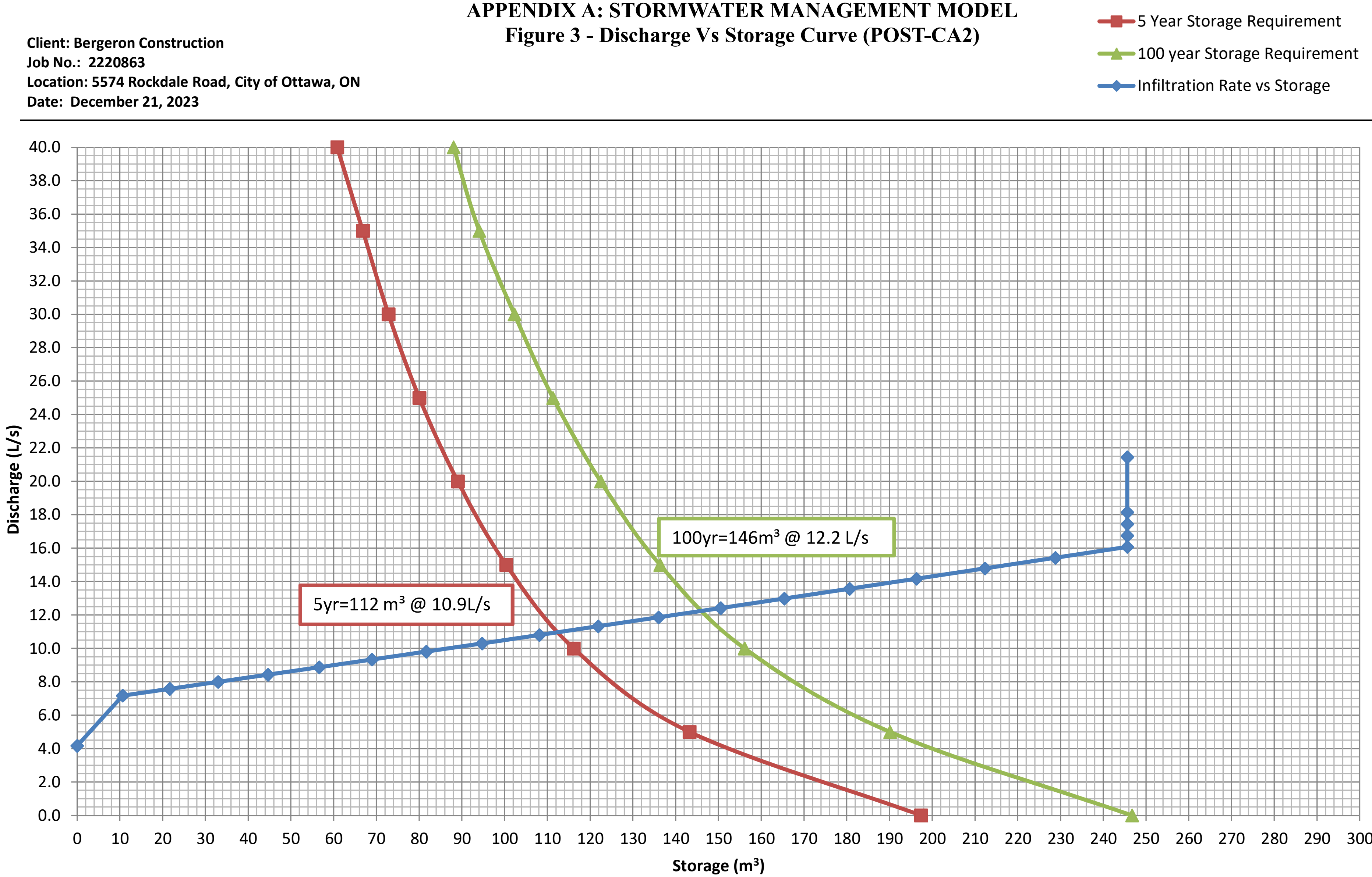
APPENDIX A - STORMWATER MANAGEMENT MODEL

Figure 4 - Elevation Vs Storage Curve (POST-CA2)



Client: Bergeron Construction
Job No.: 2220863
Location: 5574 Rockdale Road, City of Ottawa, ON
Date: December 21, 2023

APPENDIX A: STORMWATER MANAGEMENT MODEL
Figure 3 - Discharge Vs Storage Curve (POST-CA2)



Appendix B: Water Service

- Water Flow Analysis
- OBC Fire Flow Calculations

Water Flow Analysis

Pipe Sections	Along	End	Grade Elevation Start m	Grade Elevation End m	Hydraulic Grade line Start m	Hydraulic Grade line End m	Pressure P_{start} kPa	Pressure P_{end} kPa	Q m^3/sec	V m/sec	D m	Length m	A m^2
Senario 1 - Average Daily Flow - Duty Pump													
Rockdale (min HGL)	Main	Hydrant	78.4	78.5	108.40	108.40	294	293	0.000120	0.0038	0.2	120	0.031416
Hydrant	Service	Mechanical Room	78.5	80.1	108.40	108.39	293	277	0.000120	0.0611	0.05	35	0.001963
Senario 2 - Peak Hour - Duty Pump													
Rockdale (max HGL)	Main	Hydrant	78.4	78.5	108.40	108.40	294	293	0.000640	0.0204	0.2	120	0.031416
Hydrant	Service	Mechanical Room	78.5	80.1	108.40	108.39	293	277	0.000640	0.3259	0.05	35	0.001963
Senario 3 - Peak Hour - Max Day													
Rockdale (min HGL)	Main	Hydrant	78.4	78.5	119.30	119.30	401	400	0.000640	0.0204	0.2	120	0.031416
Hydrant	Service	Mechanical Room	78.5	80.1	119.30	119.29	400	384	0.000640	0.3259	0.05	35	0.001963
Senario 4 - Average Daily Flow - Duty Pump													
Rockdale (min HGL)	main	Hydrant	78.4	78.5	108.40	108.40	294	293	0.000120	0.0038	0.2	120	0.031416
Hydrant	Service	2nd storey	78.5	85.1	108.40	108.39	293	228	0.000120	0.0611	0.05	35	0.001963
Senario 5 - Peak Hour - Duty Pump													
Rockdale (max HGL)	Main	Hydrant	78.4	78.5	108.40	108.40	294	293	0.000640	0.0204	0.2	120	0.031416
Hydrant	Service	2nd storey	78.5	85.1	108.40	108.39	293	228	0.000640	0.3259	0.05	35	0.001963
Senario 6 - Peak Hour - Max Day													
Rockdale (min HGL)	Main	Hydrant	78.4	78.5	119.30	119.30	401	400	0.000640	0.0204	0.2	120	0.031416
Hydrant	Service	2nd storey	78.5	85.1	119.30	119.29	400	335	0.000640	0.3259	0.05	35	0.001963
Senario 7 - Fire Flow - 90 L/s gives 98.3 HGL - Actual Flow = 75 L/s assume 99 HGL													
Rockdale (min HGL)	Service	Hydrant	78.4	78.5	99.00	95.59	202	167	0.075120	2.3911	0.2	120	0.031416
Hydrant	Service	Mechanical Room	78.5	80.1	95.59	95.58	167	152	0.000120	0.0611	0.05	35	0.001963

Kollaard Associates
Engineers

FIRE FLOW REQUIREMENTS

Client:	**Bergeron Construction**
Job No.:	**220863**
Location:	**5574 Rockdale Road**
Date:	**23023-12-21**

Fire Water Storage and Supply Flow Rate Requirements

The following equation from the latest version of the Ontario Building Code (2012) was used for calculation of the on-site supply rates required to be supplied by the hydrants.

Formulae:

$$Q = KVS_{Tot}$$
$$S_{Tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + S_{side4} + ...]$$

OBC Classification of Building Use	Group, Division	2-Storey Wood Frame Residential "C"		
Assumed Type of Construction	Combustible	Building is of Combustible construction		
Water Supply Coefficient (Table 1, OBC)	K	23		
Exposure Distance 1	north	>10	m	
Exposure Distance 2	south	>10	m	
Exposure Distance 3	east	>10	m	
Exposure Distance 4	west	>10	m	
Spatial Coefficient 1	Sside	0		
Spatial Coefficient 2	Sside	0		
Spatial Coefficient 3	Sside	0		
Spatial Coefficient 4	Sside	0		
Total Spatial Coefficient	Stot	1		
Average Building Height	H	10.1	m	
Building Footprint	A	600	sq.m	
Total Building Volume	V	6,055	cu.m	
Minimum Supply of Water	Q	139,264	L	
Required Fire Flow	Qf	4500	L/min	*per Table 2 on A-3.2.5.7 of the OBC*
		75	L/s	
		1189	US gpm	

OBC - Table 2 of A-3.2.5.7.
REQUIRE MINIMUM WATER SUPPLY FLOW RATE (L/min)

Qf =	2700	If Q ≤ 108 000 L
Qf =	3600	108 000L < Q ≤ 135 000 L
Qf =	4500	135 000L < Q ≤ 162 000 L
Qf =	5400	162 000L < Q ≤ 190 000 L
Qf =	6300	190 000L < Q ≤ 270 000 L
Qf =	9000	Q > 270 000 L

Appendix C: Sewage System Design

Not included with this submission. An updated permit is required and must be obtained by the owner/developer.

Appendix D: Correspondence

- City of Ottawa – Boundary Conditions (Inserted from BLEL report)

Michael Jans

From:	Alvey, Harry [Harry.Alvey@ottawa.ca]
Sent:	August-07-13 1:07 PM
To:	'Michael Jans'
Cc:	Fitzpatrick, Anne
Subject:	RE: 5574 Rockdale, vars

Good Afternoon Michael;

Here are the water boundary conditions as you requested;

The boundary conditions depend strongly on pump selection. Ignoring fires, minimum pressure actually occurs during basic (average) demand conditions when the duty pump is running. During peak hour or fire conditions, the duty pump does not operate. Larger capacity pumps with higher discharge pressures operate during these conditions.

Boundary conditions at the site are as follows:

Basic Day average= 115.4 m
Minimum pressure during Basic Day = 108.4m
Peak Hour on Max Day = 119.3 m

The system is not designed to supply the required fire demand. The development will need to consider the fire supply limitation, adjust building design accordingly, and/or provide additional on-site fire fighting measures. Below I have provided two boundary conditions based on fire flows that would result in the range of roughly 20 psi and above at the property.

FF = 95 L/s, Max Day + Fire = 93.6 m (~21 psi)
FF = 90 L/s, Max Day + Fire = 98.3 m (~28 psi)

For the record, a 3 hour fire flow of 95 L/s at max day would drop the pump station clearwells to 30% full, assuming a starting point of 75%.

If you have any questions or need any additional information let me know.

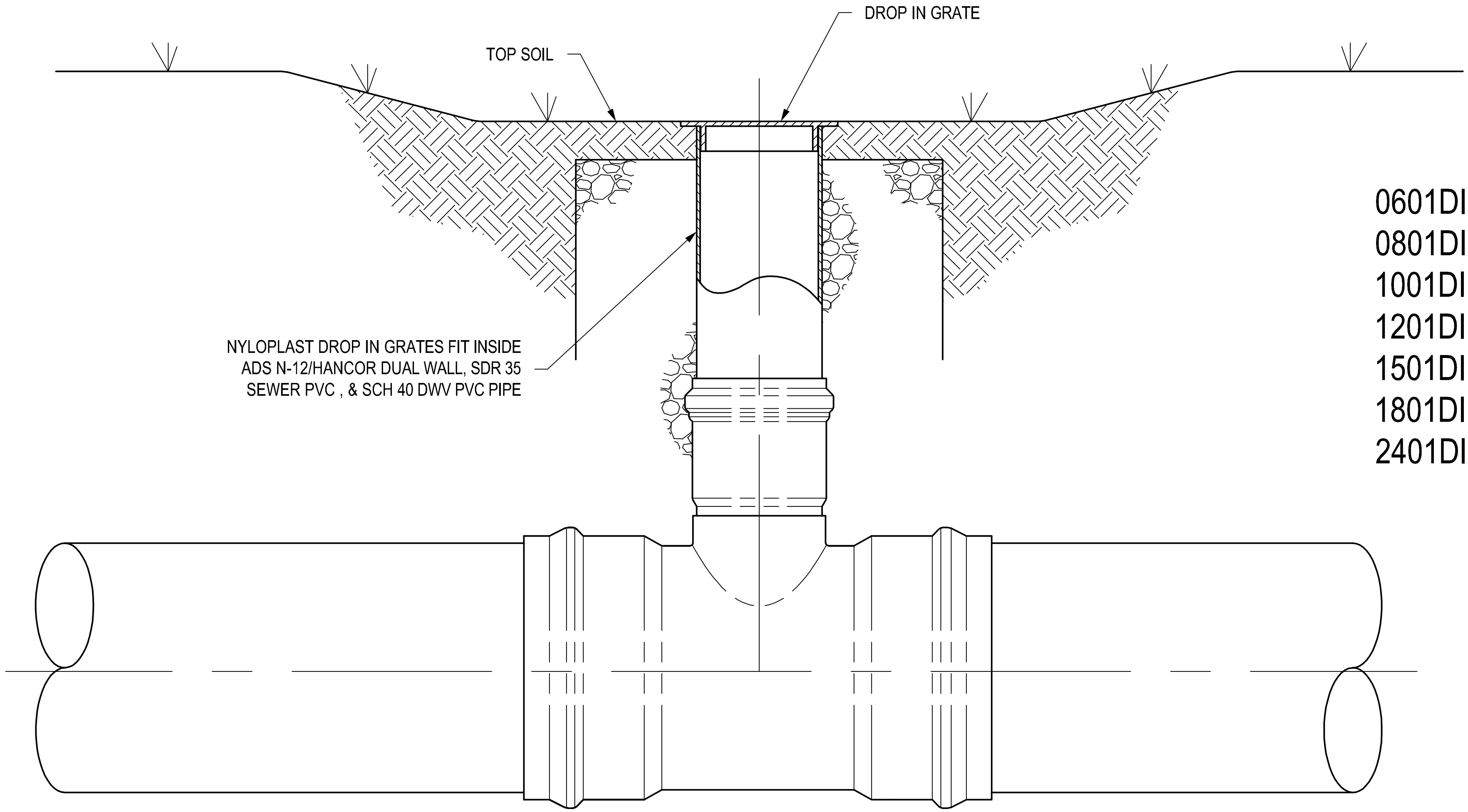
Sincerely;

Harry

Harry R. Alvey
Senior Infrastructure Approval Engineer
Development Review Rural Services

Appendix E: Product Information

- Nyloplast Grates
- Nyloplast Tees



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DRAWN BY AWA	MATERIAL		
DATE 5-4-00			
REVISED BY EBC	PROJECT NO./NAME	TITLE	
DATE 2-9-10		6 IN - 24 IN DROP IN GRATE INSTALLATION	
DWG SIZE A	SCALE 1:8 SHEET 1 OF 1	DWG NO. 7001-110-017	REV E

ADS Nyloplast
3130 VERONA AVE
BUFORD, GA 30518
PHN (770) 932-2443
FAX (770) 932-2490
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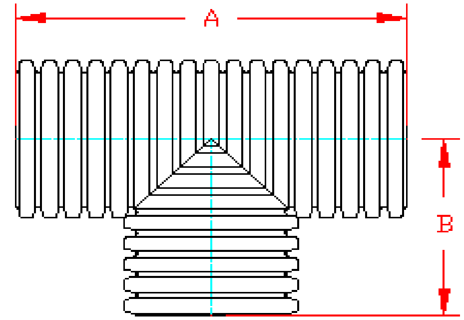
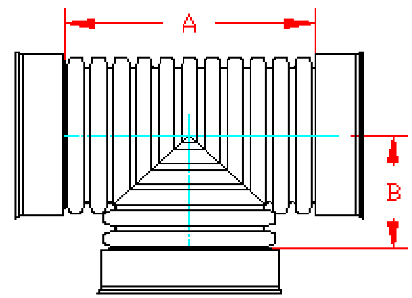
DUAL WALL FABRICATED TEES
4" - 30" DIAMETER

	PART #	PIPE SIZE	A	B	JOINT
**	0460AN	4 in (100 mm)	12.8 in (324 mm)	6.2 in (158 mm)	*
**	0661AN	6 in (150 mm)	16.5 in (419 mm)	8.1 in (207 mm)	*
**	0862AN	8 in (200 mm)	21.0 in (533 mm)	10.6 in (268 mm)	*
**	1063AN	10 in (250 mm)	26.0 in (660 mm)	12.6 in (320 mm)	*
	1264AN	12 in (300 mm)	30.7 in (780 mm)	15.4 in (390 mm)	*
	1264AN65B	12 in (300 mm)	21.1 in (536 mm)	10.6 in (268 mm)	WT
	1565AN	15 in (375 mm)	38.9 in (987 mm)	19.4 in (494 mm)	*
	1565AN65B	15 in (375 mm)	23.3 in (592 mm)	11.7 in (296 mm)	WT
	1866AN	18 in (450 mm)	42.9 in (1089 mm)	21.4 in (545 mm)	*
	1866AN65B	18 in (450 mm)	26.8 in (681 mm)	13.4 in (340 mm)	WT
	2467AN	24 in (600 mm)	50.4 in (1280 mm)	25.2 in (640 mm)	*
	2467AN65B	24 in (600 mm)	37.8 in (960 mm)	18.9 in (480 mm)	WT



PLAIN END



WT (INCLUDES 3 GASKETS)

* = PLAIN END
ST = SOIL TIGHT
WT = WATER TIGHT
** LIMITED AVAILABILITY. PLEASE SEE INJECTION MOLDED FITTING SECTION FOR OTHER AVAILABLE FITTINGS

NOTE: ALL FITTINGS DIMENSIONS ARE FOR REFERENCE ONLY

© 2023 Advanced Drainage Systems, Inc.

Advanced Drainage Systems, Inc.

DRAWING #:	2400	
DRAWN BY:	JCB	05.23.07
APPROVED BY:	JCB	06.26.07
REVISIONS:	TJR	4/28/2016

LIST OF DRAWINGS

PRE–DEVELOPMENT

SCALE = 1:300

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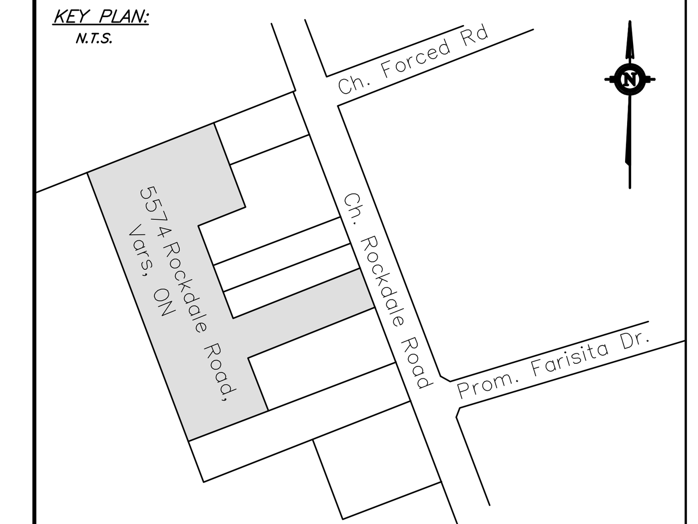
DRAWING NUMBER: 220863–PRE

KEY PLAN: N.T.S.



SCALE: 0 1 2 3 4 5 10 15 20

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12. REFERENCE TO KOLLAARD FILE #220863 FOR SITE SERVICING AND STORMWATER MANAGEMENT DESIGN.

No.	REVISION	DATE	BY
1.	ISSUED IN RESPONSE TO REVIEW COMMENTS #9	DEC 21, 2023	NJR

Kollaard Associates
Engineers
(613) 860–0923
info@kollaard.ca
P.O. BOX 189, 210 PRESCOTT ST.
KEMPTVILLE, ONTARIO
K0G 1J0 FAX (613) 258–0475
http://www.kollaard.ca

CLIENT:
MR. JP BERGERON
880 SMITH ROAD
NAVAN, ON, K4B 1N9

PROJECT:
PROPOSED 12 UNIT APARTMENT BUILDING

LOCATION:
5574 ROCKDALE ROAD, VARS ON, K0A 3H0

LICENSED PROFESSIONAL ENGINEER
21.DEC.2023
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGNED BY: NJR	CHECKED BY: SD
DRAWN BY: JR	APPROVED BY: SD
DATE: DEC 21, 2023	
KOLLAARD FILE NUMBER: 220863	

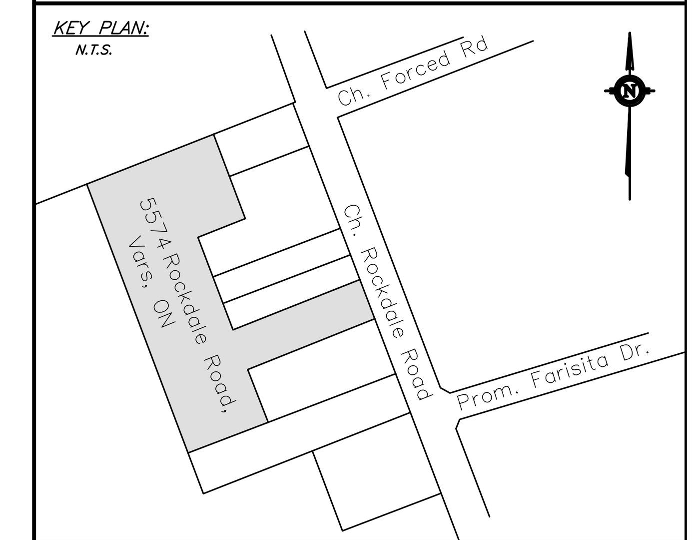
DRAWING NUMBER: 220863–PRE

DRAWING NAME: PRE–DEVELOPMENT CONDITIONS

DRAWING NUMBER:

220863–POST

KEY PLAN:
N.T.S.



SCALE:

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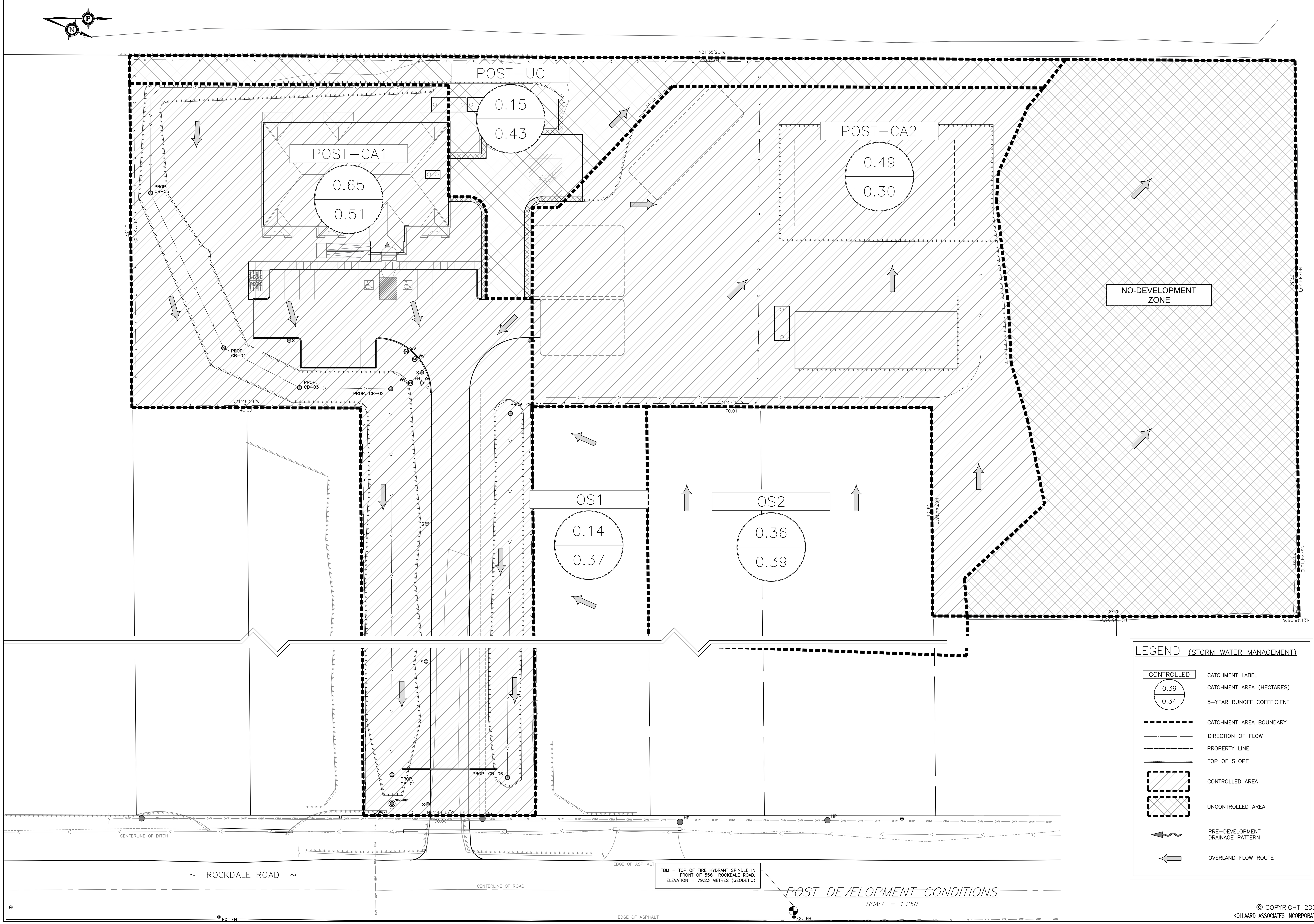
DESIGNED BY: NJR	CHECKED BY: SD
DRAWN BY: JR	APPROVED BY: SD
DATE: DEC 21, 2023	
KOLLAARD FILE NUMBER: 220863	

DRAWING NUMBER:

220863–POST

DRAWING NAME:

POST–DEVELOPMENT CONDITIONS



LEGEND (STORM WATER MANAGEMENT)

CONTROLLED	CATCHMENT LABEL
0.39	CATCHMENT AREA (HECTARES)
0.34	5–YEAR RUNOFF COEFFICIENT
	CATCHMENT AREA BOUNDARY
	DIRECTION OF FLOW
	PROPERTY LINE
	TOP OF SLOPE
	CONTROLLED AREA
	UNCONTROLLED AREA
	PRE–DEVELOPMENT DRAINAGE PATTERN
	OVERLAND FLOW ROUTE

POST DEVELOPMENT CONDITIONS
SCALE = 1:250

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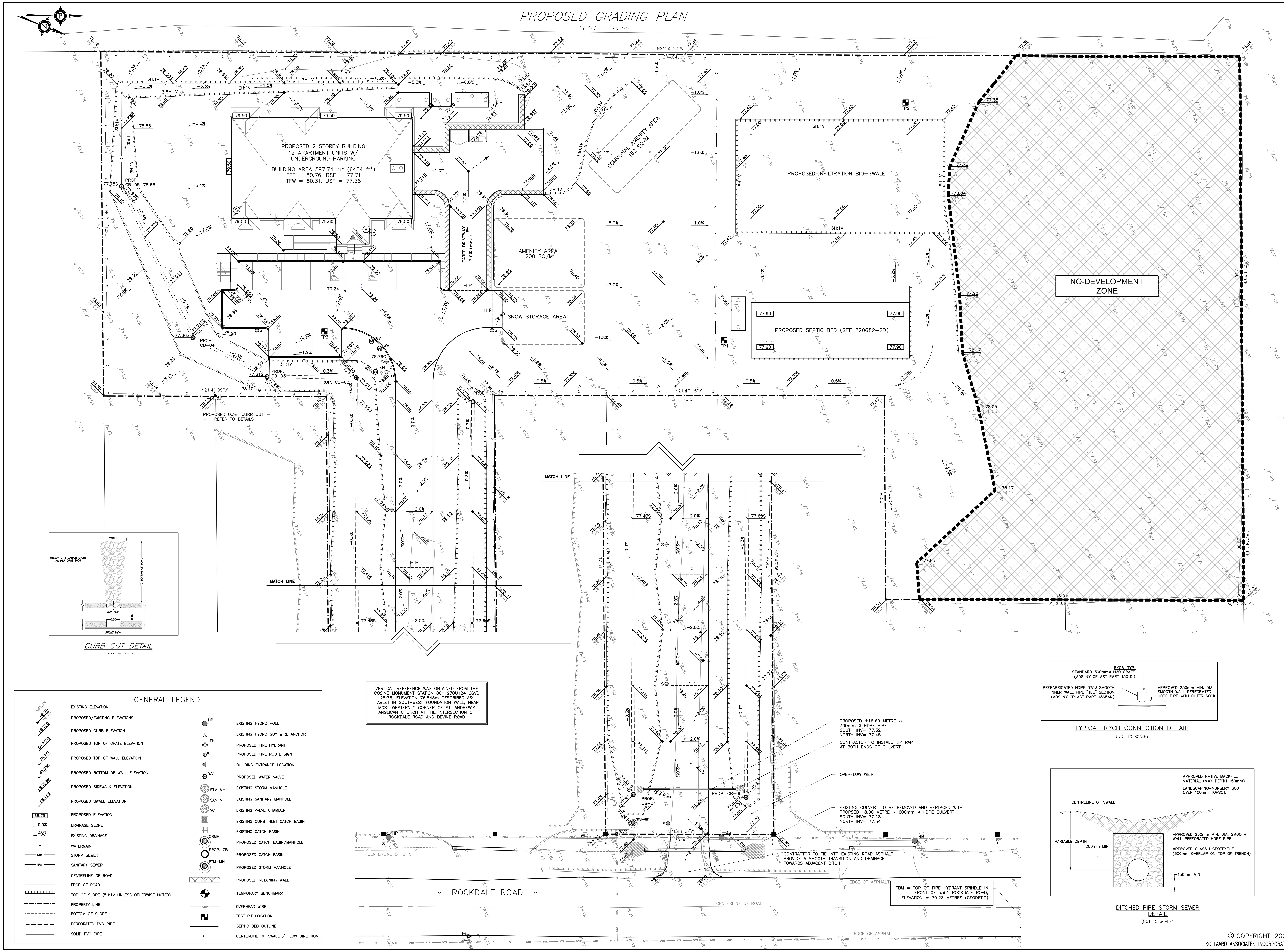

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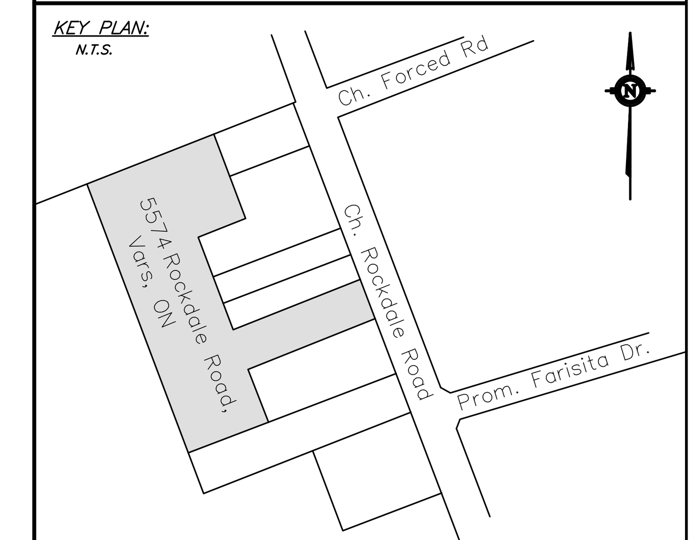
DRAWING NUMBER: 220863-GR

KEY PLAN:
N.T.S.



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PROVINCE OF ONTARIO

DESIGNED BY:	CHECKED BY:
NJR	SD
DRAWN BY:	APPROVED BY:
JR	SD

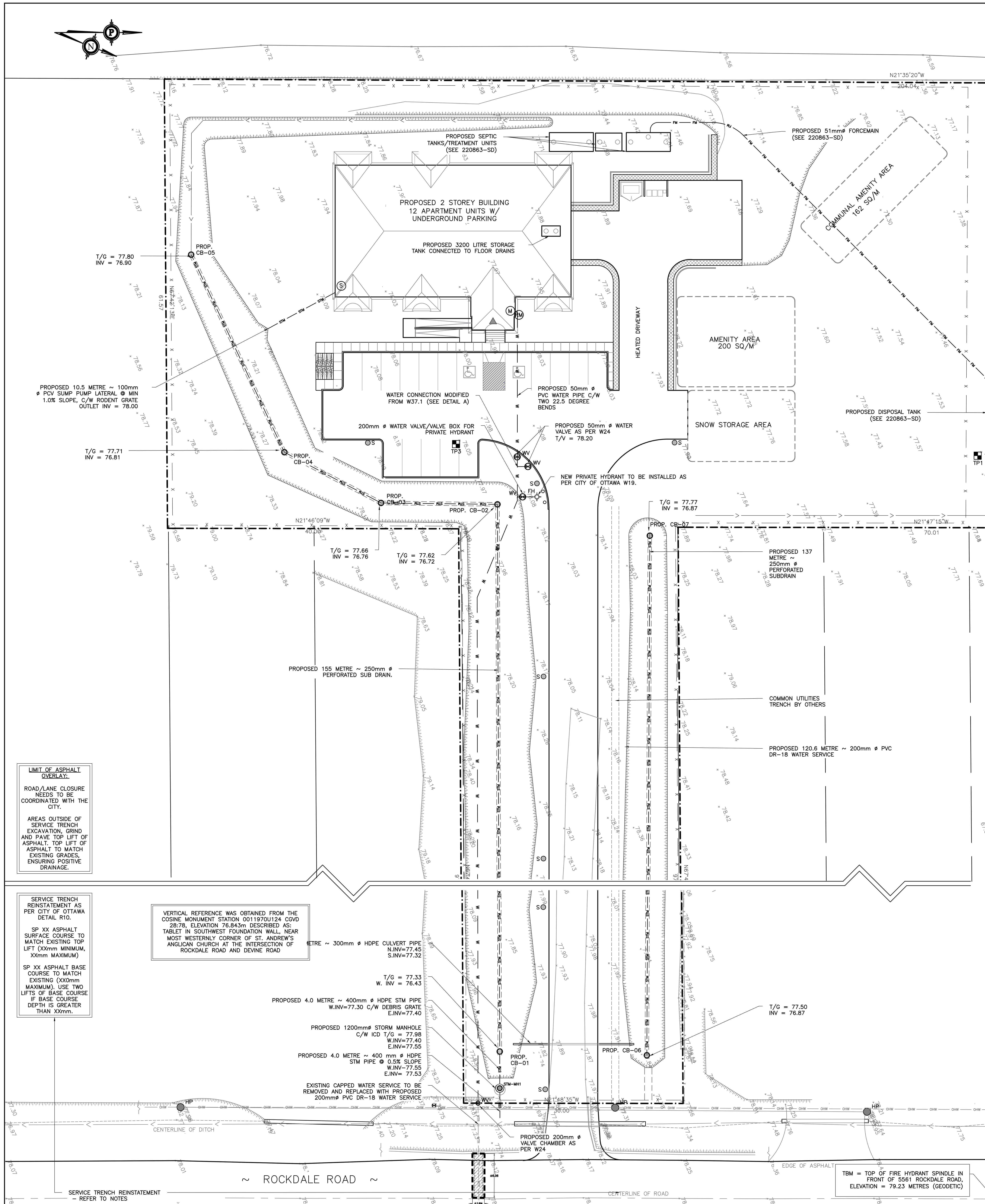
DATE: DEC 21, 2023

KOLLAARD FILE NUMBER: 220863

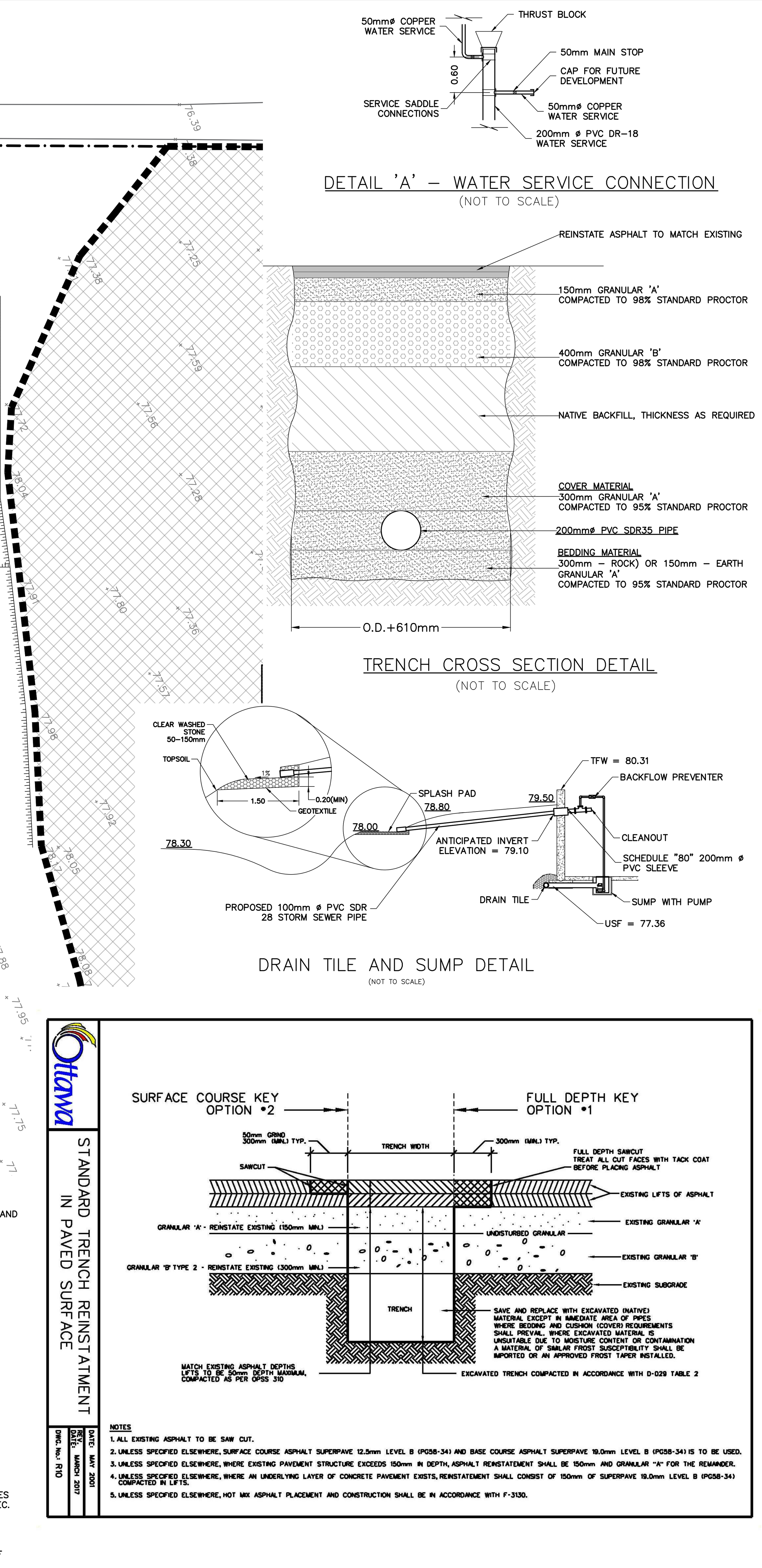
DRAWING NUMBER: 220863-GR

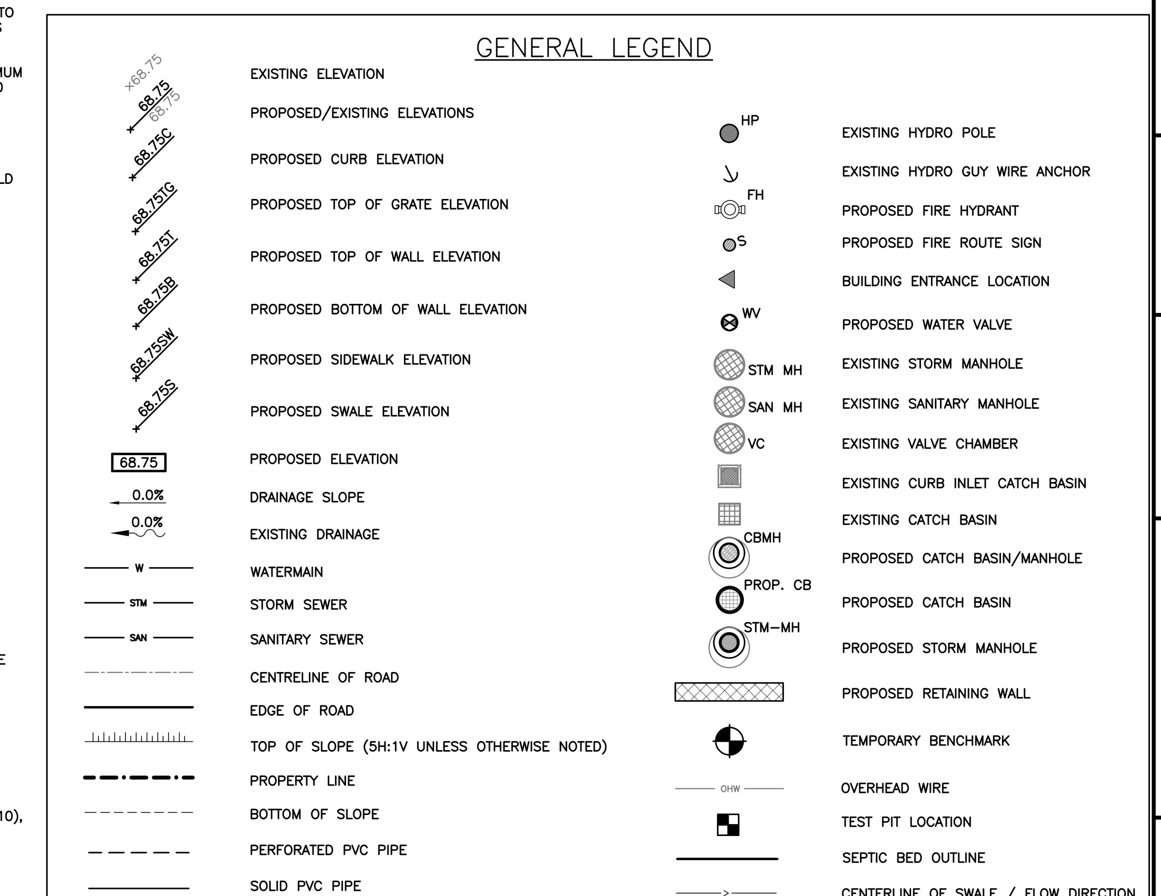
DRAWING NAME:
PROPOSED GRADING PLAN

ARCH D



PROPOSED SERVICING PLAN
SCALE = 1:300





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WATERMAIN NOTES:

1. CITY TO SUPPLY, INSTALL & DISINFECT THE WATER SERVICE; CONTRACTOR TO EXCAVATE, BACKFILL AND REINSTATE THE ROADWAY AS PER STD DWG R10.
2. SPECIFICATIONS:

ITEM	SPEC. No.	City Std Dwg No.
WATERMAIN BEDDING AND BACKFILL OPSD	802.010/802.031	W17 (trench detail)
CATHODIC PROTECTION OPSD	1109.010	W40
PRESSURE TESTING AWWA	C-605-5	
CHLORINATION AWWA	C-651-05	
WATERMAIN MATERIAL	PVC DR18	(CLASS 150)
WATER SERVICE MATERIAL	TYPE K COPPER	

3. WATERMAIN SHALL BE MINIMUM 2.4m DEPTH BELOW GRADE UNLESS OTHERWISE INDICATED. WHERE LESS THAN 2.4m COVER, THERMAL INSULATION IS TO BE PROVIDED AS PER CITY STD DWG W22 (In shallow trenches), W23 (At open structures).
4. A MINIMUM OF 0.5m VERTICAL CLEARANCE IS REQUIRED BETWEEN THE WATERMAINS AND ALL UTILITIES AND SEWERS. IN LOCATIONS WHERE THIS IS NOT ACHIEVABLE, MUST FOLLOW PROCEDURE F-6-1 SEC. 5.2 OF THE ONTARIO DRINKING WATER RESOURCES ACT.
5. METALLIC WARNING TAPE SHALL BE USED OVER ALL WATERMAINS.
6. INSTALL AND TEST TRACER WIRE FOR ALL PROPOSED WATERMAIN IN ACCORDANCE WITH THE CITY OF OTTAWA DESIGN STANDARDS AS SPECIFIED IN SECTION 8.28.
7. EXISTING WATERMAIN INFORMATION SHOWN IS BASED ON BEST CURRENT INFORMATION. CONTRACTOR TO VERIFY EXACT LOCATION OF WATERMAIN AND REPORT ANY DISCREPANCIES TO KOLLAARD ASSOCIATES INC.
8. WATER SHUTOFF VALVE AND VALVE BOX TO BE WITHIN THE ROAD ALLOWANCE AND LOCATED A MINIMUM OF 1.0 METRES FROM THE BUILDING FOUNDATION. TYPICAL PRIVATE SERVICE AS PER STD. DWG. W50 (with the exception that the V&VB are to be located 1.0 m minimum from the foundation wall); VALVE BOX ASSEMBLY AS PER STD. DWG. W24.
9. CONNECTIONS AT ELBOWS AND TEEs IN WATER MAINS SHOULD BE MADE WITH THE USE OF JOINT RESTRAINERS DESIGNED FOR WATERMAIN APPLICATION. JOINT AND PIPE RESTRAINERS SHOULD MEET THE REQUIREMENTS OF AWWA C900, C905 AND C907 AND ASTM F1674-11. JOINT RETRAINERS SHOULD BE INSTALLED AS PER MANUFACTURERS RECOMMENDATIONS.
10. ALL CONNECTORS, RODS AND VALVE BOLTS SHALL BE STAINLESS STEEL.
11. VALVES ARE TO BE OPERATED BY CITY OF OTTAWA STAFF ONLY.
12. NO CONNECTION TO EXISTING WATER NETWORK SHALL BE COMPLETED UNTIL A WATER PERMIT IS OBTAINED FROM THE CITY OF OTTAWA AND CITY OF OTTAWA FORCES ARE ON HAND TO MAKE THE CONNECTION.

SEWER NOTES:

1. SUPPLY AND CONSTRUCT ALL SEWERS AND APPURTENANCES IN ACCORDANCE WITH THE CITY OF OTTAWA STANDARDS AND SPECIFICATIONS AND ONTARIO PROVINCIAL STANDARDS FOR ROADS AND PUBLIC WORKS.
2. SPECIFICATIONS:

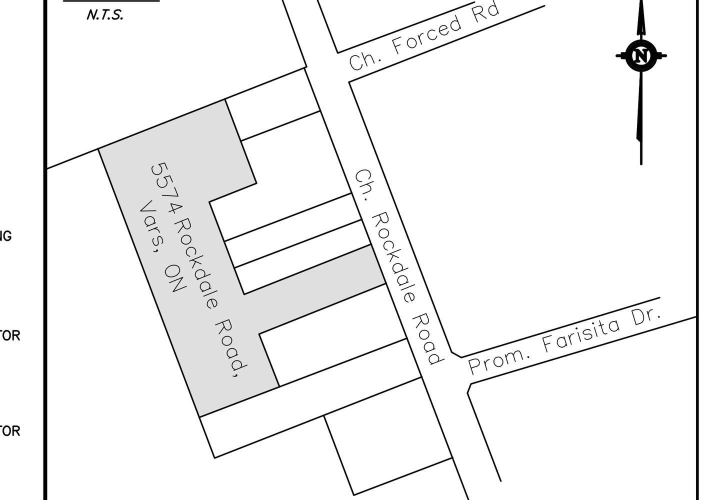
ITEM	PART. No.	CITY STD DWG No.
SUBDRAIN CATCH BASINS	ADS NYLOPLAST 1501DI	S30 & S31
SUBDRAIN CATCH BASINS	ADS NYLOPLAST 1565AN	S30 & S31

SEWER TRENCH:
SITE SERVICES EXCAVATION, BEDDING & BACKFILL AS PER THE RECOMMENDATIONS OF THE GEOTECHNICAL INVESTIGATION PREPARE BY OTHERS

3. INSULATE ALL STORM PIPES THAT HAVE LESS THAN 1.5m COVER WITH THERMAL INSULATION. PROVIDE 150mm CLEARANCE BETWEEN PIPE AND INSULATION.
4. PIPE BEDDING, COVER AND BACKFILL ARE TO BE COMPACTED TO AT LEAST 98% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY.
5. FLEXIBLE CONNECTIONS ARE REQUIRED FOR CONNECTION PIPES TO MANHOLES (FOR EXAMPLE KOR-N-SEAL, PSX: POSITIVE SEAL AND DURASEAL). SANITARY RUBBER GASKET TYPE JOINTS SHALL CONFORM TO CSA (B-182.2,3,4).
6. STORM MANHOLES AND CBMHS ARE TO HAVE 300mm SUMPS (AS PER SUMP DETAIL ON OPSD 701.010), UNLESS OTHERWISE INDICATED.
7. BUILDING CONTRACTOR TO PROVIDE TEMPORARY ADDITIONAL GRANULAR BACKFILL ABOVE SHALLOW CULVERTS AND STORM SEWERS TO SUPPORT HEAVY CONSTRUCTION EQUIPMENT.
8. CONTRACTOR TO TELEVISE (CCTV) ALL PROPOSED SEWERS, 200mmø OR GREATER PRIOR TO BASE COURSE ASPHALT. UPON COMPLETION OF CONTRACT, THE CONTRACTOR IS RESPONSIBLE TO FLUSH AND CLEAN ALL SEWERS & APPURTENANCES TO MUNICIPAL SATISFACTION.
9. CULVERT INSTALLATION AS PER OPSD 803.030

DRAWING NUMBER: 220863-SER

KEY PLAN:
N.T.S.



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DRAWN BY: JR	APPROVED BY: SD

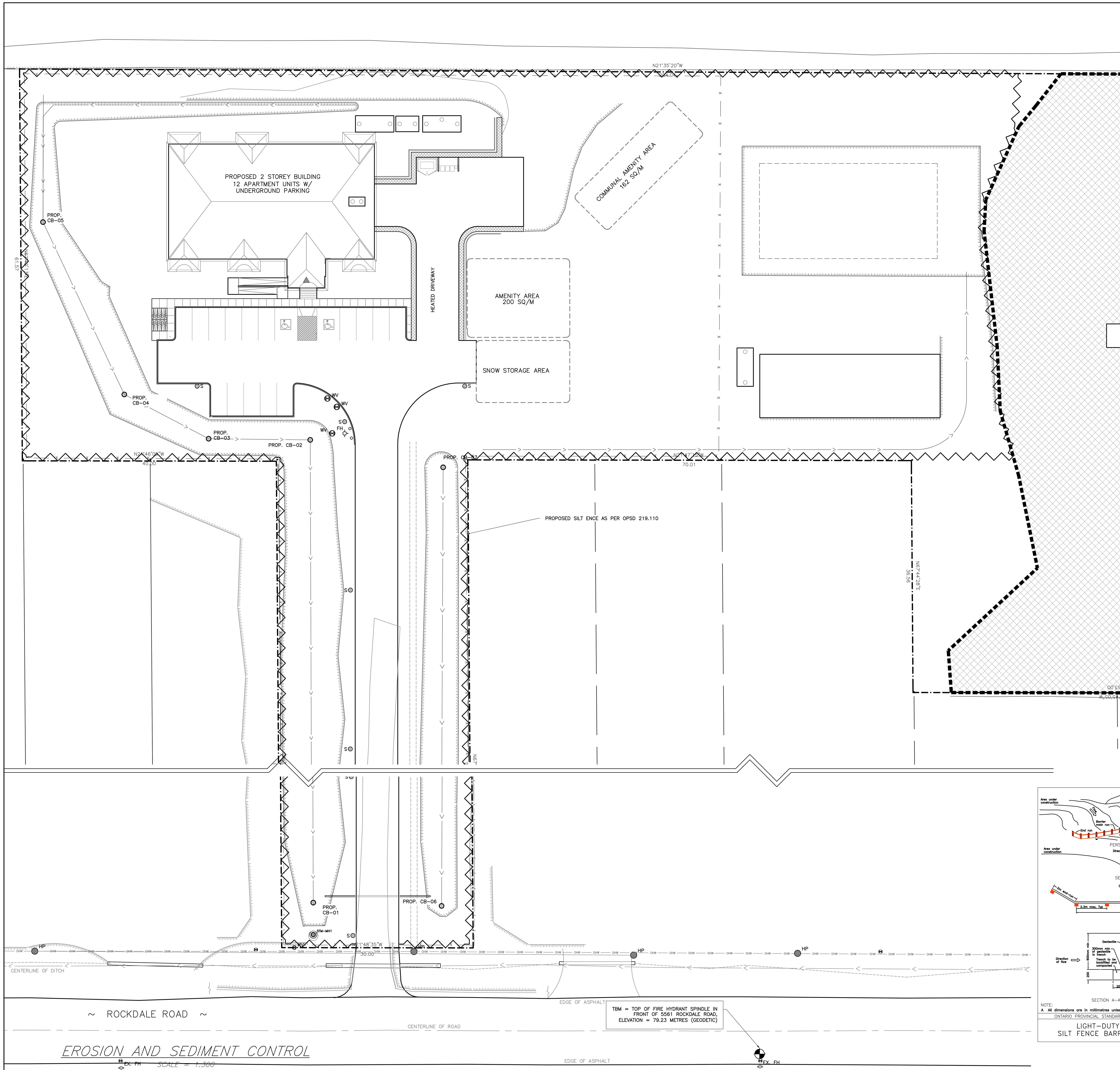
DATE: DEC 21, 2023

KOLLAARD FILE NUMBER: 220863

DRAWING NUMBER: 220863-SER

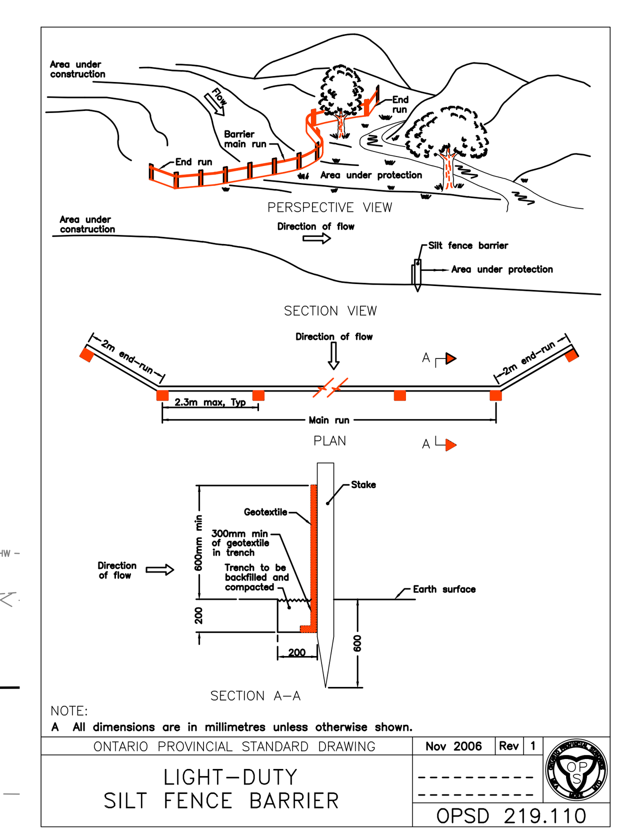
DRAWING NAME: PROPOSED SERVICING PLAN

ARCH D



EROSION AND SEDIMENT CONTROL

SCALE = 1:300



LEGEND

- SILT FENCE BARRIER
- STRAW BALE FLOW CHECK DAM
- TOP OF SLOPE
- PROPERTY LINE
- DITCH CENTERLINE/DIRECTION OF FLOW
- EDGE OF ROAD

EROSION AND SEDIMENT CONTROL NOTES:

1. THE CONTRACTOR SHALL IMPLEMENT BEST MANAGEMENT PRACTICES, TO PROVIDE FOR PROTECTION OF THE AREA DRAINAGE SYSTEM AND THE RECEIVING WATERCOURSE, 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 CITY OF OTTAWA, 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.

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.
- INSTALL FILTER CLOTH ACROSS ALL EXISTING AND PROPOSED CATCH BASINS AND CATCH BASIN MANHOLES PRIOR TO CONSTRUCTION.

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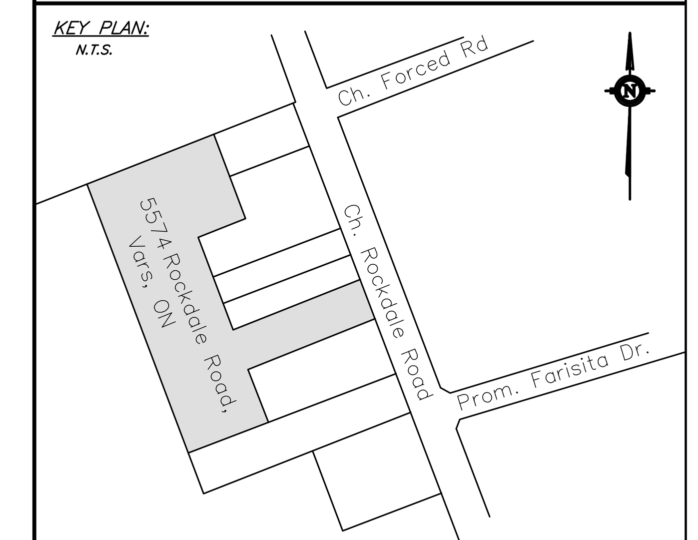
DRAWING NUMBER: 220863-ESC

KEY PLAN: N.T.S.



SCALE: 0 1 2 3 4 5 10 15 20

GENERAL NOTES:

1. ALL DIMENSIONS AND ELEVATIONS ARE IN METRES. THIS DRAWING IS NOT FOR CONSTRUCTION UNTIL APPROVED BY THE RELEVANT AUTHORITIES.
2. THIS IS NOT A LEGAL SURVEY. SITE BOUNDARIES AND TOPOGRAPHIC INFORMATION WAS PROVIDED BY ARPENTAGE DUTRISAC SURVEYING INC.
3. TBM = TOP OF FIRE HYDRANT SPINDLE IN FRONT OF 5561 ROCKDALE ROAD, ELEVATION = 79.23 METRES (GEODETIC).
4. ANY CHANGES MADE TO THIS PLAN MUST BE VERIFIED AND APPROVED BY KOLLAARD ASSOCIATES INC.
5. COORDINATE AND SCHEDULE ALL WORK WITH OTHER TRADES AN CONTRACTORS.
6. DETERMINE THE EXACT LOCATION, SIZE, MATERIAL AND ELEVATION OF ALL EXISTING UTILITIES PRIOR TO COMMENCING CONSTRUCTION. PROTECT AND ASSUME RESPONSIBILITY FOR ALL EXISTING UTILITIES WHETHER OR NOT SHOWN ON THIS DRAWING.
7. OBTAIN ALL NECESSARY PERMITS AND APPROVALS FROM THE CITY AND CONSERVATION AUTHORITY BEFORE COMMENCING CONSTRUCTION.
8. BEFORE COMMENCING CONSTRUCTION OBTAIN AND PROVIDE PROOF OF COMPREHENSIVE, ALL RISK AND OPERATIONAL LIABILITY INSURANCE FOR $2,000,000.00. INSURANCE POLICY TO NAME OWNERS, ENGINEERS AND ARCHITECTS AS CO-INSURED.
9. RESTORE ALL DISTURBED AREAS ON-SITE AND OFF-SITE, INCLUDING TRENCHES AND SURFACES ON PUBLIC ROAD ALLOWANCES TO EXISTING CONDITIONS OR BETTER TO THE SATISFACTION OF THE MUNICIPALITY AND ENGINEER.
10. REMOVE FROM SITE ALL EXCESS EXCAVATED MATERIAL, ORGANIC MATTER AND DEBRIS UNLESS OTHERWISE INSTRUCTED BY ENGINEER. EXCAVATE AND REMOVE FROM SITE ANY CONTAMINATED MATERIAL. ALL CONTAMINATED MATERIAL SHALL BE DISPOSED OF AT A LICENCED LANDFILL FACILITY.
11. CONTRACTOR TO PROVIDE THE CONSULTANT WITH A GENERAL PLAN OF SERVICES INDICATING ALL SERVICING AS-BUILT INFORMATION SHOWN ON THIS PLAN. AS-BUILT INFORMATION MUST INCLUDE: PIPE MATERIAL, SIZES, LENGTHS, SLOPES, INVERT & T/G ELEVATIONS, STRUCTURE LOCATIONS, VALVE AND HYDRANT LOCATIONS, T/WM ELEVATIONS AND ANY ALIGNMENT CHANGES, ETC.
12. REFERENCE TO KOLLAARD FILE #220863 FOR SITE SERVICING AND STORMWATER MANAGEMENT DESIGN.

No.	REVISION	DATE	BY
1.	ISSUED IN RESPONSE TO REVIEW COMMENTS #9	DEC 21, 2023	NJR

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Engineers
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CLIENT:
MR. JP BERGERON
880 SMITH ROAD
NAVAN, ON, K4B 1N9

PROJECT:
PROPOSED 12 UNIT
APARTMENT BUILDING

LOCATION:
5574 ROCKDALE ROAD, VARS
ON, K0A 3H0

LICENSED PROFESSIONAL ENGINEER
21.DEC.2023
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGNED BY: NJR	CHECKED BY: SD
DRAWN BY: JR	APPROVED BY: SD
DATE: DEC 21, 2023	
KOLLAARD FILE NUMBER: 220863	

DRAWING NUMBER: 220863-ESC

DRAWING NAME: EROSION SEDCIMENT CONTROL PLAN