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SERVICING AND STORMWATER MANAGEMENT REPORT
Residential Apartment Buildings

6173 RENAUD ROAD
OTTAWA, ONTARIO

Prepared For:

Teak Developments
31 Woodview Crescent
Ottawa, Ontario
K1B 3B1

PROJECT #: 190867

City of Ottawa SPC Application File # D07-12-20-0094

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1 INTRODUCTION

Kollaard Associates was retained by Mr. George Elias of Teak Developments to complete a Site Servicing and Stormwater Management Report for a new residential development in the City of Ottawa, Ontario.

1.1 Purpose

This report will address the serviceability of the proposed site, specifically relating to the adequacy of the existing municipal storm sewer, sanitary sewer, and watermains to hydraulically convey the necessary storm runoff, sanitary sewage and water demands that will be placed on the existing system as a result of the proposed development located at 6173 Renaud Road, Ottawa, Ontario. The report shall summarize the stormwater management (SWM) design requirements and proposed works that will address stormwater flows arising from the site under post-development conditions. The report and will identify and address any stormwater servicing concerns and also describe any measures to be taken during construction to minimize erosion and sedimentation.

1.2 Proposed Development

The development being proposed by Mr. George Elias is located on the north side of Renaud Road within the City of Ottawa and has a total area of 0.3444 hectares.

The property is within Ward 2 – Innes of the City of Ottawa. The property is legally described as Part of Lot 5 Concession 3 (Ottawa Front) Geographic Township of Gloucester, City of Ottawa; Part 5 of R.Plan 5R-2853 PIN 04404-0228. A topographic plan of Survey has been included in Appendix E. The property known as 6173 Renaud Road is currently occupied by an existing single family residential dwelling. It is understood that the owner of the subject site intends to demolish the existing building.

The proposed development is to consist of two "townhome buildings. One of the buildings will be a 16 unit residential "back to back stacked townhome" style building. This building will 8 units having basement and ground floor levels and 8 units having third and fourth floor levels. The second will be an 8 unit residential "back to back townhome" style building.

1.3 Referenced Documents

The following documents have been referenced during the preparation of this Servicing and Stormwater management Report. These documents are publicly available or have been provided as part of the Site Plan Control Application and are not included with this report.

- Geotechnical Investigation Report Prepared by Kollaard Associates Inc.
- Site Plan prepared by Rosaline J. Hill Architect Inc.
- Preliminary Architectural drawings of the Proposed Buildings
- City of Ottawa Sewer Design Guidelines October 2012 as amended by technical bulletins
    - ISDTB-2014-01, PIEDTB-2016-01, ISTB-2018-01, ISTB-2018-04
- City of Ottawa Design Guidelines Water Distribution as amended by technical bulletins
    - ISD 2010-2, ISDTB-2014-02, ISDTB-2018-02
- Master Servicing Study (MSS) EUC Infrastructure Servicing Study Update as prepared by Stantec Consulting Ltd, March 2005
- Page Road Development – Storm Drainage Plan Drawing # SD-1 Project 160400477 Rev 4 dated 2011 Feb 17 prepared by Stantec Consulting Ltd.

2 STORMWATER DESIGN

2.1 Stormwater Management Design Criteria

2.1.1 Background

The proposed residential development is within the Gloucester East Urban Community (EUC), adjacent to Mud Creek. A Master Servicing Study (MSS) EUC Infrastructure Servicing Study Update was prepared by Stantec Consulting Ltd, March 2005 for this community. The site is bound by residential development with Renaud Road at the south end of the site and Trailsedge Way at the north end of the site. There is an existing 975 mm diameter truck storm sewer along Renaud Road and an existing 825 mm diameter trunk storm sewer along Trailsedge Way. These sewers outlet via trunk sewers indentified in the Stantec MSS Update to EUC Pond #3. EUC Pond #3 was designed to be an end of pipe treatment facility for stormwater runoff. The Stantec MSS Update identified that the trunk sewers be sized based on a rate of 85L/s/ha.

The residential subdivision known as the Page Road Development is located along the north side of Trailsedge Way adjacent to the subject site. The existing 825 mm diameter trunk storm sewer was installed as part of the development of this subdivision. A review of the Storm Drainage Plan drawing SD-1 Revision 4 dated February 17, 2011 completed by Stantec Consulting Ltd indicates that the storm sewer design for this 825 mm diameter storm sewer included the north half of the site in the external contributing area to the storm sewer EXT-107 using a runoff coefficient of C=0.52.

2.1.2 Minor System 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" and Section 8 "Stormwater Management" as amended.

A time of concentration is to be calculated and to be no less than 10 minutes. Alternatively a pre-development time of concentration of 20 minutes could be used without calculation or engineered justification.

The storm sewers have been designed and sized based on the rational formula and the Manning's Equation under free flow conditions for the 5-year storm using a 10-minute inlet time.

The runoff rate generated during a post development 5 year design storm event will be attenuated to the lesser of the 5 year pre-development runoff rate or 85 L/s/ha.

2.1.3 Major System Design Criteria

The major system has been designed to accommodate on-site detention with sufficient capacity to attenuate the runoff rate generated onsite during a 100-year design storm to 85 L/s/ha.

On site storage is provided and calculated for up to the 100-year design storm. Calculations of the required storage volumes have been prepared using the Visual OTTHYMO Software program and have been provided in Appendix A.

2.1.4 Quality Control

The proposed development is within the EUC and the runoff from the proposed development will be conveyed to the EUC Pond #3. The EUC Pond #3 has been designed to provide quality control for the catchment area and to achieve the required treatment levels.

2.1.5 Approval Authorities

The approval authorities for the proposed stormwater management facility will consist of the Rideau Valley Conservation Authority (RVCA) and the City of Ottawa

The proposed development is residential with a single owner of both proposed buildings. The proposed stormwater management design is limited to a single site with no appreciable offsite runoff. Discharge from the site will be to an existing municipal storm sewer. As such, it is considered that an MECP ECA will not be required for the proposed stormwater management facility.

2.2 Stormwater Quantity Control

Peak Flow for runoff quantities for the Pre-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 ***m³/s***
C is the Runoff Coefficient, **Dimensionless**
A is the runoff area in ***hectares***
i is the storm intensity measure in ***mm/hr***

The hydrologic modeling software, Visual OTTHYMO (V2.6.3) was used to assess the post-development stormwater conditions at the site. The post-development conditions for the uncontrolled catchment areas having an impervious ratio of less than 20 percent were calculated using the NASHHYD watershed command. The post-development conditions for the controlled catchment areas having an impervious ratio of more than 20 percent were calculated using the STANDHYD watershed command.

The NASHYD hydrograph method uses the Nash instantaneous unit hydrograph which is made of a cascade of 'N' linear reservoirs and is used to model rural areas. The STANDHYD hydrograph method is used to simulate runoff flows from urban watersheds and uses two parallel standard instantaneous unit hydrographs modeled at the same time to combine the effective rainfall intensity over the pervious and impervious surfaces.

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 3 return periods were considered consisting of the 2, 5 and 100-year events. The formulas for each are:

2-Year Event

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

5-Year Event

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

100-Year Event

$$i = \frac{1735.071}{(t_c + 6.014)^{0.82}}$$

where t_c is time of concentration

The post-development analysis, completed using Visual OTTHYMO, considered the following storm events:
Simulation Number 1 – 6 hour 5 year Chicago
Simulation Number 2 – 12 hour 5 year Chicago
Simulation Number 3 – 6 hour 100 year Chicago
Simulation Number 4 – 12 hour 100 year Chicago
Simulation Number 5 – 12 hour 2 year Chicago

2.2.1 Runoff Coefficients – Pre-Development

Runoff coefficients for impervious surfaces (roofs, asphalt, and concrete) were taken as 0.90, for gravel surfaces were taken as 0.7 and pervious surfaces (grass) were taken as 0.25.

A 25% increase for the post development 100-year runoff coefficients was used as per City of Ottawa guidelines. Refer to Appendix A for pre-development runoff coefficients.

2.2.2 Curve Number - Post-Development

The NasHyd hydrograph method which uses the SCS loss method for pervious areas was used to model post development conditions for the uncontrolled areas. Runoff Curve Numbers (CN) are utilized in the SCS hydrology method. The Curve Number is a function of soil type ground cover, and antecedent moisture conditions. For the purposes of analysis presented in this report, the surface cover was considered to be Open Space (lawns) in good condition, Soil Type D (silty clay subgrade soils) gives CN = 80, and Impervious give CN = 98. The CN values were taken from the *Ottawa Sewer Design Guidelines* Table 5.9 (2004.)

2.2.3 Initial Abstraction and Potential Storage - Post-Development

The initial abstraction includes all losses before runoff begins, and includes water retained in surface depressions, water taken up by vegetation, evaporation, and infiltration. This value is related to characteristics of the soil and the soil cover. Initial abstraction is a function of the potential storage and is generally assumed to be equal to 0.2S where S is the potential storage. It is considered that for lower CN values, the relationship IA = 0.2S tends to overestimate the initial abstraction resulting in underestimated peak runoff.

As such, suggested guidelines are as follows:
CN ≤ 70 IA = 0.075S
CN > 70 ≤ 80 IA = 0.10S
CN > 80 ≤ 90 IA = 0.15S
CN > 90 IA = 0.2S

The potential storage S is related to the runoff coefficient as follows:
S = (25400/CN) - 254

2.2.4 Manning Coefficients, Depression Storage, Infilration – Post-Development

The Manning Roughness (n) Coefficients for overland flow selected for impervious site areas (MNI) was assumed to be 0.013 based on the City of Ottawa Sewer Design Guidelines: Appendix 6-C Manning Coefficient values for street and gutter flow assuming smooth asphalt.
The Manning's roughness coefficient for pervious surfaces (MNP) was selected to be 0.25 based on sheet flow through good quality grass in the previous areas.

Depression storage values entered into the model ware the default values obtained from Section 5.4.5.4 of the City of Ottawa Sewer Design Guidelines. The depression storage values used are 1.57 mm for impervious areas and 4.67 m for pervious grassed areas.

As previously indicated, the controlled areas were modeled using the StandHyd hydrograph method. The losses over the surfaces were calculated by the Horton's soil infiltration equation where the infiltration capacity rate is an exponential function of time, which decays to a constant rate. The Horton's equation variables were obtained from Section 5.4.5.5 of the Sewer Design Guideline where f_c = 13.2 mm/hr, f_o = 76.2 mm/hr and k = 0.00115 s^{-1}.

2.2.5 Time of Concentration

The time of concentration for pre-development conditions was calculated using the FAA method or Airport Formula.

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

The time of concentration for post-development conditions was taken as 10 minutes in accordance with recommendations from the City of Ottawa's Sewer design Guidelines.

2.2.6 Pre-development Site Conditions

As previously indicated, the site is located between Renaud Road and Trailsedge Way within the City of Ottawa. The site has a total area of about 3444 square metres and is partially developed. The site is currently occupied by a single family residential dwelling with an inground pool and associated hardscaped areas having a total footprint of about 439 square metres and an asphalt surfaced driveway with a surface area of about 180 square metres. The site is within a residential area with new development along the east side of the site and northwest side of the site. The pervious areas of the site are in general grass covered.

As indicated on drawing 190867-PRE, runoff from a portion of the adjacent rowhouse development is directed on to the site. This area includes a portion of the roof area and the rear yards between the site and the adjacent rowhouse units. During post-development conditions, the runoff from these offsite areas will be intersected by means of a shallow swale adjacent to the property line and will be directed without control to either Renaud Road or Trailsedge Way. As such, the offsite area has not been included in the stormwater model under either pre- or post-development conditions. In addition, the offsite areas will not contribute flow to any of the onsite sewer system.

As indicated on drawing 180966-PRE, runoff is directed from the building envelope to side yard property line swales and to the front and back of the site. The swales along the side property lines direct flow to the front and back of the site and intersect the flow from the adjacent site preventing offsite flow onto the site. The site has been divided into two catchment areas PRE-CA1 and PRE-CA2 to model the pre-development runoff rates to Trailsedge Way and to Renaud Road respectively.

2.2.6.1 Pre-development Runoff Coefficients

The predevelopment runoff coefficient for the site was calculated using weighted average based on the existing ground surface conditions as follows:

$$C = \frac{\left(A_{imp}\ x\ 0.9 + A_{gravel} x\ 0.7 + A_{soft}\ x\ 0.25\right)}{A_{total}}$$

PRE-CA1 (5 yr)

$$C = \frac{(0.033\ x\ 0.9 + 0.00 x\ 0.7 + 0.182\ x\ 0.25)}{0.216} = 0.35$$

PRE-CA2 (5 yr)

$$C = \frac{(0.0289\ x\ 0.9 + 0.00 x\ 0.7 + 0.100\ x\ 0.25)}{0.129} = 0.40$$

Based on the existing ground cover the pre-development runoff coefficient for the area directing runoff to Trailsedge Way was calculated to be 0.35 for a five year storm event. Based on the existing ground cover the pre-development runoff coefficient for the area directing runoff to Renaud Road was calculated to be 0.40 for a five year storm event.

2.2.6.2 Pre-development Time of Concentration

The time of concentration for pre-development conditions was calculated using the FAA method or Airport Formula to be 13 minutes.

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

Where: t_c = time of concentration
C = Runoff coefficient = 0.35
l_c = length of flow path = 51
S = slope of flow path = 1.5

t_c = 12.82 minutes which was rounded to the nearest minute as 13 minutes.

2.2.6.3 Pre-development Runoff Rate

Using the City of Ottawa IDF curve for a 5-year storm event, the storm intensity at a 13 minute time of concentration is 90.63 mm/hr.

Catchment Area to Trailsedge Way
Using the Rational Method with a storm intensity of 90.63 mm/hr, and the previously calculated runoff coefficient, the pre-development runoff rate for the 5-year design storm the catchment area out letting to Trailsedge Way is:

5 year = 0.35 x 90.63 x 0.2155 / 360 = 19.0 L/s

As previously indicated, the stormwater management design completed by Stantec for the 825 mm diameter storm sewer along Trailsedge Way was designed considering a contribution from the north half or 0.172 hectares of the site using a runoff coefficient of C=0.52. Using the Rational Method with a storm intensity of 90.63 mm/hr, C=0.52 and a catchment area of 0.172 ha provides:

5 year = 0.52 x 90.63 x 0.172 / 360 = 22.5 L/s

Also as previously indicated, the stormwater management criteria states the post-development runoff rate from the site should be restricted to the lesser of the pre-development runoff rate or 85 L/s/ha.

A runoff rate of 85 L/s/ha for the pre-development area contributing to Trailsedge Way results in an allowable runoff rate of 85 * 0.216 = 18.4 L/s.

Since the runoff rate of 18.4 L/s is less than the allowable runoff rate or 19.0 L/s when considering the pre-development conditions for the 5 year event, and also less than the external runoff or 22.5 L/s accounted for by Stantec, the allowable runoff rate of 18.4 L/s will govern for both the 5 year and 100 year events.

Catchment Area to Renaud Road
The pre-development runoff rate from the catchment area out letting to Renaud Road is:
5 year = 0.40 x 90.63 x 0.129 / 360 = 13.0 L/s

A runoff rate of 85 L/s/ha results in an allowable runoff rate of 85 * 0.129 = 11.0 L/s.

Since the runoff rate of 11.0 L/s is less than the allowable runoff rate when considering the pre-development conditions for the 5 year event, the allowable runoff rate of 11.0 L/s will govern for both the 5 year and 100 year events.

2.2.7 Controlled and Uncontrolled Areas

For the purposes of this storm water management design, the site has been divided into uncontrolled and controlled areas as outlined on drawing 190867-POST. The controlled areas are defined as area CA1 and CA2 and uncontrolled areas are defined as UA1 and UA2.

CA1 consists of the portion buildings which directs runoff to the parking area surface between the buildings as well as the parking area, landscaped areas and walkways between the buildings. CA2 consists of the remaining portion of the roofs, the parking area west of the buildings, the landscaped areas and walkways between this parking area and the buildings, as well as a portion of the landscaped area between the 8 unit rowhouse building and Trailsedge Way.

UA1 consists of the ground surface area along the perimeter of the site that directs runoff north towards Trailsedge Way without restriction. There is an existing relatively low, poorly drained, area at the southeast corner of the adjacent property known as 125 Trailsedge Way. This area is adjacent to the midpoint of the west side of the subject site. In order to provide outlet for runoff generated on this area, a low sloped swale has been included adjacent to the west property line of the subject site. Due to the existing elevations of the neighbouring property with respect to the ground surface elevation in the Trailsedge Road Allowance, the swale has a slope of about 0.1 percent. In order to reduce the potential for surface ponding in the swale, this swale will be subdrained with clear stone and 150 mm diameter perforated drain tile. The clearstone will extend to the ground surface. There is no proposed outlet for the subdrain. The subdrain is intended to improve the conveyance of offsite runoff to Trailsedge Way, promote infiltration and reduce surface ponding.

UA2 consists of the ground surface area along the perimeter of the site the landscaped area between the 16 unit stacked rowhouse building and Renaud Road that directs runoff south to Renaud Road without restriction.

Runoff from CA1 will be directed by means of downspouts and sheet flow to the parking area between the buildings where it will collected by means of a catch basin which outlets by means of storm sewer to the storage tanks below the parking area in CA2. Runoff from the remaining portion of the building roofs will be directed by eaves troughs and downspouts to an onsite storm sewer which will direct the runoff to an underground storage tank located below the parking area along the west side of the site in CA2. Runoff from the controlled parking areas, walkways and landscaped surfaces will be directed by means of sheet flow to catch basins which will capture the runoff.

The catch basins will discharge to the underground storage tanks as well. The release from the catchbasin in CA1 as well as the discharge from the storage tank will be controlled by means of a Hydrovex Flow Regulators. Discharge from the site will be released to the storm sewer along Trailsedge Way. Post-development site conditions are summarized in the following Table 2.1.

The following post-development runoff conditions have been built into the stormwater management facility:

- The walkways along the side of the building will be surfaced with permeable pavers.
- No credit in terms of reduced runoff has been assumed for the permeable pavers along the walkway areas.

Table 2.1 - Post Development Site Conditions

Parameters	Controlled Area CA1	Controlled Area CA2	Uncontrolled Area UA1	Uncontrolled Area UA2
Hydrograph Number	1	2	4	3
DT (calculation time step)	5 min	5 min	5 min	5 min
CN (curve number)	93	88	84	85
C (Runoff Coefficient)	0.72	0.46	0.41	0.41
Area	1066	1402	424	552
XIMP (Directly Connected Impervious area)	0.60	0.42	N/A	N/A
TIMP (Total Impervious Area)	0.73	0.46	N/A	N/A
DWF (dry weather flow)	0	0	0	0

LOSS Method	program default		N/A	N/A
MNP (Manning's roughness sheet flow)	0.25	0.25	N/A	N/A
DPSI (Depression Storage Imperv.)	1.57 mm	1.57 mm	N/A	N/A
LGI (length to width ratio)	program default Area = 1.5 x L^2		N/A	N/A
MNI (Manning's roughness channel flow)	0.013	0.013	N/A	N/A
IA (initial abstraction)	N/A	N/A	7.1	7.0
N (number of linear reservoirs)	N/A	N/A	3	3
TP (time to peak)	N/A	N/A	0.17 h (10 min)	

2.2.8 Uncontrolled Area Runoff

The runoff from the uncontrolled areas as calculated using the NasHyd hydrograph method to as follows:

The uncontrolled runoff from UA1 directed to Trailsedge Way is:
Simulation Number 1 – 6 hour 5 year Chicago = 3 L/s
Simulation Number 2 – 12 hour 5 year Chicago = 3 L/s
Simulation Number 3 – 6 hour 100 year Chicago = 7 L/s
Simulation Number 4 – 12 hour 100 year Chicago = 7 L/s
Simulation Number 5 – 12 hour 2 year Chicago = 2 L/s

The uncontrolled runoff from UA2 directed to Renaud Road is:

Simulation Number 1 – 6 hour 5 year Chicago = 4 L/s
Simulation Number 2 – 12 hour 5 year Chicago = 4 L/s
Simulation Number 3 – 6 hour 100 year Chicago = 9 L/s
Simulation Number 4 – 12 hour 100 year Chicago = 10 L/s
Simulation Number 5 – 12 hour 2 year Chicago = 2 L/s

2.2.9 Allowable Release Rate to Trailsedge Way

As previously indicated, the stormwater management design criteria requires that post-development runoff rates be limited to the lesser of the pre-development runoff rate for the site or 85L/s/hectare. As such, the stormwater management criteria requires that the maximum runoff rate from the site to Trailsedge Way be restricted to 85L/s/ha x 0.216 ha = 18.4 L/s.

The maximum allowable runoff rate from the site to Trailsedge Way during 100 year post development storm is 18.4 L/s. It is noted that the pre-development runoff rate directed from the site to Trailsedge Way during a 100 year storm event is 50.5 L/s.

Storm water runoff from the controlled areas CA as well as from the uncontrolled area UA1 is directed to Trailsedge Way. Uncontrolled runoff from UA2 is directed to Renaud Road and does not affect the allowable release rate to Trailsedge Way. The allowable release rate from the controlled area is equal to the total allowable runoff rate less the runoff rate from the uncontrolled area UA1.

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

For the 5-year Storm event
$\mathbf{Q_{controlled}}$ = 18.4 – 3 = 15.4 L/s

For the 100-year Storm event
$\mathbf{Q_{controlled}}$ = 18.4 – 7 = 11.4 L/s

Since the allowable release rate during the 100-year storm is more restrictive than the allowable release rate during the 5-year storm event, the allowable release rate for the 100 year storm event is the governing criteria.

2.2.10 Post Development Restricted Flow and Storage

In order to meet the stormwater quantity control restriction, the post development runoff rate from the controlled areas of the site cannot exceed the above allowable release rates. Runoff generated on the controlled areas of the site in excess of the allowable release rate will be temporarily stored on the parking area surface between the buildings and within undersurface storage tanks placed within the north half of the parking area along the west side of the site. The stored water will be released at a controlled rate during and following the storm event.

2.2.10.1 Catchment CA1

In order to achieve the allowable controlled area storm water release rate, storm water runoff from the surface areas in CA1 will be directed by sheet flow to the parking area surface between the buildings. The runoff collected on the center parking area surface will be outlet by means of catch basin CB1 and discharged to the proposed storm sewer under the west parking area by means of a 250 mm diameter PVC storm pipe. The discharge rate from CB1 will be restricted by means of a Hydrovex Flow Regulator Model 75-SVHV-1. It is emphasized that flow from catchment area CA1 does not discharge directly from the site but discharges to storm sewer and storage in catchment CA2 where it will be further restricted. The purpose of restricting the flow in CA1 is to reduce the volume of the underground storage required under the west parking area in CA2.

Stormwater Storage will be provided in CA1 on the parking area surface and below grade in a clear stone reservoir. The clear stone reservoir will have a foot print of 20 m^2 and a depth of 1.1 metre for a total stone volume of 22 m^3. The stone used in the reservoir will consist of 25 mm clean washed crushed stone (septic stone). Conservatively considering the void ratio in the stone to be 0.35, there will be a stormwater storage volume in the clearstone of 7.7 m^3. The clearstone will be wrapped in a 6 ounce per square yard non-woven geotextile filter fabric. The clearstone storage will be connected to the catch basin using a 250 mm diameter perforated pipe.

The Hydrovex Flow Regulator to be placed in CB1 can be order using the following specification:

Model	75-SVHV-1
Pipe Outlet	250 mm PVC SDR 35
Discharge	8.0 L/s
Upstream Head	2.7 m
Catch basin Dimensions	0.6 x 0.6 metres
Minimum Clearance	0.45 m
HGL @ Design Head	86.0 m
Invert Elevation	83.3 m

The catch basin in the center parking area will have a top of grate elevation of 85.75 metres and the outlet pipe will have an invert of 83.30 metres. The center parking area will overflow across the full width of the drive aisle between the west parking area and the center parking area at an elevation of 86.05 metres resulting in a maximum ponding depth of 0.3 metres on the surface of the center parking area. Overflow from the center parking area will be in the form of weir flow.

The outlet restriction in CB1 from the storage on the center parking area will result in the release rate and storage requirement on the center parking area as summarized the following Table 2.2.

Table 2.2 – Summary of Post-Development Release rates and Storage Requirement
Center Parking Area

Return period	Actual Release rate	Required Storage	Available Storage Below Grade	Total Available Storage	Required Storage Depth*	Available Storage Depth
(years)	(L/s)	(m^3)	(m^3)	(m^3)	(m)	(m)
Controlled Catchment Area CA1						
2	7	9	9.2	41.2	0.0	0.30
5	8	15	9.2	41.2	0.15	0.30
100	8	34	9.2	41.2	0.27	0.30

* On the parking Area surface. The maximum depth occurs over the catchbasin in the center of the drive aisle. The maximum depth at the parking spaces during a 5 year event will be 0.12 metres due to the slope of the parking area surface.

2.2.10.2 Catchment CA2

In order to achieve the allowable controlled area storm water release rate, storm water runoff originating from the roof areas in CA2 will be captured by means of eaves troughs and will be directed by downspouts to storm sewers which will outlet to underground storage tanks in below the west parking area. The stormwater runoff originating from ground surfaces in CA2 will be directed by means of sheet flow to catch basins within the area. The runoff collected from the ground surface and roof areas in CA2 as well as the discharge from CB1 in catchment area CA1 will be directed to the underground storage tanks below the west parking area.

Since the native soils at the site consist of highly plastic silty clay, there will be no significant infiltration from the tanks to the surrounding soils. For this reason, the proposed stormwater tanks have been designed as storage tanks only and not infiltration tanks. The geotechnical report for the site indicates that the groundwater level is expected to be encountered at an elevation of about 82.4 metres. Since the expected groundwater level will be below the tanks and the hydraulic conductivity of the surrounding highly plastic silty clay is low, there is also expected to be no significant infiltration from the ground into the tanks. Therefore, the potential of an elevated groundwater level has no significant impact on the design of the proposed storage tanks.

The discharge into the underground storage tanks below the west parking area from CA1 was accounted for by the OTTHYMO Program.

The underground storage will be divided into two groups with each group containing modules forming one storage tank. The flow to and from the storage tanks will be facilitated by means of a 250 mm diameter storm sewer, located at the north end of each tank, connected to a catchbasin manhole. Each catchbasin manhole is connected to the proposed storm sewer along the west parking water which discharges to a maintenance hole STMH1 located in the drive aisle between the west parking area and the north property line. Release from the tanks to the maintenance hole will be uncontrolled. Discharge from maintenance hole STMH1 will be controlled by a Hydrovex Flow Regulator Model 100-SVHV-2 and will be directed to the existing 825 mm diameter trunk sewer along Trailsedge Way. The outlet pipe from maintenance hole STMH1 will have an invert of 81.85 m.

The Hydrovex Flow Regulator can be order using the following specification:

Model	100-SVHV-2
Pipe Outlet	250 mm PVC SDR 35
Discharge	11.8 L/s
Upstream Head	2.75 m
Maintenance Hole Diameter	1.2 metres
Minimum Clearance	0.45 m
HGL @ Design Head	84.6 m
Invert Elevation	81.85 m

The above outlet restrictions from the underground storage tanks result in the storage requirement as summarized the following Table 2.3.

Table 2.3 – Summary of Post-Development Release rates and Storage Requirement West Parking Area

Return period	Actual Release rate	Required Storage	Available Storage Below Grade	Total Available Storage	Required Surface Storage Depth	Available Surface Storage Depth
(years)	(L/s)	(m^3)	(m^3)	(m^3)	(m)	(m)
Controlled Catchment Area CA2						
2	10	7	57.9	62.3	0	0.10
5	10	19	57.9	62.3	0	0.10
100	12	50	57.9	62.3	0	0.10

One of the two groups of the underground storm tanks will consist of 66 Brentwood ST-36 modular storage tanks and the other group will consist of 60 Brentwood ST-36 modular storage tanks. The groups of tanks will placed in a single layer having a total footprint of about 19.2 metres long by 2.7 metres wide (1 group will be 10.1 metres long, the other 9.1 metres long). Each Brentwood ST-36 modular unit is 0.457 m x 0.914 m x 0.914 m (W x L x H) and has a void ratio of 0.969. The bottom of the storage tanks will be at an elevation of 83.4 metres. The top of these storage tanks will be at 84.31 metres. The lowest finished ground surface above these storage tanks will be at an elevation of 85.2 metres. The storage tanks will be equipped with sumps at each inlet location and at the outlet location. The sump will consist of an additional module installed below the main tank at each location. The sumps will facilitate sediment trapping and drainage of the tanks.

Overflow from the storage tanks onto the west parking area surface will occur by means of the grate on CBMH4 at an elevation of 85.15. The west parking area surface will overflow to Trailsedge Way at an elevation of 85.25 metres. This provides additional storage volume on the parking area surface of about 4.4 m^3 at a surface ponding depth of 0.10 m. The minimum grade within the controlled area adjacent the building is at an elevation of 95.9 which is 0.6 metres above the overflow elevation. The minimum grade within the window wells will be 85.7 metres, which is 0.45 metres above the overflow elevation ensuring that stormwater ponding will not negatively affect the window well drainage.

2.2.11 Total Runoff Rate from Site

As indicated in the stormwater management criteria, the stormwater runoff from the site had to be less than or equal to the lesser of the pre-development conditions or 85L/s/ha. Additional consideration was provided in section 2.2.6.3 of this report to ensure that the total runoff from the site to Trailsedge Way did not exceed the runoff rate from the site considered by Stantec during the design of the Trailsedge Way storm sewer.

The total runoff rate from the site to Trailsedge Way during the 5 year and 100 year design storms was obtained from the results of the analysis completed using the OTTHYMO Stormwater management model included in Appendix B of this report. The model also provides the runoff rate to Renaud Road. The results of the analysis are summarized in the following table 2.4. It is noted that the results of the two year storm event are not included as the flow restrictions required during the 100 year storm events required to meet the governing allowable flow rate determine the design.

Table 2.4 Summary of Stormwater Runoff

	Catchment ID	Catchment Area	Outlet Location	5 year Storm Event Runoff	100 year Storm Event Runoff
		m^2		L/s	L/s
Predevelopment					
Pre-dev	PreCA1	2155	Trailsedge Way	19.0	39.0
Stantec Allowance				22.5	N/A
85 L/s/ha				18.4*	18.4*

Pre-dev	PreCA2	1289	Renaud Road	13.0	25.6
Stantec Allowance				N/A	N/A
85 L/s/ha				11.0*	11.0*
Post-Development					
OTTHYMO	CA1+CA2+UA1	2892	Trailsedge Way	13.0	18.0
OTTHYMO	UA2	552	Renaud Road	4	10

* Governing allowable flow rate.

From the above table 2.4: The post-development runoff from the site to Trailsedge Way during both the 5 year and 100 year storm events is less than the allowable runoff rate to Trailsedge Way. The post-development runoff from the site to Renaud Road during both the 5 year and 100 year storm events is less than the allowable runoff rate to Renaud Road.

2.2.12 Underground Storage Tanks

The underground storage will be provided using Brentwood StormTank Modular Tanks. A Brentwood StormTank Module is a subsurface storage unit load-rated for use under surfaces such as parking lots, athletic fields, and parks as well as landscaped areas. Design information for the Brentwood StormTanks is provided in Appendix B. It is considered that there are similar modular stormwater management systems that are directly comparable to the Brentwood Modular Tank system. The developer / sewer contractor may propose the use of an alternative equivalent modular product. Shop drawings should be submitted to the design engineer prior to acceptance of equivalency. Shop drawings should be submitted to the design engineer or the Brentwood StormTank or accepted equivalent system for approval prior to installation.

The City of Ottawa Sewer design guideline indicates that an assumed constant flow rate during a storm event underestimates the required storage during a storm event. The discharge rate from the proposed underground storage tank will range from 9.3 L/s when the tank is near empty to 11.9 L/s when the tank is full and 13.0 L/s at overflow to Trailsedge. The discharge rate when the proposed underground storage is half full is 10.8 L/s. The required storage volume assuming a discharge rate of 10.8 L/s would be 51 m^3 during a 100 year storm event which is less than the total storage available below the parking surface.

As previously indicated, the underground tanks are comprised of ST-36 Modular Units. The modules will be placed in one group of 66 modules and one group of 60 for total of 126 modules. The tanks will be wrapped in a nonwoven 6 oz/yd^2 geotextile filter fabric to prevent stone intrusion into the tanks. The tanks will then be surrounded with a 200 mm thick layer of 25 mm clearstone on all sides and a 200 mm thick clearstone layer on the bottom and a 400 mm thick layer on the top. It is understood that this will potentially promote infiltration into the adjacent soils below the tank. The clearstone will also be separated from the surrounding soils by a nonwoven geotextile. The discharge rate from the tanks into the surrounding soils has not been accounted for in the design as the surrounding soils are silty clay. Since the bottom of the tanks will be below the level of the adjacent foundations, infiltration from the tanks will be below the foundations and will not have an impact on the groundwater level at the foundation level.

It is noted that the tank will have an additional modules placed below the tank bottom at the inlet/outlet to provide sedimentation sumps and to facilitate the tank outlet. The additional modules have not been included in the available storage calculations as they could be partially filled prior to the beginning of the storm event.

As previously indicated, discharge from the underground storage tank is by means of STMH1. The restriction on the runoff rate from the underground storage tank is provided by a Hydrovex ICD in STMH1.

2.3 Offsite Runoff and Side Yard Swales

As previously indicated, the runoff from the adjacent properties to the east and west of the proposed development is will be intersected by the proposed side yard swales and directed to either Trailsedge Way or Renaud Road. The offsite runoff will not be directed to the onsite stormwater management works and was not included in the analysis of the pre- and post-development conditions. The offsite catchment area for each of the swales was estimated based on available topographic information and imagery obtained from GeoOttawa mapping. The portion of the uncontrolled onsite area contributing to each swale was added to the offsite area to determine the peak runoff rate in each swale. It is noted that this uncontrolled area has already been included in the previously completed analysis used to design the onsite stormwater management works.

The runoff rate in each swale due to the offsite contributing area was determined using Visual OTTHYMO, and considering the following storm events:
Simulation Number 2 – 12 hour 5 year Chicago
Simulation Number 4 – 12 hour 100 year Chicago
Simulation Number 5 – 12 hour 2 year Chicago

The NASHHYD hydrograph was used for each of the catchment areas contributing runoff to the swales. The catchment area contributing runoff to the east swale was estimated to have an offsite area of 540 m^2 and a total area of 775 m^2 with a runoff coefficient of C = 0.40 and a curve number CN = 84. The catchment area contributing runoff to the west swale was estimated to have an offiste area of 517 m^2 and a total area of 650 with a runoff coefficient of C = 0.30 and a curve number CN = 81.

The resulting analysis provided the peak runoff rate in each swale as summarized in the following Table 2.5. This runoff rate was used to determined to the maximum flow depth and velocity in each side yard swale.

Table 2.5 – Peak Runoff in Sideyard Swales

Storm Event	East Side Swale	West Side Swale
12 hour 2 year Chicago	0.002 m^3/s	0.001 m^3/s
12 hour 5 year Chicago	0.005 m^3/s	0.003 m^3/s
12 hour 100 year Chicago	0.013 m^3/s	0.009 m^3/s

2.3.1 East Side Yard Swale

The east side yard swale was designed with a "V" shaped bottom, a longitudinal slope of 1.5 percent and 3H:1V side slopes. The above flow rates will result in flow depths and velocities as as summarized in Table 2.6 below:

Table 2.6 – Flow Depth and Velocity in East Swale

Storm Event	Depth	Velocity
12 hour 2 year Chicago	0.03 m	0.31 m/s
12 hour 5 year Chicago	0.06 m	0.44 m/s
12 hour 100 year Chicago	0.09 m	0.58 m/s

2.3.2 West Side Yard Swale

The west side yard swale is designed to be trapezoidal shaped with a bottom width of 0.3 m, 3H:1V side slopes, and a longitudinal slope of about 0.1 percent. The above flow rates will result in flow depths and velocities as as summarized in Table 2.7 below:

Table 2.7 – Flow Depth and Velocity in West Swale

Storm Event	Depth	Velocity
12 hour 2 year Chicago	0.03 m	0.10 m/s
12 hour 5 year Chicago	0.04 m	0.12 m/s
12 hour 100 year Chicago	0.09 m	0.19 m/s

2.4 Stormwater Quality Control

As previously indicated in the report, quality control requirements will be met by the storm water management ECU storm pond 3.

In addition to the offsite end of pipe facilities, the following onsite quality control measures are proposed.

The major source of stormwater contamination from a development site is the onsite surface parking areas and walkways.

The surface areas at the site consist of the roof of the building, the landscaped areas, parking areas and the walkways.

- The roof of the building is not considered to be a major source of suspended solids contamination.
- The runoff from surface area of the parking areas will be directed to catch basins equipped with standard sumps. The catch basins will outlet to the stormwater storage tank at a location where an additional sump has been built into the tank for secondary sedimentation.
- The landscaped areas are not considered to be a source of suspended contamination as the landscaped areas provide vegetative filtration of the surface runoff and the vegetation and landscaping protects the ground surface reducing the potential for erosion and eliminating the landscaped ground surface area as a source of suspended solids.
- The walkways and amenity area can be a source of suspended solids especially during winter snow and ice removal. The use of permeable unit pavers reduces the amount of salt and other snow and ice removal products required. In addition, the runoff from the majority of the walkway and amenity area is directed to the adjacent landscaped surface prior to being collected or discharged from the site.

Best management practices will be incorporated at the site to reduce potential suspended solid contamination. Snow and Ice control management practices which include proper timing of the application of the salt and sand will be incorporated to reduce contamination from winter snow and ice removal.

2.5 Stormwater System Operation and Maintenance

2.5.1 Inlet Control Device (ICD)

The inlet control device (ICD) should be inspected on a semi-annual basis and following major storm events. Any blockages, trash or debris should be removed.

2.5.2 Catchbasin/ Manhole and Inspection Ports

The catchbasin / manhole and inspection ports (including sediment traps in storm tanks) should be cleaned with a hydrovac excavation truck following completion of construction, paving of the asphaltic concrete surface, placement of the walkway and exterior parking pavers and establishment of adequate grass cover on the landscaped areas.

Following the initial cleaning these structures should be inspected on a semi-annual basis and following major storm events. Any blockages, trash or debris should be removed. Once the sediment accumulation in the catchbasin / manhole has reached a level equal to 0.2 metres below the outlet invert of the structure, or a thickness of 0.15 metres in the sediment traps, the sediment should be removed by hydro excavation.

2.5.3 Brentwood StormTank Storage Tanks

Detailed installation, operation and maintenance guidelines are provided in the StormTank Module Design Guide included in Appendix B. In general maintenance procedures consist of Inspection and cleaning as follows:

Inspection:

- Inspect all observation ports, inflow and outflow connections, and the discharge area.
- Identify and log any sediment and debris accumulation, system backup, or discharge rate changes.
- If there is a sufficient need for cleanout, contact a local cleaning company for assistance.

Cleaning:

- If a pretreatment device is installed, follow manufacturer recommendations.
- Using a vacuum pump truck, evacuate debris from the inflow and outflow points.
- Flush the system with clean water, forcing debris from the system.
- Repeat steps 2 and 3 until no debris is evident.

2.6 Storm Sewer Design

The on-site storm sewers were designed to be in general conformance with the City of Ottawa Sewer Design Guidelines (October 2012). Specifically, storm sewers were sized using Manning's Equation, assuming a roughness coefficient N = 0.013, to accommodate the uncontrolled runoff from the 5-year storm, under 'open-channel' conditions. The uncontrolled runoff was determined using the rational method and the City of Ottawa IDF curve for a 10-minute time of concentration. Refer to Storm Sewer Design Sheet in Appendix A.

The storage volume within the storm pipes and structures (catch basins and maintenance holes) has not been utilized in the calculations for available storage in the proposed stormwater management facility. Since these unaccounted volumes are small, this will have no significant impact to the stormwater management facility and any impact that does occur will not have a negative effect to the design.

2.7 Storm Sewer Main Along Trailsedge Way

The storm sewer system drawing *Storm Sewer System Revision 2 March 2005 Dwg No. STM* in the Master Servicing Study Gloucester East Urban Community (EUC) Infrastructure Servicing

Study Update, (MSS) indicates that the north half of the proposed development site is be serviced from Trailsedge while the south half of the site is to be serviced from Renaud Road.

Dwg No. STM of the MSS indicates that:

- The storm sewer along Renaud Road, as indicated by the MSS as the receiver of runoff from the site, is part of the catchment area discharging to Storm Manhole 601.
- The storm sewer along Trailsedge Way receiving the proposed storm discharge from the site is part of the catchment area discharging to Storm Manhole 602.

- The Storm Sewer Calculation Sheet (Rational Method) – Pond 3 associated with Dwg No. STM clearly shows the storm sewer flow from Node 601A to Node 601, then from Node 601 to Node 602, then from Node 602 to Node 603. As such, all the flow from the site was intended by the MSS to be included in the storm sewer system to which the runoff from the site is directed.
- Since the discharge from the portion of the site, intended to be directed to Renaud Road, is discharged to the same storm sewer system as intended by the MSS, the proposed design is in keeping with the MSS and there will be sufficient capacity as determined by the MSS.

Alternatively:

As previously indicated, the existing 825 mm storm sewer main along Trailsedge Way was installed during the development of the adjacent residential subdivision development known as the Page Road Development. The stormwater management design for this development was Completed by Stantec Consulting Ltd. Stantec drawing number SD-1, Project number 160400477 Rev 4 dated February 17, 2011 indicates that the storm sewer design included runoff from the north half of the subject site and considered the subject site area to have a runoff coefficient equal to C=0.52.

As indicated in section 2.2.6.3 of this report, the portion of the catchment area EXT-107 occupied by the subject site, which was included by Stantec, would result in a runoff of 22.5 L/sec during a 5 year storm event. Since this flow is greater than the allowable flow from the site determined by the runoff criteria of 18.4 L/sec, a flow rate greater than the allowable flow from the site has been accounted for in the design of the Trailsedge Way storm sewer trunk. As such, there is sufficient capacity within the 825 mm diameter Trailsedge Way storm sewer trunk to accommodate the allowable flow from the site.

3 SANITARY SEWER DESIGN

As previously indicated, the site is within the Gloucester East Urban Community. The site is currently occupied by a single family dwelling which will be demolished prior to the proposed development.

Sewage discharges will be domestic in type and in compliance with the City of Ottawa Sewer Use By-law. The anticipated peak sanitary flow from the building will be a total of approximately 0.69 L/s.

The sanitary sewage flow for the proposed building was calculated based on the City of Ottawa Sewer Design Guidelines (Section 4.4.1.2) and incorporated Technical Bulletin ISTB-2018-01.

3.1 Design Flows

As previously indicated, the proposed development will consist of one 16 unit residential "back to back stacked townhome" style building and one 8-unit "back to back townhome" style building. The 16 unit building will contain 12 – two bedroom units and 4 – three bedroom units. The 8 unit building will contain 6 – two bedroom units and 2 – three bedroom units.

Residential

Total domestic pop:

2 Bedroom units (18) x 2.1 ppu:	37.8 rounded to 38
3 Bedroom units (6) x 3.1 ppu:	18.6 rounded to 19
Total:	56.4 rounded to 57

$Q_{Domestic}$ = 57 x 280 L/person/day x (1/86,400 sec/day) = 0.18 L/sec

$$\text{Peaking Factor} = 1 + \frac{14}{4 + (57/1000)^{0.5}} \times 0.8 = 3.64 \text{ - maximum } 4.0$$

$Q_{Peak\ Domestic}$ = 0.18 L/sec x 3.64 = 0.67 L/sec

Infiltration

$Q_{Infiltration}$ = 0.33 L/ha/sec x 0.34444 ha = 0.11 L/sec

Total Peak Sanitary Flow = 0.67 + 0.11 = 0.79 L/sec

3.2 Sanitary Service Lateral

A private sanitary sewer main will be extended beneath the west parking area from the existing sanitary sewer along Trailsedge Way to a proposed manhole near the west end of the southern of the two buildings. A single sanitary service will be extended from each building to the proposed private sanitary main.

The Ontario Building Code specifies minimum pipe size and maximum hydraulic loading for sanitary sewer pipe. OBC 7.4.10.8 (2) states "Horizontal sanitary drainage pipe shall be designed to carry no more than 65% of its full capacity." A 135 mm diameter sanitary service with a minimum slope of 1.0% has a capacity of 11.51 Litres per second.

The maximum peak sanitary flows from one building is 0.57 L/sec (38 x 280 L/person/day x (1/86,400 sec/day) x 3.67 = 0.57). Since 0.57 L/sec is much less than 0.65 x 11.51 = 7.48 L/s, the sanitary service would be properly sized if greater than or equal to 135 mm in diameter.

Table 3.1 Fixture Unit Consideration per Building

Apartment Unit Type	Number of Apartments	Number of fixture units per apartment	Total number of Fixture Units.
• 2 Bedroom 1.5 bathrooms	8	17.0	136
• 2 Bedroom 2.5 bathrooms	4	23.0	92
• 3 Bedroom 2.5 bathrooms	4	23.0	92
• Total fixtures			320

From Table 7.4.10.8, the allowable number of fixture units for a 135 mm diameter sanitary service pipe at 1.0% slope is 390. There are approximately 320 fixtures in the building. As such a 135 mm diameter sanitary service will technically be adequate to meet the hydraulic demands for the proposed sanitary flow. It is considered however that a minimum sanitary service size of 152 mm diameter be used for multiunit residential development of the sized proposed. Both sanitary services should however be equipped with a backflow preventer.

The proposed sanitary services will be connected to the proposed private sanitary main at inverts of 83.15 for the south building and 82.50 for the north building. The proposed sanitary main will connect to the existing sanitary sewer along Trailsedge way at a proposed invert of 80.45 metres. The minimum underside of footing elevation for the proposed buildings is 84.30 metres. As such the proposed building grade will be above the HGL of the sanitary sewers. The proposed private sanitary main will be connected to the existing sanitary main in accordance with City of Ottawa Standard Drawing S11.1.

3.3 Sanitary Main

The sanitary sewer system drawing *Sanitary Sewer System Revision 2 March 2005 Dwg No. SAN* in the Master Servicing Study Gloucester East Urban Community (EUC) Infrastructure Servicing Study Update, (MSS) indicates that the north half of the proposed development site is be serviced from Trailsedge while the south half of the site is to be serviced from Renaud Road.

In the following section, the estimated demand on the existing sanitary sewer along Trailsedge is compared to the capacity of the existing sewer to determine if there is sufficient capacity for the additional flow from the proposed development. The existing demand was calculated by considering both the area and population indicated on Dwg No. SAN as well as the estimated area and population determined using the as-built infrastructure data provided on the City of Ottawa geoOttawa online mapping system.

It is noted that the actual construction of the sanitary sewer system differs from the proposed construction indicated in the MSS.

3.3.1 Demand Calculated Using Dwg No. SAN

The Trailsedge Way sanitary sewer is indicated by Dwg No. SAN to service a residential development with a catchment area of 8 hectares with a population of 395.

$Q_{Domestic}$ = 395 x 280 L/person/day x (1/86,400 sec/day) = 1.28 L/sec

$$\text{Peaking Factor} = 1 + \frac{14}{4 + (395/1000)^{0.5}} \times 0.8 = 3.42 \text{ - maximum } 4.0$$

$Q_{Peak\ Domestic}$ = 1.28 L/sec x 3.42 = 4.38 L/sec

<u>Infiltration</u>

$Q_{Infiltration}$ = 0.33 L/ha/sec x 8 ha = 2.64 L/sec

Total Peak Sanitary Flow = 4.38 + 2.64 = 7.02 L/sec

3.3.2 Demand Estimated From geoOttawa

The existing sanitary sewer main along Trailsedge services the Trailsedge residential development north of the subject site. This existing development is mostly occupied by rowhouse (townhouse) development. The contributing area to the 200 mm diameter PVC sanitary sewer along Trailsedge way adjacent the site is approximately 9.2 hectares. Using imagery obtained from the City of Ottawa geoOttawa online mapping system, the number of units per hectare was estimated to be 38. This provides a total of 350 units and a population of 945 persons.

$Q_{Domestic}$ = 945 x 280 L/person/day x (1/86,400 sec/day) = 3.06 L/sec

$$\text{Peaking Factor} = 1 + \frac{14}{4 + (945/1000)^{0.5}} \times 0.8 = 3.25 \text{ - maximum } 4.0$$

$Q_{Peak\ Domestic}$ = 3.06 L/sec x 3.25 = 9.96 L/sec

<u>Infiltration</u>

$Q_{Infiltration}$ = 0.33 L/ha/sec x 9.2 ha = 3.04 L/sec

Total Peak Sanitary Flow = 9.96 + 3.04 = 13 L/sec

3.3.3 Capacity of Existing Sewer

The existing sanitary sewer main along Trailsedge way consists of a 200 mm diameter PVC sewer at a slope of 0.33% and a capacity of 18.9 Litres per second. As such the existing sanitary demand on the 200 mm sewer adjacent the site is equal to 13 / 18.9 = 69 percent of the capacity of the sewer. This 200 mm sewer discharges to a 300 mm sewer approximately 60 metres downstream of the proposed connection location. The 300 mm sanitary sewer has a length of about 97 metres and discharges into the 600 mm trunk sewer along Renaud Rd

The additional peak demand resulting from the proposed development consists of 1.0 L/sec which will increase the demand on the existing 200 mm sewer from 69% of its capacity to 74 percent of its capacity leaving a residual capacity of 26 percent. Alternatively, the total demand on the existing sewer along Renaud Rd following the completion of the proposed development will be (7.02+1.0) / 18.9 = 42.4 percent of the capacity of the sewer when considering the information provided in the MSS leaving a residual capacity of about 58 percent.

Therefore, it is considered that there is sufficient capacity in the existing sanitary sewer for the proposed development.

4 WATERMAIN DESIGN

4.1 Water Demand

The water demand for the proposed development was calculated based on the City of Ottawa Water Distribution Design Guidelines as follows:

Residential

Total domestic pop:	
2 Bedroom units (18) x 2.1 ppu:	37.8 rounded to 38
3 Bedroom units (6) x 3.1 ppu:	18.6 rounded to 19
Total:	56.4 rounded to 57

Residential Average Daily Demand = 350 L/c/d.

- Average daily demand of 350 L/c/day x 57 persons = 19,950 Litres/day or 0.23 L/s
- Maximum daily demand (factor of 2.5) is 0.23 L/s x 2.5 = 0.58 L/s
- Peak hourly demand (factor of 2.2) = 0.58 L/s x 2.2 = 1.27 L/s

It is noted that the residential demand at the time the flows were submitted for boundary conditions was originally based on 2 buildings containing 16 units each. As such, the residential flow demand submitted for boundary conditions consisted of an average daily demand of 0.4 L/s and a maximum hourly demand of 2.21 L/s.

4.2 Fire Flow

Fire flow protection requirements were calculated in accordance with City of Ottawa Technical Bulletin ISTB-2021-03. That is: "The requirements for levels of fire protection on private property in urban areas are covered in the Ontario Building Code (OBC). If this approach yields a fire flow greater than 9,000 L/min then the Fire Underwriter's Survey methodology shall be used. Calculations of the fire flow required are provided in Appendix D. The fire flow requirements calculated using the OBC are 5,400 L/min, or 90 L/s. Since this demand is less than 9,000 L/min the OBC calculation will be used.

A request for boundary conditions was submitted to the City of Ottawa in January of 2020. The fire flow calculations were completed before City of Ottawa Technical Bulletin ISTB-2021-03 was released. As such the fire flow demand calculations were completed using the FUS methodology and the fire flow demand was determined to be 166.7 L/s.

4.3 Boundary Conditions and Sufficiency of Existing Infrastructure

The proposed development is within the City of Ottawa water distribution network pressure zone 2E. From the City of Ottawa Digital Pressure Model Minimum static pressure mapping there is expected to be a minimum pressure of 380 kPa at the site which corresponds to a hydraulic grade line of about 123.7 m.

The boundary conditions were provided to Kollaard Associates for a connection to Trailsedge Way and have been included in Appendix E. The boundary conditions provided are summarized in the following Table 4.1

Table 4.1 – Summary of Boundary Conditions

Demand Scenario	Head (m)	Pressure[1] (psi)	Pressure (kPa)
Maximum HGL	130.6	64.8	446.8
Peak Hour	126.5	58.9	406.1
Max Day plus Fire	119.5	48.9	337.1
1 – Ground Elevation = 85.1 m			

4.3.1 Existing Water Service

The site is currently occupied by a single family dwelling which has a residential water service connected to the 305 mm water main along Renaud Road. This water service will not be sufficient for the proposed development. The existing water service will be replaced beginning at the existing stand pipe and will be connected to the proposed watermain extended across the site from Trailsedge Way. The connection will be made by means of a reducer at the end of the proposed main. The water pipe used to replace the existing stand pipe and the reducer should be a single length with no joints and should match the diameter of the existing service. This will provide looping through the site and will prevent any dead end sections of watermain pipe.

The existing service diameter and the condition of the existing service should be confirmed prior installation of the watermain and reducer. If the existing service and standpipe are in poor condition, the existing water service is to be abandoned at the main. The existing water service could then be either replaced in its entirety or remain abandoned and a private hydrant could be added for flushing purposes.

4.3.2 Existing Fire Hydrants

The existing fire hydrants within the vicinity of the site are located as follows: At the northwest corner of the site across Trailsedge Way; 50 metres east of the site across Trailsedge Way, 55 metres east of the site across Renaud Road; 31 metres west of the site across Renaud Road.

City of Ottawa Technical Bulletin ISTB-2018-02 Appendix I Table 1 provides guidance with respect to maximum flow from to be considered from a given hydrant. From this table, a Class AA hydrant can contribute a maximum flow of 5,700 L/min when located less than 75 metres from the building and 3,800 L/min when located between 75 and 150 metres from the building.

Since the above existing hydrants are between 75 and 150 metres from the proposed building, these hydrants can be expected to provide contributions of 3,800 L/min to the required fire flow for a total combined flow of 11,400 L/min. As previously indicated, the required fire flow is 90 L/sec or 5,400 L/min. The existing hydrants are considered to be sufficient to meet the required fire flow at the site.

Table 4.2 – Summary of Fire Hydrants

Building	Fire Flow Demand (L/min)	Fire Hydrant(s) within 75m	Fire Hydrant(s) within 150 m	Combined Fire Flow (L/min)
Residential Rowhouse	5,400 L/min	0	3	11,400 L/min

4.4 Proposed Service

The City of Ottawa Design Guidelines – Water Distribution as amended by technical bulletin ISDTB-2014-02 indicates that if possible water distribution systems are to be designed to provide residual pressures of 345 to 552 kPa in all occupied areas outside of the public right-of-way.

In accordance with MOE Guidelines, the distribution system shall be sized so that system pressures during the maximum hourly demand flows are no less than 276 kPa (40 psi) under normal operating conditions.

The largest proposed building is a 3 storey residential building with a ground floor elevation of 87.55 metres. The existing ground surface elevation adjacent the site at Trailsedge Way is 85.15 metres. Assuming a height of 3 metres per floor, the fourth floor fixtures will have a maximum elevation of about 94.5 metres.

The pressure loss between the watermain and the first floor and the pressure loss between the watermain and the fourth floor were calculated using Bernoulli's Equation in combination with the Darcy-Weisback Equation and the Colebrook Equations.

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

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

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

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

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

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

An excel spreadsheet was utilized to facilitate the calculations and is included in Appendix C.

Using the above minimum HGL, a 50 mm service diameter would result in a residual pressure during maximum hourly demand on the ground floor of about 372 kPa. Due to the height of the proposed building a hydraulic grade line of 126.1 results in residual pressure on the top floor of the proposed building of about 312 kPa using a 50 mm diameter service and about 314 kPa using a 100 mm diameter service during maximum hourly demand. The maximum pressure which will occur on the first floor will be at Max HGL and average daily flow and corresponds to 416 kPa. The minimum pressure during fire flow conditions on the ground floor will be 307 kPa.

Alternatively - Neglecting Minor Losses:

$$HGL = \frac{P}{\gamma} + Z$$

$$P = (HGL - Z) \times \gamma$$

ɣ = 9.79 KN/m^3 (unit weight of water)

P = Pressure (KPa) at the Street Z = 84.9

- Minimum pressure P = (126.5 – 84.9) x γ = 407 KPa

P = Pressure (KPa) at First Floor Z = 88.15

- Minimum pressure P = (126.5 – 88.15) x γ = 375 KPa

P = Pressure (KPa) at Third Floor Z = 94.35

- Minimum pressure P = (126.5 – 94.5) x γ = 313 KPa

P = Pressure (KPa) at First Floor Z = 88.15

- Maximum pressure P = (130.6 – 87.55) x γ = 421 KPa

Neglecting minor and frictional pipe losses in the lateral, the maximum pressure at the ground floor water meter is below 552 KPa Neglecting minor and frictional pipe losses in the lateral, the minimum pressure at the third floor is above 276 KPa.

The proposed buildings will not be equipped with sprinklers.

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 #190867-ECP Erosion 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.

Filter socks should be installed across existing storm manhole and catch basin lids. As well, filter socks should be installed across the proposed catch basin lids immediately after the catch basins are placed. The filter socks should only be removed once the asphaltic concrete is installed and the site is cleaned.

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

This report addresses the adequacy of the existing municipal storm and sanitary sewer system and watermains to service the proposed development of two rowhouse buildings at 6173 Renaud Road. Based on the analysis provided in this report, the conclusions are as follows:

SWM for the proposed development will be achieved by restricting the 100 year post development flow to less than 85L/s/ha or 29.27 L/s for the entire site.

The peak sewage flow rate from the proposed development will be 1.0 L/sec. The existing municipal sanitary sewer will have adequate capacity to accommodate the minimal increase in peak flow. The City has not identified any capacity issues in the existing sanitary sewer system and the calculations based on the Master Servicing Study indicate sufficient capacity.

The existing municipal watermain along Trailsedge Way will have adequate capacity to service the proposed development. There are sufficient hydrants in close proximity to the site to meet the fire demands for the site.

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.

____
Steven deWit, P.Eng.

Appendix A: Storm Design Information

- Sheet 1 – Pre-Development Runoff and Allowable Release Rate Calculations
- Sheet 2 – Available Storage and Discharge Rate Calculation -CA1
- Sheet 3 – Available Storage and Discharge Rate Calculation-CA2
- Figure 1 – CA1 – Stage Storage Curve
- Sheet 4 – Storm Sewer Design Sheet
- Visual OTTHYMO Detailed Output File

APPENDIX A: STORMWATER MANAGEMENT MODEL

SHEET 1 - PRE-DEVELPOPMENT RUNFF AND ALLOWABLE RELEASE RATE CALCULATIONS

Client: Teak Developments
Job No.: 190867
Location: 6173 Renaud Road
Date: April 13, 2022

Pre Dev run-off Coefficient "C" - PRE-CA1

Area	Surface	Ha	5yr C	C_{avg}
Total	Gravel	0.0000	0.70	**0.35**
0.2155	Building	0.0331	0.90	
	Driveway	0.0000	0.90	
	Landscaping	0.1824	0.25	
	Offsite Areas	0.0000	0.25	

Pre Dev run-off Coefficient "C" - PRE-CA2

Area	Surface	Ha	5yr C	100yr C
Total	Gravel	0.0000	0.70	0.88
0.1289	Building	0.0108	0.90	1.00
	Driveway	0.0181	0.90	1.00
	Landscaping	0.1000	0.25	0.31
	Offsite Areas	0.0000	0.25	0.31
		C_{avg}	**0.40**	**0.46**

PRE DEVELOPMENT FLOW (PRE-CA1)

5 Year Event

Pre Dev.	C	Intensity	Area
2 Year	0.35	90.63	0.216
2.78CIA= 19.00			
	19.0 L/s		

**Use a 13 minute time of concentration for 5 year

Total Pre-Dev. Runoff Rate 5 year Event: **19.0 L/s**

100 Year Event

Maximum Allowable Post-Development Runoff Rate
85 L/s/ha = 85 * 0.216
= **18.4 L/s**

Runoff Rate Accounted For by Stantec
Q = 2.78CIA= 22.6 L/s
C = 0.52
A = 0.1722
I = 90.6298

Total Allowable Runoff Rate 100 year Event:
= **18.4 L/s**

Alternatively:

Pre Dev Time of Concentration "t_c" — Airport Formula

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

C = Runoff Coefficient	0.35
lc = length of flow path	51
Elevation Change	1.3
S = Slope of flow path	2.5

t_c = **12.82**

Total t_c **13** *min*

PRE DEVELOPMENT FLOW (PRE-CA2)

5 Year Event

Pre Dev.	C	Intensity	Area
2 Year	0.40	90.63	0.129
2.78CIA= 12.99			
		13.0 L/s	

Total Pre-Dev. Runoff Rate 5 year Event: **13.0 L/s**

PRE DEVELOPMENT FLOW

100 Year Event

Pre Dev.	C	Intensity	Area
2 Year	0.46	155.11	0.129
2.78CIA= 25.57			
		25.6 L/s	

Maximum Allowable Post-Development Runoff Rate
85 L/s/ha = 85 * 0.129
= **11.0 L/s**

APPENDIX A: STORMWATER MANAGEMENT MODEL

Sheet 2 - AVAILABLE STORAGE AND DISCHARGE RATE CALCULATION - CATCHMENT AREA CA1

Client: Teak Developments
Job No.: 190867
Location: 6173 Renaud Road
Date: April 13, 2022

Inlet control Device		
Model	75SVHV-1	
Invert Elevation	83.3	m
HGL @ Design Head	86.00	m
Design Head	2.70	m
Dischage	8.0	L/s

Weir Information	
Width	6.7
Coeff, Cd:	0.85
Weir Invert	86.05

							ICD Flow		Weir Flow			
Stage, WSE Elev (m)	Comments	Layer Thickness (m)	Top Layer Area (m²)	Bottom Layer Area (m²)	Layer Volume (m^3)	Quantity Storage (m3)	Head* (m)	Orifice Flow (m^3/sec)	Head* (m)	Weir Flow (m^3/sec)	Combined Outflow (L/sec)	Quantity Storage m3)
86.10		0.050	298.0	298.0	14.9	56.1	2.800	0.0082	0.050	0.1880	196.2	56.1
86.05		0.050	298.0	199.0	12.3	41.2	2.750	0.0081	0.000	0.0000	8.1	41.2
86.00		0.050	199.0	144.0	8.5	28.8	2.700	0.0081	0.000	0.0000	8.1	28.8
85.95		0.050	144.0	83.0	5.6	20.3	2.650	0.0080	0.000	0.0000	8.0	20.3
85.90		0.050	83.0	40.0	3.0	14.7	2.600	0.0079	0.000	0.0000	7.9	14.7
85.85		0.100	40.0	12.5	2.5	11.7	2.550	0.0077	0.000	0.0000	7.7	11.7
85.75	Catchbasin Grate Elevation	1.150	0.6	0.6	0.7	9.2	2.450	0.0076	0.000	0.0000	7.6	9.2
84.60	Top of Clear Stone	0.200	20.0	20.0	1.5	8.5	1.300	0.0055	0.000	0.0000	5.5	8.5
84.40		0.200	20.0	20.0	1.5	7.0	1.100	0.0050	0.000	0.0000	5.0	7.0
84.20		0.200	20.0	20.0	1.5	5.4	0.900	0.0046	0.000	0.0000	4.6	5.4
84.00		0.200	20.0	20.0	1.5	3.9	0.700	0.0041	0.000	0.0000	4.1	3.9
83.80		0.300	20.0	20.0	2.3	2.4	0.500	0.0035	0.000	0.0000	3.5	2.4
83.50	Bottom of Clearstone	0.200	0.6	0.6	0.1	0.1	0.200	0.0010	0.000	0.0000	1.0	0.1
83.30		0.000	0.6	0.0	0.0	0.0	0.000	0.0000	0.000	0.0000	0.0	0.0

.

Storage Provided in Clear Stone

Clear Stone Storage Dimensions	Length (m)	5	Width (m)	4
Void Ratio	0.35			

Orifice FLOW

$Q_{ORIFICE} = C\,A\,(2\,g\,H)^{0.5}$

where:

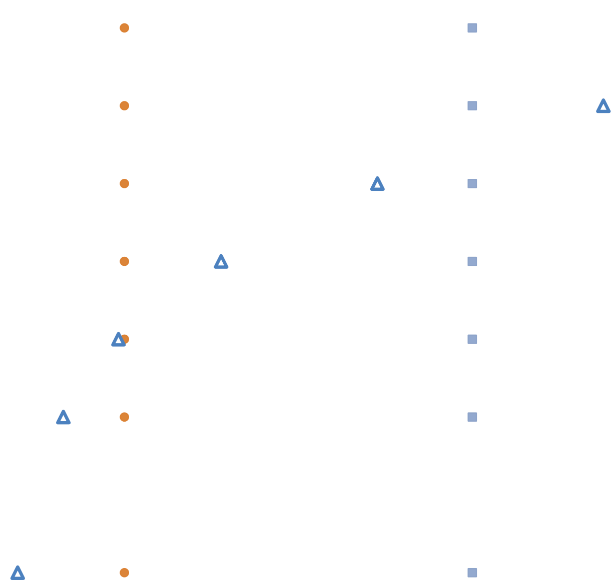
C = Discharge Coefficient

$Q_{ORIFICE}$ = Orifice Flow (m^3/s)

A = Orifice Area (m^2)

g = Accel due to Gravity (9.81 m/s^2)

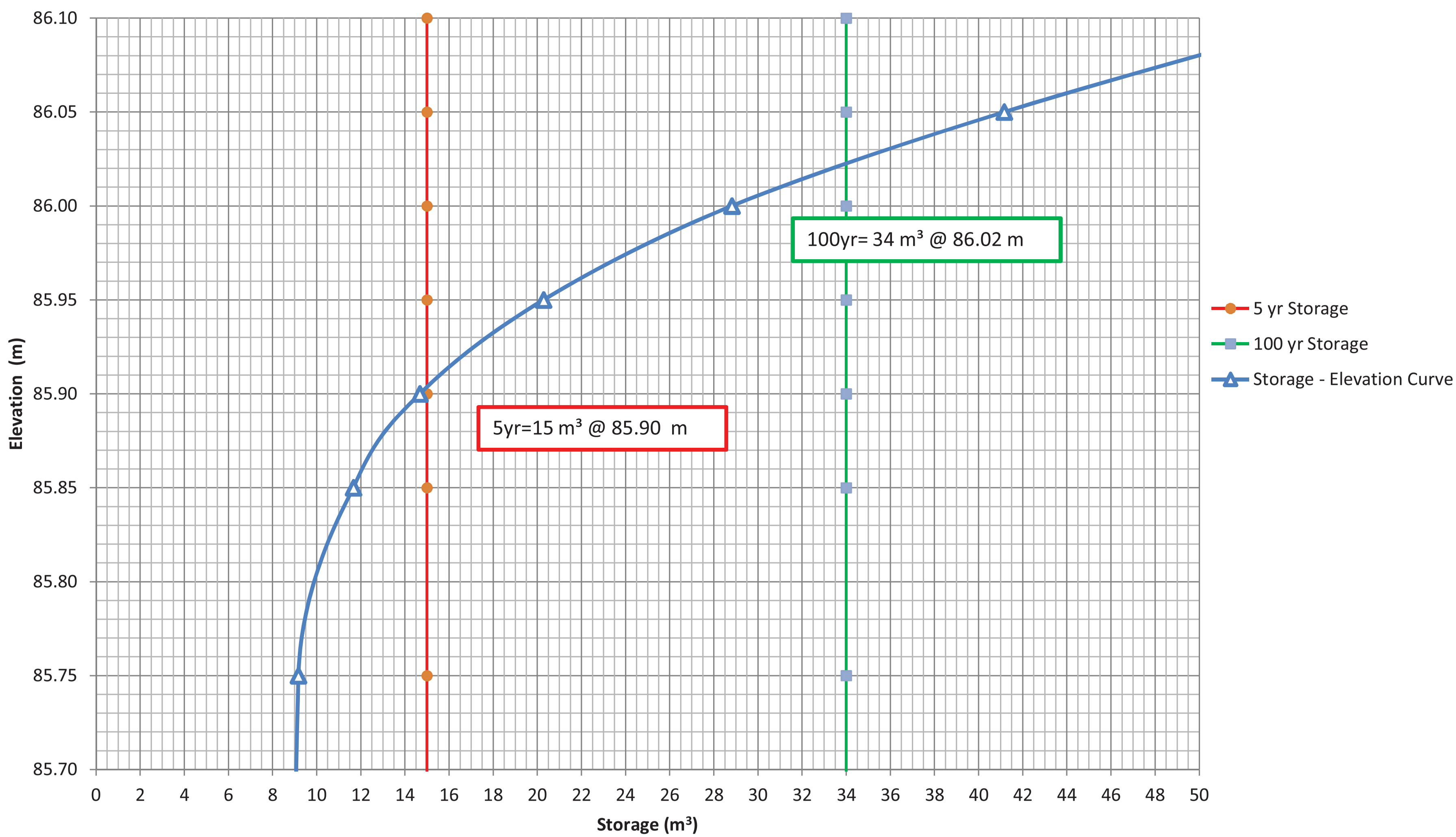
H = Head above centre of orifice (m)



Client: Teak Developments
Job No.: 190867
Location: 6173 Renaud Road, Ottawa, ON
Date: March 29, 2021

APPENDIX A: FIGURE 1

CA1 - Stage - Storage Curve



APPENDIX A: STORMWATER MANAGEMENT MODEL

Sheet 3 - AVAILABLE STORAGE AND DISCHARGE RATE CALCULATION - CATCHMENT AREA CA2

Client: **Teak Developments**
Job No.: **190867**
Location: **6173 Renaud Road**
Date: **April 13, 2022**

Storage Volume Required	5 year	L/s	Allowable Release	5 year	15.4	L/s
	100 year	L/s	Rate	100 year	11.4	L/s

Storage Provided in Storage Tanks

Tank Type	Brentwood Tanks	ST - 36		ST - 36
Tank Dimentions	Height	0.914	Total Volume	0.38
	Length	0.914	Storage Volume	0.37
	Width	0.457	Percent Voids	0.97

Proposed Tank Configuration	6 Rows Width by		21 Rows Length		
ST - 36	6 x 0.457	2.742	21 x 0.914	19.19	
Number of Tank Modules	126				60

Inlet Control Device =	Hydrovex 100SVHV-2	Min Grade @ Tanks	85.15 m
Invert of Outlet Pipe / ICD =	81.85 m	Bottom of Tank	83.59
HGL @ Design Head	85.25 m		
Design Head	2.75 m		
Discharge	17 L/s		

Elevation	Tank Depth	Layer Thickness	Layer Area	Layer Volume	Layer Thickness	Layer Area	Layer Volume	Cum. Volume	Head on ICD	Release Rate
m	m	m	m^2	m^3	m	m^2	m^3	m^3	m	L/s
			Surface							
85.25	0.10	0.05	89.0	3.06				62.3	3.4	13.0
85.2	0.05	0.05	37.0	0.73				59.2	3.35	12.9
85.15	0.00	0	1.0	0.00				58.5	3.3	12.7
		Brentwood Tanks				Clear Stone				
84.7	1.3	0.1	0.00	0.00	0.1	59.30	2.08	57.9	2.85	11.9
84.6	1.2	0.1	0.00	0.00	0.1	59.30	2.08	55.8	2.75	11.8
84.5	1.1	0.1	0.00	0.00	0.1	59.30	2.08	53.7	2.65	11.7
84.4	1	0.09	0.00	0.00	0.09	59.30	1.87	51.6	2.55	11.6
84.31	0.91	0.01	52.63	0.51	0.01	9.87	0.03	49.8	2.46	11.5
84.3	0.9	0.05	52.63	2.56	0.05	9.87	0.17	49.2	2.45	11.5
84.25	0.85	0.05	52.63	2.56	0.05	9.87	0.17	46.5	2.4	11.4
84.2	0.8	0.05	52.63	2.56	0.05	9.87	0.17	43.8	2.35	11.3
84.15	0.75	0.05	52.63	2.56	0.05	9.87	0.17	41.0	2.3	11.2
84.1	0.7	0.05	52.63	2.56	0.05	9.87	0.17	38.3	2.25	11.1
84.05	0.65	0.05	52.63	2.56	0.05	9.87	0.17	35.6	2.2	11.0
84	0.6	0.05	52.63	2.56	0.05	9.87	0.17	32.8	2.15	10.8
83.95	0.55	0.05	52.63	2.56	0.05	9.87	0.17	30.1	2.1	10.7
83.9	0.5	0.05	52.63	2.56	0.05	9.87	0.17	27.3	2.05	10.6
83.85	0.45	0.05	52.63	2.56	0.05	9.87	0.17	24.6	2	10.5
83.8	0.4	0.05	52.63	2.56	0.05	9.87	0.17	21.9	1.95	10.4
83.75	0.35	0.05	52.63	2.56	0.05	9.87	0.17	19.1	1.9	10.3
83.7	0.3	0.05	52.63	2.56	0.05	9.87	0.17	16.4	1.85	10.1
83.65	0.25	0.05	52.63	2.56	0.05	9.87	0.17	13.7	1.8	10.0
83.6	0.2	0.05	52.63	2.56	0.05	9.87	0.17	10.9	1.75	9.8
83.55	0.15	0.05	52.63	2.56	0.05	9.87	0.17	8.2	1.7	9.7
83.5	0.1	0.05	52.63	2.56	0.05	9.87	0.17	5.5	1.65	9.6
83.45	0.05	0.05	52.63	2.56	0.05	9.87	0.17	2.7	1.6	9.4
83.4	0	0	52.63	0.00	0	9.87	0.00	0.0	1.55	9.3

APPENDIX A: STORMWATER MANAGEMENT MODEL
Sheet 4 - Storm Sewer Design Sheet

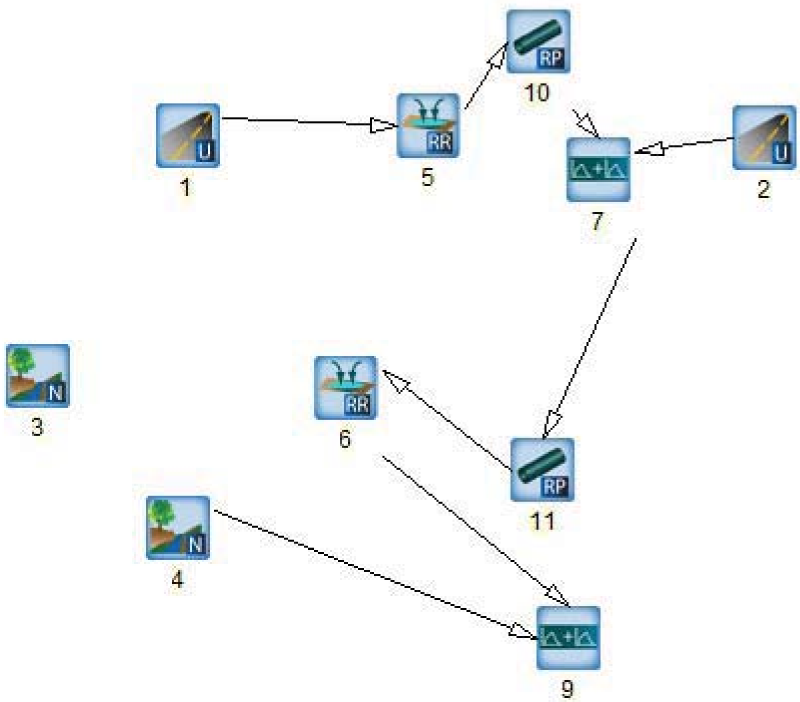
Client: Teak Developments
Job No.: 190867
Location: 6173 Renaud Road
Date: March 11, 2022

Storm Sewer Design Sheet (5-yr storm)

LOCATION									TIME	RAINFALL	PEAK
FROM	TO	Total Area (ha)	C 0.25	C 0.50	C 0.90	Actual R ('C')	INDIV 2.78 AR	ACCUM 2.78 AR	OF CONC.	INTENSITY I	FLOW Q (l/s)
CA1	CA2	0.107	0.0288	0.0000	0.078	0.72	0.21	0.21	10.00	104.19	22.37
CA2	Trailsedge	0.140	0.075	0.000	0.065	0.55	0.21	0.43	10.00	104.19	44.76

PROPOSED SEWER									Controlled /Uncontrolled	Controlled Flow	ICD
TYPE OF PIPE	PIPE SIZE (mm)	PIPE SLOPE (%)	LENGTH (m)	CAPACITY (l/s)	FULL FLOW VELOCITY (m/s)	TIME OF FLOW (min.)	EXCESS CAPACITY (l/s)	Q/Qfull		(L/s)	
PVC	250.00	2.70	17.0	97.81	1.99	0.14	75.45	0.23	Controlled	8	Hydrovex 75 SVHV 1
PVC	250.00	2.10	70.5	86.26	1.76	0.67	41.51	0.52	Controlled	11	Hydrovex 100 SVHV 2

Rainfall Intensity = $998.071/(T+6.053)^{-0.814}$ T= time in minutes
(City of Ottawa, 5 year storm)



Schematic Summary Table

Hydrograp h No.	Model Type	Item Represented	Comment
1	STANDHYD	Catchment Area CA1	Controlled Area Catchment Including majority of building area and Parking between buildings
2	STANDHYD	Catchment Area CA2	Remaining Controlled Area of the Site
3	NASHYD	Catchment Area UA2	Uncontrolled Catchment Area which Outlets to Renaud Road
4	NASHYD	Catchment Area UA1	Uncontrolled Catchment Area which Outlets to Trailsedge Way
5	Route Reservoir	Storage in Parking Area in CA1	Stage storage and outlet control for Parking Area and subsurface storage in CA1
6	Route Reservoir	Storage in Parking Area in CA2	Stage Storage and outlet control for Parking Area and subsurface storage in CA2
10, 11	Route Pipe	Storm Pipe	Represent the storm pipe between the Storage in CA1 and CA2 and between the Storage in CA2.
7,9	ADD-HYD	Add Hydrograph	Link used to add two hydrographs in the routing

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================================================================================

       V    V   I    SSSSS  U   U    A    L
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

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

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


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  ****************************
  ** SIMULATION NUMBER:   1 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 998.071
| Ptotal= 49.04 mm |                          B=   6.053
--------------------                          C=    .814
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  6.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33

                 TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                  .17    1.78 |  1.67    9.61 |  3.17    4.87 |  4.67    2.31
                  .33    1.94 |  1.83   24.17 |  3.33    4.30 |  4.83    2.19
                  .50    2.13 |  2.00  104.19 |  3.50    3.86 |  5.00    2.08
                  .67    2.37 |  2.17   32.04 |  3.67    3.51 |  5.17    1.99
                  .83    2.68 |  2.33   16.34 |  3.83    3.22 |  5.33    1.90
                 1.00    3.10 |  2.50   10.96 |  4.00    2.98 |  5.50    1.82
                 1.17    3.68 |  2.67    8.29 |  4.17    2.77 |  5.67    1.75
                 1.33    4.58 |  2.83    6.69 |  4.33    2.60 |  5.83    1.68
                 1.50    6.15 |  3.00    5.63 |  4.50    2.44 |  6.00    1.62


-------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0003) |   Area    (ha)=    .06   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   7.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.
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```
                  ---- TRANSFORMED HYETOGRAPH ----
     TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
      hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
     .083    1.78 | 1.583    9.61 | 3.083    4.87 |  4.58    2.31
     .167    1.78 | 1.667    9.61 | 3.167    4.87 |  4.67    2.31
     .250    1.94 | 1.750   24.17 | 3.250    4.30 |  4.75    2.19
     .333    1.94 | 1.833   24.17 | 3.333    4.30 |  4.83    2.19
     .417    2.13 | 1.917  104.19 | 3.417    3.86 |  4.92    2.08
     .500    2.13 | 2.000  104.19 | 3.500    3.86 |  5.00    2.08
     .583    2.37 | 2.083   32.04 | 3.583    3.51 |  5.08    1.99
     .667    2.37 | 2.167   32.04 | 3.667    3.51 |  5.17    1.99
     .750    2.68 | 2.250   16.34 | 3.750    3.22 |  5.25    1.90
     .833    2.68 | 2.333   16.34 | 3.833    3.22 |  5.33    1.90
     .917    3.10 | 2.417   10.96 | 3.917    2.98 |  5.42    1.82
    1.000    3.10 | 2.500   10.96 | 4.000    2.98 |  5.50    1.82
    1.083    3.68 | 2.583    8.29 | 4.083    2.77 |  5.58    1.75
    1.167    3.68 | 2.667    8.29 | 4.167    2.77 |  5.67    1.75
    1.250    4.58 | 2.750    6.69 | 4.250    2.60 |  5.75    1.68
    1.333    4.58 | 2.833    6.69 | 4.333    2.60 |  5.83    1.68
    1.417    6.15 | 2.917    5.63 | 4.417    2.44 |  5.92    1.62
    1.500    6.15 | 3.000    5.63 | 4.500    2.44 |  6.00    1.62

     Unit Hyd Qpeak  (cms)=    .012

     PEAK FLOW       (cms)=    .004 (i)
     TIME TO PEAK    (hrs)=   2.167
     RUNOFF VOLUME    (mm)=  20.262
     TOTAL RAINFALL   (mm)=  49.038
     RUNOFF COEFFICIENT   =    .413

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| CALIB              |
| STANDHYD  (0001)   |   Area    (ha)=    .11
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  73.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=        .08          .03
     Dep. Storage      (mm)=       1.57         4.67
     Average Slope      (%)=       1.00         3.00
     Length             (m)=      26.70        26.00
     Mannings n            =       .013         .250

     Max.Eff.Inten.(mm/hr)=     104.19       101.71
                over (min)       5.00         5.00
     Storage Coeff.  (min)=       1.14 (ii)    4.59 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .23
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .02          .01          .027 (iii)
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     TIME TO PEAK    (hrs)=       2.00          2.00           2.00
     RUNOFF VOLUME    (mm)=      47.47         15.65          34.74
     TOTAL RAINFALL   (mm)=      49.04         49.04          49.04
     RUNOFF COEFFICIENT   =        .97           .32            .71

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20           K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0002) |   Area    (ha)=    .14
|ID= 1 DT= 5.0 min |   Total Imp(%)=  46.00   Dir. Conn.(%)=  42.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .06           .08
     Dep. Storage     (mm)=       1.57          4.67
     Average Slope     (%)=       1.00          3.00
     Length            (m)=      30.60         26.00
     Mannings n           =       .013          .250

     Max.Eff.Inten.(mm/hr)=     104.19         50.66
                over (min)       5.00         10.00
     Storage Coeff.  (min)=       1.23 (ii)     7.57 (ii)
     Unit Hyd. Tpeak (min)=       5.00         10.00
     Unit Hyd. peak  (cms)=        .33           .13
                                                               *TOTALS*
     PEAK FLOW       (cms)=        .02           .01            .022 (iii)
     TIME TO PEAK    (hrs)=       2.00          2.08           2.00
     RUNOFF VOLUME    (mm)=      47.47          9.59          25.50
     TOTAL RAINFALL   (mm)=      49.04         49.04          49.04
     RUNOFF COEFFICIENT   =        .97           .20            .52

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20           K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0004) |   Area    (ha)=    .04   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=   7.10   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17
```

```
     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .003 (i)
     TIME TO PEAK    (hrs)=   2.167
     RUNOFF VOLUME    (mm)=  19.392
     TOTAL RAINFALL   (mm)=  49.038
     RUNOFF COEFFICIENT   =    .395

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0005) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .0076       .0009
                           .0010      .0001   |    .0079       .0015
                           .0035      .0002   |    .0081       .0029
                           .0041      .0004   |    .0082       .0041
                           .0046      .0005   |    .1960       .0056
                           .0055      .0009   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0001)       .107       .027        2.00       34.74
     OUTFLOW: ID= 1 (0005)       .107       .008        2.17       34.58

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 29.42
                   TIME SHIFT OF PEAK FLOW         (min)= 10.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0014

-------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0010)|     PIPE Number     =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 250.00
| DT=  5.0 min     |     Length       (m)=  19.50
--------------------     Slope      (m/m)=   .030
                         Manning n       =   .013

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)      (cms)        (m/s)        min
        .01     .193E-01        .0           .56          .58
        .03     .537E-01        .0           .87          .37
        .04     .970E-01        .0          1.12          .29
        .05     .147E+00        .0          1.33          .24
        .07     .201E+00        .0          1.51          .21
        .08     .259E+00        .0          1.68          .19
        .09     .320E+00        .0          1.82          .18
        .11     .383E+00        .0          1.94          .17
        .12     .447E+00        .0          2.05          .16
        .13     .511E+00        .1          2.14          .15
```

```
      .14     .574E+00        .1           2.22          .15
      .16     .637E+00        .1           2.29          .14
      .17     .698E+00        .1           2.34          .14
      .18     .756E+00        .1           2.37          .14
      .20     .811E+00        .1           2.39          .14
      .21     .860E+00        .1           2.39          .14
      .22     .904E+00        .1           2.36          .14
      .24     .938E+00        .1           2.30          .14
      .25     .957E+00        .1           2.10          .15
                                <---- hydrograph ---->   <-pipe / channel->
                          AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                          (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
     INFLOW : ID= 2 (0005)  .11      .01    2.17  34.58       .05      1.22
     OUTFLOW: ID= 1 (0010)  .11      .01    2.08  34.58       .05      1.23


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

--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0010):     .11    .008     2.08    34.58
        + ID2= 2 (0002):     .14    .022     2.00    25.50
          ==================================================
          ID = 3 (0007):     .25    .029     2.00    29.42

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0011)|      PIPE Number     =   1.00
| IN= 2---> OUT= 1 |      Diameter    (mm)= 150.00
| DT=  5.0 min     |      Length       (m)=  70.50
--------------------      Slope      (m/m)=   .020
                          Manning n       =   .013

     **** WARNING: MINIMUM PIPE SIZE REQUIRED =   168.68   (mm)FOR FREE FLOW.
                   THIS SIZE WAS USED IN THE ROUTING.
                   THE CAPACITY OF THIS PIPE  =      .03  (cms)

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)      (cms)        (m/s)        min
        .01     .318E-01        .0           .35         3.35
        .02     .884E-01        .0           .55         2.15
        .03     .160E+00        .0           .70         1.67
        .04     .241E+00        .0           .84         1.41
        .04     .331E+00        .0           .95         1.23
        .05     .427E+00        .0          1.05         1.12
        .06     .527E+00        .0          1.14         1.03
        .07     .630E+00        .0          1.22          .96
        .08     .735E+00        .0          1.29          .91
        .09     .841E+00        .0          1.35          .87
```

```
        .10     .945E+00        .0          1.40          .84
        .11     .105E+01        .0          1.44          .82
        .12     .115E+01        .0          1.47          .80
        .12     .124E+01        .0          1.49          .79
        .13     .133E+01        .0          1.50          .78
        .14     .142E+01        .0          1.50          .78
        .15     .149E+01        .0          1.49          .79
        .16     .154E+01        .0          1.45          .81
        .17     .158E+01        .0          1.32          .89
                                      <---- hydrograph ---->   <-pipe / channel->
                                 AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                                 (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
     INFLOW : ID= 2 (0007)       .25      .03    2.00  29.42       .14      1.50
     OUTFLOW: ID= 1 (0011)       .25      .03    2.00  29.40       .14      1.50


 -------------------------------------------------------------------------------
 --------------------
 | RESERVOIR (0006)  |
 | IN= 2---> OUT= 1  |
 | DT=  5.0 min      |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
 --------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                            .0000      .0000    |   .0112       .0041
                            .0093      .0001    |   .0115       .0049
                            .0097      .0008    |   .0118       .0056
                            .0103      .0019    |   .0130       .0062
                            .0107      .0030    |   .0000       .0000

                                    AREA      QPEAK        TPEAK        R.V.
                                    (ha)      (cms)        (hrs)        (mm)
     INFLOW : ID= 2 (0011)          .247       .030         2.00        29.40
     OUTFLOW: ID= 1 (0006)          .247       .010         2.50        29.41

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 33.63
                   TIME SHIFT OF PEAK FLOW         (min)= 30.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0018

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

 --------------------
 | ADD HYD   (0009) |
 |   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
 --------------------         (ha)    (cms)    (hrs)     (mm)
           ID1= 1 (0006):       .25    .010     2.50    29.41
         + ID2= 2 (0004):       .04    .003     2.17    19.39
           ==================================================
           ID = 3 (0009):       .29    .013     2.17    27.94

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 -------------------------------------------------------------------------------
   ****************************
   ** SIMULATION NUMBER:   2 **
   ****************************
```

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--------------------
| CHICAGO STORM    |     IDF curve parameters: A= 998.071
| Ptotal= 56.17 mm |                           B=   6.053
--------------------                           C=    .814
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  = 12.00 hrs
                         Storm time step    = 10.00 min
                         Time to peak ratio =   .33
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.17	.94	3.17	3.68	6.17	2.77	9.17	1.30
.33	.98	3.33	4.58	6.33	2.60	9.33	1.27
.50	1.02	3.50	6.15	6.50	2.44	9.50	1.24
.67	1.06	3.67	9.61	6.67	2.31	9.67	1.20
.83	1.11	3.83	24.17	6.83	2.19	9.83	1.17
1.00	1.16	4.00	104.19	7.00	2.08	10.00	1.15
1.17	1.22	4.17	32.04	7.17	1.99	10.17	1.12
1.33	1.28	4.33	16.34	7.33	1.90	10.33	1.10
1.50	1.36	4.50	10.96	7.50	1.82	10.50	1.07
1.67	1.44	4.67	8.29	7.67	1.75	10.67	1.05
1.83	1.54	4.83	6.69	7.83	1.68	10.83	1.03
2.00	1.65	5.00	5.63	8.00	1.62	11.00	1.01
2.17	1.78	5.17	4.87	8.17	1.57	11.17	.99
2.33	1.94	5.33	4.30	8.33	1.51	11.33	.97
2.50	2.13	5.50	3.86	8.50	1.47	11.50	.95
2.67	2.37	5.67	3.51	8.67	1.42	11.67	.93
2.83	2.68	5.83	3.22	8.83	1.38	11.83	.92
3.00	3.10	6.00	2.98	9.00	1.34	12.00	.90

```
-------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0003) |   Area     (ha)=     .06   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=    7.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=     .17

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

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.083	.94	3.083	3.68	6.083	2.77	9.08	1.30
.167	.94	3.167	3.68	6.167	2.77	9.17	1.30
.250	.98	3.250	4.58	6.250	2.60	9.25	1.27
.333	.98	3.333	4.58	6.333	2.60	9.33	1.27
.417	1.02	3.417	6.15	6.417	2.44	9.42	1.24
.500	1.02	3.500	6.15	6.500	2.44	9.50	1.24
.583	1.06	3.583	9.61	6.583	2.31	9.58	1.20
.667	1.06	3.667	9.61	6.667	2.31	9.67	1.20

```
              .750    1.11 | 3.750   24.17 | 6.750    2.19 |  9.75    1.17
              .833    1.11 | 3.833   24.17 | 6.833    2.19 |  9.83    1.17
              .917    1.16 | 3.917  104.19 | 6.917    2.08 |  9.92    1.15
             1.000    1.16 | 4.000  104.19 | 7.000    2.08 | 10.00    1.15
             1.083    1.22 | 4.083   32.04 | 7.083    1.99 | 10.08    1.12
             1.167    1.22 | 4.167   32.04 | 7.167    1.99 | 10.17    1.12
             1.250    1.28 | 4.250   16.34 | 7.250    1.90 | 10.25    1.10
             1.333    1.28 | 4.333   16.34 | 7.333    1.90 | 10.33    1.10
             1.417    1.36 | 4.417   10.96 | 7.417    1.82 | 10.42    1.07
             1.500    1.36 | 4.500   10.96 | 7.500    1.82 | 10.50    1.07
             1.583    1.44 | 4.583    8.29 | 7.583    1.75 | 10.58    1.05
             1.667    1.44 | 4.667    8.29 | 7.667    1.75 | 10.67    1.05
             1.750    1.54 | 4.750    6.69 | 7.750    1.68 | 10.75    1.03
             1.833    1.54 | 4.833    6.69 | 7.833    1.68 | 10.83    1.03
             1.917    1.65 | 4.917    5.63 | 7.917    1.62 | 10.92    1.01
             2.000    1.65 | 5.000    5.63 | 8.000    1.62 | 11.00    1.01
             2.083    1.78 | 5.083    4.87 | 8.083    1.57 | 11.08     .99
             2.167    1.78 | 5.167    4.87 | 8.167    1.57 | 11.17     .99
             2.250    1.94 | 5.250    4.30 | 8.250    1.51 | 11.25     .97
             2.333    1.94 | 5.333    4.30 | 8.333    1.51 | 11.33     .97
             2.417    2.13 | 5.417    3.86 | 8.417    1.47 | 11.42     .95
             2.500    2.13 | 5.500    3.86 | 8.500    1.47 | 11.50     .95
             2.583    2.37 | 5.583    3.51 | 8.583    1.42 | 11.58     .93
             2.667    2.37 | 5.667    3.51 | 8.667    1.42 | 11.67     .93
             2.750    2.68 | 5.750    3.22 | 8.750    1.38 | 11.75     .92
             2.833    2.68 | 5.833    3.22 | 8.833    1.38 | 11.83     .92
             2.917    3.10 | 5.917    2.98 | 8.917    1.34 | 11.92     .90
             3.000    3.10 | 6.000    2.98 | 9.000    1.34 | 12.00     .90

     Unit Hyd Qpeak  (cms)=    .012

     PEAK FLOW       (cms)=    .004 (i)
     TIME TO PEAK    (hrs)=   4.083
     RUNOFF VOLUME    (mm)=  25.624
     TOTAL RAINFALL   (mm)=  56.170
     RUNOFF COEFFICIENT   =    .456

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0001) |   Area    (ha)=    .11
|ID= 1 DT= 5.0 min |   Total Imp(%)=  73.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .08           .03
     Dep. Storage     (mm)=       1.57          4.67
     Average Slope     (%)=       1.00          3.00
     Length            (m)=      26.70         26.00
     Mannings n           =       .013          .250

     Max.Eff.Inten.(mm/hr)=     104.19        111.24
               over (min)         5.00          5.00
```

```
     Storage Coeff.  (min)=       1.14 (ii)    4.59 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .23
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .02          .01          .027 (iii)
     TIME TO PEAK    (hrs)=       4.00         4.00         4.00
     RUNOFF VOLUME    (mm)=      54.60        17.14        39.62
     TOTAL RAINFALL   (mm)=      56.17        56.17        56.17
     RUNOFF COEFFICIENT   =        .97          .31          .71

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 76.20          K  (1/hr)=  4.14
            Fc   (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0002) |   Area    (ha)=    .14
|ID= 1 DT= 5.0 min |   Total Imp(%)=  46.00   Dir. Conn.(%)=  42.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .06          .08
     Dep. Storage     (mm)=       1.57         4.67
     Average Slope     (%)=       1.00         3.00
     Length            (m)=      30.60        26.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     104.19        55.75
                over (min)       5.00        10.00
     Storage Coeff.  (min)=       1.23 (ii)    7.33 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=        .33          .13
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .02          .01          .023 (iii)
     TIME TO PEAK    (hrs)=       4.00         4.08         4.00
     RUNOFF VOLUME    (mm)=      54.60        10.87        29.24
     TOTAL RAINFALL   (mm)=      56.17        56.17        56.17
     RUNOFF COEFFICIENT   =        .97          .19          .52

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 76.20          K  (1/hr)=  4.14
            Fc   (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
```

```
--------------------
| CALIB            |
| NASHYD    (0004) |   Area     (ha)=    .04   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=   7.10   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .003 (i)
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  24.608
     TOTAL RAINFALL   (mm)=  56.170
     RUNOFF COEFFICIENT   =    .438

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0005) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |    .0076      .0009
                          .0010      .0001    |    .0079      .0015
                          .0035      .0002    |    .0081      .0029
                          .0041      .0004    |    .0082      .0041
                          .0046      .0005    |    .1960      .0056
                          .0055      .0009    |    .0000      .0000

                               AREA      QPEAK       TPEAK       R.V.
                               (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0001)     .107       .027        4.00       39.62
     OUTFLOW: ID= 1 (0005)     .107       .008        4.17       39.50

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 29.01
                   TIME SHIFT OF PEAK FLOW         (min)= 10.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0015

--------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0010)|     PIPE Number      =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 250.00
| DT=  5.0 min     |     Length       (m)=  19.50
--------------------     Slope      (m/m)=   .030
                         Manning n        =   .013

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)      (cms)        (m/s)        min
        .01     .193E-01        .0           .56          .58
        .03     .537E-01        .0           .87          .37
        .04     .970E-01        .0          1.12          .29
        .05     .147E+00        .0          1.33          .24
        .07     .201E+00        .0          1.51          .21
```

```
      .08     .259E+00        .0          1.68          .19
      .09     .320E+00        .0          1.82          .18
      .11     .383E+00        .0          1.94          .17
      .12     .447E+00        .0          2.05          .16
      .13     .511E+00        .1          2.14          .15
      .14     .574E+00        .1          2.22          .15
      .16     .637E+00        .1          2.29          .14
      .17     .698E+00        .1          2.34          .14
      .18     .756E+00        .1          2.37          .14
      .20     .811E+00        .1          2.39          .14
      .21     .860E+00        .1          2.39          .14
      .22     .904E+00        .1          2.36          .14
      .24     .938E+00        .1          2.30          .14
      .25     .957E+00        .1          2.10          .15
                                  <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                            (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
     INFLOW : ID= 2 (0005)   .11      .01    4.17  39.50        .05      1.22
     OUTFLOW: ID= 1 (0010)   .11      .01    4.08  39.50        .05      1.23


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

--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |        AREA     QPEAK     TPEAK      R.V.
--------------------        (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0010):     .11     .008      4.08     39.50
        + ID2= 2 (0002):     .14     .023      4.00     29.24
          ==================================================
          ID = 3 (0007):     .25     .030      4.00     33.67

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0011)|     PIPE Number      =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 150.00
| DT=  5.0 min     |     Length       (m)=  70.50
--------------------     Slope      (m/m)=   .020
                         Manning n        =   .013

     **** WARNING: MINIMUM PIPE SIZE REQUIRED =   170.36   (mm)FOR FREE FLOW.
                   THIS SIZE WAS USED IN THE ROUTING.
                   THE CAPACITY OF THIS PIPE  =      .03  (cms)

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME     FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)       (cms)        (m/s)        min
        .01     .324E-01        .0           .35         3.33
        .02     .902E-01        .0           .55         2.14
        .03     .163E+00        .0           .71         1.66
        .04     .246E+00        .0           .84         1.40
        .04     .338E+00        .0           .96         1.23
```

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          .05      .435E+00         .0          1.06         1.11
          .06      .537E+00         .0          1.15         1.02
          .07      .643E+00         .0          1.23          .96
          .08      .750E+00         .0          1.30          .91
          .09      .857E+00         .0          1.36          .87
          .10      .964E+00         .0          1.41          .84
          .11      .107E+01         .0          1.45          .81
          .12      .117E+01         .0          1.48          .79
          .13      .127E+01         .0          1.50          .78
          .13      .136E+01         .0          1.51          .78
          .14      .144E+01         .0          1.51          .78
          .15      .152E+01         .0          1.50          .79
          .16      .157E+01         .0          1.46          .81
          .17      .161E+01         .0          1.33          .89
                                       <---- hydrograph ---->   <-pipe / channel->
                                 AREA     QPEAK    TPEAK   R.V.   MAX DEPTH   MAX VEL
                                 (ha)     (cms)    (hrs)   (mm)      (m)       (m/s)
     INFLOW : ID= 2 (0007)        .25       .03     4.00  33.67       .14       1.51
     OUTFLOW: ID= 1 (0011)        .25       .03     4.00  33.66       .14       1.51


-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |   .0112       .0041
                          .0093      .0001    |   .0115       .0049
                          .0097      .0008    |   .0118       .0056
                          .0103      .0019    |   .0130       .0062
                          .0107      .0030    |   .0000       .0000

                                    AREA      QPEAK       TPEAK       R.V.
                                    (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0011)          .247       .031        4.00       33.66
     OUTFLOW: ID= 1 (0006)          .247       .010        4.50       33.66

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 32.99
                   TIME SHIFT OF PEAK FLOW         (min)= 30.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0019

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--------------------
| ADD HYD   (0009) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):      .25     .010     4.50    33.66
        + ID2= 2 (0004):      .04     .003     4.17    24.61
          ==================================================
          ID = 3 (0009):      .29     .013     4.17    32.33

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
```

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  ****************************
  ** SIMULATION NUMBER:   3 **
  ****************************

--------------------
| CHICAGO STORM    |     IDF curve parameters: A=1735.071
| Ptotal= 82.29 mm |                           B=   6.014
--------------------                           C=    .820
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  6.00 hrs
                         Storm time step    = 10.00 min
                         Time to peak ratio =   .33

                  TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                   .17    2.90 |  1.67   15.96 |  3.17    8.02 |  4.67    3.77
                   .33    3.16 |  1.83   40.64 |  3.33    7.08 |  4.83    3.57
                   .50    3.48 |  2.00  178.50 |  3.50    6.34 |  5.00    3.40
                   .67    3.87 |  2.17   54.03 |  3.67    5.76 |  5.17    3.24
                   .83    4.39 |  2.33   27.31 |  3.83    5.28 |  5.33    3.09
                  1.00    5.07 |  2.50   18.23 |  4.00    4.88 |  5.50    2.97
                  1.17    6.04 |  2.67   13.73 |  4.17    4.54 |  5.67    2.85
                  1.33    7.54 |  2.83   11.05 |  4.33    4.24 |  5.83    2.74
                  1.50   10.16 |  3.00    9.28 |  4.50    3.99 |  6.00    2.64


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--------------------
| CALIB            |
| NASHYD    (0003) |    Area     (ha)=     .06   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |    Ia       (mm)=    7.00   # of Linear Res.(N)= 3.00
--------------------    U.H. Tp(hrs)=     .17

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


                                ---- TRANSFORMED HYETOGRAPH ----
                  TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                  .083    2.90 | 1.583   15.96 | 3.083    8.02 |  4.58    3.77
                  .167    2.90 | 1.667   15.96 | 3.167    8.02 |  4.67    3.77
                  .250    3.16 | 1.750   40.64 | 3.250    7.08 |  4.75    3.57
                  .333    3.16 | 1.833   40.64 | 3.333    7.08 |  4.83    3.57
                  .417    3.48 | 1.917  178.50 | 3.417    6.34 |  4.92    3.40
                  .500    3.48 | 2.000  178.50 | 3.500    6.34 |  5.00    3.40
                  .583    3.87 | 2.083   54.03 | 3.583    5.76 |  5.08    3.24
                  .667    3.87 | 2.167   54.03 | 3.667    5.76 |  5.17    3.24
                  .750    4.39 | 2.250   27.31 | 3.750    5.28 |  5.25    3.09
                  .833    4.39 | 2.333   27.31 | 3.833    5.28 |  5.33    3.09
                  .917    5.07 | 2.417   18.23 | 3.917    4.88 |  5.42    2.97
                 1.000    5.07 | 2.500   18.23 | 4.000    4.88 |  5.50    2.97
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                1.083    6.04 | 2.583   13.73 | 4.083    4.54 |  5.58    2.85
                1.167    6.04 | 2.667   13.73 | 4.167    4.54 |  5.67    2.85
                1.250    7.54 | 2.750   11.05 | 4.250    4.24 |  5.75    2.74
                1.333    7.54 | 2.833   11.05 | 4.333    4.24 |  5.83    2.74
                1.417   10.16 | 2.917    9.28 | 4.417    3.99 |  5.92    2.64
                1.500   10.16 | 3.000    9.28 | 4.500    3.99 |  6.00    2.64

     Unit Hyd Qpeak  (cms)=    .012

     PEAK FLOW       (cms)=    .009 (i)
     TIME TO PEAK    (hrs)=   2.083
     RUNOFF VOLUME    (mm)=  47.015
     TOTAL RAINFALL   (mm)=  82.290
     RUNOFF COEFFICIENT   =    .571

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| CALIB              |
| STANDHYD  (0001) |   Area    (ha)=    .11
|ID= 1 DT= 5.0 min |   Total Imp(%)=  73.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .08          .03
     Dep. Storage     (mm)=       1.57         4.67
     Average Slope     (%)=       1.00         3.00
     Length            (m)=      26.70        26.00
     Mannings n          =        .013         .250

     Max.Eff.Inten.(mm/hr)=     178.50       245.22
                over (min)       5.00         5.00
     Storage Coeff.  (min)=        .92 (ii)     3.70 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .25
                                                            *TOTALS*
     PEAK FLOW       (cms)=        .03          .02           .050 (iii)
     TIME TO PEAK    (hrs)=       2.00         2.00           2.00
     RUNOFF VOLUME    (mm)=      80.72        41.06          64.86
     TOTAL RAINFALL   (mm)=      82.29        82.29          82.29
     RUNOFF COEFFICIENT   =        .98          .50            .79

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20          K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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| CALIB              |
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| STANDHYD  (0002) |   Area    (ha)=    .14
|ID= 1 DT= 5.0 min |   Total Imp(%)=  46.00   Dir. Conn.(%)=  42.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       .06          .08
     Dep. Storage     (mm)=      1.57         4.67
     Average Slope     (%)=      1.00         3.00
     Length            (m)=     30.60        26.00
     Mannings n           =      .013         .250

     Max.Eff.Inten.(mm/hr)=    178.50       151.07
                over (min)       5.00        10.00
     Storage Coeff.  (min)=      1.00 (ii)    5.23 (ii)
     Unit Hyd. Tpeak (min)=      5.00        10.00
     Unit Hyd. peak  (cms)=       .34          .16
                                                             *TOTALS*
     PEAK FLOW       (cms)=       .03          .02              .051 (iii)
     TIME TO PEAK    (hrs)=      2.00         2.08              2.00
     RUNOFF VOLUME    (mm)=     80.72        32.67             52.85
     TOTAL RAINFALL   (mm)=     82.29        82.29             82.29
     RUNOFF COEFFICIENT   =       .98          .40               .64

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20           K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0004) |   Area    (ha)=    .04   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   7.10   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .007 (i)
     TIME TO PEAK    (hrs)=   2.083
     RUNOFF VOLUME    (mm)=  45.579
     TOTAL RAINFALL   (mm)=  82.290
     RUNOFF COEFFICIENT   =    .554

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0005) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
```

```
                                .0000       .0000   |    .0076       .0009
                                .0010       .0001   |    .0079       .0015
                                .0035       .0002   |    .0081       .0029
                                .0041       .0004   |    .0082       .0041
                                .0046       .0005   |    .1960       .0056
                                .0055       .0009   |    .0000       .0000

                                      AREA      QPEAK       TPEAK       R.V.
                                      (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0001)            .107       .050        2.00       64.86
     OUTFLOW: ID= 1 (0005)            .107       .008        2.25       64.67

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 16.24
                   TIME SHIFT OF PEAK FLOW         (min)= 15.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0034

--------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0010)|     PIPE Number     =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 250.00
| DT=  5.0 min     |     Length       (m)=  19.50
--------------------     Slope      (m/m)=   .030
                         Manning n       =   .013

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)      (cms)        (m/s)        min
        .01     .193E-01        .0           .56          .58
        .03     .537E-01        .0           .87          .37
        .04     .970E-01        .0          1.12          .29
        .05     .147E+00        .0          1.33          .24
        .07     .201E+00        .0          1.51          .21
        .08     .259E+00        .0          1.68          .19
        .09     .320E+00        .0          1.82          .18
        .11     .383E+00        .0          1.94          .17
        .12     .447E+00        .0          2.05          .16
        .13     .511E+00        .1          2.14          .15
        .14     .574E+00        .1          2.22          .15
        .16     .637E+00        .1          2.29          .14
        .17     .698E+00        .1          2.34          .14
        .18     .756E+00        .1          2.37          .14
        .20     .811E+00        .1          2.39          .14
        .21     .860E+00        .1          2.39          .14
        .22     .904E+00        .1          2.36          .14
        .24     .938E+00        .1          2.30          .14
        .25     .957E+00        .1          2.10          .15
                                   <---- hydrograph ---->   <-pipe / channel->
                                AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                                (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
     INFLOW : ID= 2 (0005)       .11      .01    2.25  64.67       .05      1.23
     OUTFLOW: ID= 1 (0010)       .11      .01    2.17  64.66       .05      1.24

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--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0010):       .11    .008     2.17    64.66
        + ID2= 2 (0002):       .14    .051     2.00    52.85
          ==================================================
          ID = 3 (0007):       .25    .059     2.00    57.95

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0011)|     PIPE Number     =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 150.00
| DT=  5.0 min     |     Length       (m)=  70.50
--------------------     Slope      (m/m)=   .020
                         Manning n       =   .013

     **** WARNING: MINIMUM PIPE SIZE REQUIRED =   218.95   (mm)FOR FREE FLOW.
                   THIS SIZE WAS USED IN THE ROUTING.
                   THE CAPACITY OF THIS PIPE  =      .06  (cms)

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)      (cms)       (m/s)        min
        .01     .535E-01        .0          .42         2.82
        .02     .149E+00        .0          .65         1.81
        .03     .269E+00        .0          .84         1.40
        .05     .407E+00        .0          .99         1.18
        .06     .558E+00        .0         1.13         1.04
        .07     .719E+00        .0         1.25          .94
        .08     .888E+00        .0         1.36          .86
        .09     .106E+01        .0         1.45          .81
        .10     .124E+01        .0         1.53          .77
        .12     .142E+01        .0         1.60          .73
        .13     .159E+01        .0         1.66          .71
        .14     .177E+01        .0         1.71          .69
        .15     .194E+01        .0         1.75          .67
        .16     .210E+01        .1         1.77          .66
        .17     .225E+01        .1         1.79          .66
        .18     .239E+01        .1         1.79          .66
        .20     .251E+01        .1         1.77          .66
        .21     .260E+01        .1         1.72          .68
        .22     .265E+01        .1         1.57          .75
                                     <---- hydrograph ---->   <-pipe / channel->
                               AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                               (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
     INFLOW : ID= 2 (0007)      .25      .06    2.00  57.95       .18      1.79
     OUTFLOW: ID= 1 (0011)      .25      .06    2.00  57.94       .19      1.78

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--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |   .0112       .0041
                          .0093      .0001    |   .0115       .0049
                          .0097      .0008    |   .0118       .0056
                          .0103      .0019    |   .0130       .0062
                          .0107      .0030    |   .0000       .0000

                                AREA      QPEAK        TPEAK       R.V.
                                (ha)      (cms)        (hrs)       (mm)
     INFLOW : ID= 2 (0011)      .247       .062         2.00       57.94
     OUTFLOW: ID= 1 (0006)      .247       .011         2.58       57.94

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 18.52
                   TIME SHIFT OF PEAK FLOW         (min)= 35.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0048

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--------------------
| ADD HYD   (0009) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):      .25     .011     2.58    57.94
        + ID2= 2 (0004):      .04     .007     2.08    45.58
          =================================================
          ID = 3 (0009):      .29     .018     2.17    56.13

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   4 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1735.071
| Ptotal= 93.87 mm |                          B=   6.014
--------------------                          C=    .820
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  = 12.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33

                 TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                  .17    1.52 |  3.17    6.04 |  6.17    4.54 |  9.17    2.12
                  .33    1.58 |  3.33    7.54 |  6.33    4.24 |  9.33    2.06
                  .50    1.65 |  3.50   10.16 |  6.50    3.99 |  9.50    2.01
                  .67    1.72 |  3.67   15.96 |  6.67    3.77 |  9.67    1.95
```

.83	1.80	3.83	40.64	6.83	3.57	9.83	1.91
1.00	1.88	4.00	178.50	7.00	3.40	10.00	1.86
1.17	1.98	4.17	54.03	7.17	3.24	10.17	1.82
1.33	2.08	4.33	27.31	7.33	3.09	10.33	1.78
1.50	2.21	4.50	18.23	7.50	2.97	10.50	1.74
1.67	2.34	4.67	13.73	7.67	2.85	10.67	1.70
1.83	2.50	4.83	11.05	7.83	2.74	10.83	1.67
2.00	2.69	5.00	9.28	8.00	2.64	11.00	1.63
2.17	2.90	5.17	8.02	8.17	2.55	11.17	1.60
2.33	3.16	5.33	7.08	8.33	2.46	11.33	1.57
2.50	3.48	5.50	6.34	8.50	2.38	11.50	1.54
2.67	3.87	5.67	5.76	8.67	2.31	11.67	1.51
2.83	4.39	5.83	5.28	8.83	2.24	11.83	1.48
3.00	5.07	6.00	4.88	9.00	2.18	12.00	1.46

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--------------------
| CALIB              |
| NASHYD    (0003) |   Area     (ha)=     .06   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=    7.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=     .17

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

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.083	1.52	3.083	6.04	6.083	4.54	9.08	2.12
.167	1.52	3.167	6.04	6.167	4.54	9.17	2.12
.250	1.58	3.250	7.54	6.250	4.24	9.25	2.06
.333	1.58	3.333	7.54	6.333	4.24	9.33	2.06
.417	1.65	3.417	10.16	6.417	3.99	9.42	2.01
.500	1.65	3.500	10.16	6.500	3.99	9.50	2.01
.583	1.72	3.583	15.96	6.583	3.77	9.58	1.95
.667	1.72	3.667	15.96	6.667	3.77	9.67	1.95
.750	1.80	3.750	40.64	6.750	3.57	9.75	1.91
.833	1.80	3.833	40.64	6.833	3.57	9.83	1.91
.917	1.88	3.917	178.49	6.917	3.40	9.92	1.86
1.000	1.88	4.000	178.50	7.000	3.40	10.00	1.86
1.083	1.98	4.083	54.03	7.083	3.24	10.08	1.82
1.167	1.98	4.167	54.03	7.167	3.24	10.17	1.82
1.250	2.08	4.250	27.31	7.250	3.09	10.25	1.78
1.333	2.08	4.333	27.31	7.333	3.09	10.33	1.78
1.417	2.21	4.417	18.23	7.417	2.97	10.42	1.74
1.500	2.21	4.500	18.23	7.500	2.97	10.50	1.74
1.583	2.34	4.583	13.73	7.583	2.85	10.58	1.70
1.667	2.34	4.667	13.73	7.667	2.85	10.67	1.70
1.750	2.50	4.750	11.05	7.750	2.74	10.75	1.67
1.833	2.50	4.833	11.05	7.833	2.74	10.83	1.67
1.917	2.69	4.917	9.28	7.917	2.64	10.92	1.63
2.000	2.69	5.000	9.28	8.000	2.64	11.00	1.63
2.083	2.90	5.083	8.02	8.083	2.55	11.08	1.60

```
              2.167    2.90 | 5.167    8.02 | 8.167    2.55 | 11.17    1.60
              2.250    3.16 | 5.250    7.08 | 8.250    2.46 | 11.25    1.57
              2.333    3.16 | 5.333    7.08 | 8.333    2.46 | 11.33    1.57
              2.417    3.48 | 5.417    6.34 | 8.417    2.38 | 11.42    1.54
              2.500    3.48 | 5.500    6.34 | 8.500    2.38 | 11.50    1.54
              2.583    3.87 | 5.583    5.76 | 8.583    2.31 | 11.58    1.51
              2.667    3.87 | 5.667    5.76 | 8.667    2.31 | 11.67    1.51
              2.750    4.39 | 5.750    5.28 | 8.750    2.24 | 11.75    1.48
              2.833    4.39 | 5.833    5.28 | 8.833    2.24 | 11.83    1.48
              2.917    5.07 | 5.917    4.88 | 8.917    2.18 | 11.92    1.46
              3.000    5.07 | 6.000    4.88 | 9.000    2.18 | 12.00    1.46

     Unit Hyd Qpeak  (cms)=    .012

     PEAK FLOW       (cms)=    .010 (i)
     TIME TO PEAK    (hrs)=   4.083
     RUNOFF VOLUME    (mm)=  57.085
     TOTAL RAINFALL   (mm)=  93.867
     RUNOFF COEFFICIENT   =    .608

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0001) |   Area     (ha)=    .11
|ID= 1 DT= 5.0 min |   Total Imp(%)=  73.00   Dir. Conn.(%)=  60.00
--------------------
                                IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=         .08           .03
     Dep. Storage     (mm)=        1.57          4.67
     Average Slope     (%)=        1.00          3.00
     Length            (m)=       26.70         26.00
     Mannings n           =        .013          .250

     Max.Eff.Inten.(mm/hr)=      178.50        248.99
                over (min)        5.00          5.00
     Storage Coeff.  (min)=         .92 (ii)     3.70 (ii)
     Unit Hyd. Tpeak (min)=        5.00          5.00
     Unit Hyd. peak  (cms)=         .34           .25
                                                             *TOTALS*
     PEAK FLOW       (cms)=         .03           .02           .050 (iii)
     TIME TO PEAK    (hrs)=        4.00          4.00           4.00
     RUNOFF VOLUME    (mm)=       92.30         42.93          72.55
     TOTAL RAINFALL   (mm)=       93.87         93.87          93.87
     RUNOFF COEFFICIENT   =         .98           .46            .77

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20           K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.    (mm)=    .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
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     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0002) |   Area    (ha)=    .14
|ID= 1 DT= 5.0 min |   Total Imp(%)=  46.00   Dir. Conn.(%)=  42.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=         .06          .08
     Dep. Storage     (mm)=        1.57         4.67
     Average Slope     (%)=        1.00         3.00
     Length            (m)=       30.60        26.00
     Mannings n          =         .013         .250

     Max.Eff.Inten.(mm/hr)=      178.50       163.01
                over (min)         5.00        10.00
     Storage Coeff.  (min)=        1.00 (ii)    5.23 (ii)
     Unit Hyd. Tpeak (min)=        5.00        10.00
     Unit Hyd. peak  (cms)=         .34          .16
                                                              *TOTALS*
     PEAK FLOW       (cms)=         .03          .03            .053 (iii)
     TIME TO PEAK    (hrs)=        4.00         4.08            4.00
     RUNOFF VOLUME    (mm)=       92.30        35.11           59.13
     TOTAL RAINFALL   (mm)=       93.87        93.87           93.87
     RUNOFF COEFFICIENT   =         .98          .37             .63

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20          K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0004) |   Area    (ha)=    .04   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=   7.10   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .007 (i)
     TIME TO PEAK    (hrs)=   4.083
     RUNOFF VOLUME    (mm)=  55.495
     TOTAL RAINFALL   (mm)=  93.867
     RUNOFF COEFFICIENT   =    .591

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| RESERVOIR (0005) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |   .0076       .0009
                          .0010      .0001    |   .0079       .0015
                          .0035      .0002    |   .0081       .0029
                          .0041      .0004    |   .0082       .0041
                          .0046      .0005    |   .1960       .0056
                          .0055      .0009    |   .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0001)       .107       .050        4.00       72.55
     OUTFLOW: ID= 1 (0005)       .107       .008        4.25       72.37

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 16.15
                   TIME SHIFT OF PEAK FLOW         (min)= 15.00
                   MAXIMUM  STORAGE   USED        (ha.m.)=   .0034

-------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0010)|     PIPE Number      =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 250.00
| DT=  5.0 min     |     Length       (m)=  19.50
--------------------     Slope      (m/m)=   .030
                         Manning n        =   .013

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)     (cms)        (m/s)        min
        .01     .193E-01       .0           .56          .58
        .03     .537E-01       .0           .87          .37
        .04     .970E-01       .0          1.12          .29
        .05     .147E+00       .0          1.33          .24
        .07     .201E+00       .0          1.51          .21
        .08     .259E+00       .0          1.68          .19
        .09     .320E+00       .0          1.82          .18
        .11     .383E+00       .0          1.94          .17
        .12     .447E+00       .0          2.05          .16
        .13     .511E+00       .1          2.14          .15
        .14     .574E+00       .1          2.22          .15
        .16     .637E+00       .1          2.29          .14
        .17     .698E+00       .1          2.34          .14
        .18     .756E+00       .1          2.37          .14
        .20     .811E+00       .1          2.39          .14
        .21     .860E+00       .1          2.39          .14
        .22     .904E+00       .1          2.36          .14
        .24     .938E+00       .1          2.30          .14
        .25     .957E+00       .1          2.10          .15
                                   <---- hydrograph ---->   <-pipe / channel->
                              AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                              (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
```

```
     INFLOW : ID= 2 (0005)      .11       .01    4.25  72.37       .05       1.23
     OUTFLOW: ID= 1 (0010)      .11       .01    4.17  72.36       .05       1.24

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--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0010):      .11    .008     4.17    72.36
        + ID2= 2 (0002):      .14    .053     4.00    59.13
          ==================================================
          ID = 3 (0007):      .25    .061     4.00    64.84

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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--------------------
| ROUTE PIPE (0011)|     PIPE Number     =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 150.00
| DT=  5.0 min     |     Length       (m)=  70.50
--------------------     Slope      (m/m)=   .020
                         Manning n       =   .013

     **** WARNING: MINIMUM PIPE SIZE REQUIRED =   221.97   (mm)FOR FREE FLOW.
                   THIS SIZE WAS USED IN THE ROUTING.
                   THE CAPACITY OF THIS PIPE  =      .06  (cms)

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)      (cu.m.)      (cms)        (m/s)        min
        .01     .550E-01       .0            .42         2.79
        .02     .153E+00       .0            .66         1.79
        .04     .276E+00       .0            .84         1.39
        .05     .418E+00       .0           1.00         1.17
        .06     .573E+00       .0           1.14         1.03
        .07     .739E+00       .0           1.26          .93
        .08     .912E+00       .0           1.37          .86
        .09     .109E+01       .0           1.46          .80
        .11     .127E+01       .0           1.55          .76
        .12     .146E+01       .0           1.62          .73
        .13     .164E+01       .0           1.68          .70
        .14     .182E+01       .0           1.73          .68
        .15     .199E+01       .0           1.76          .67
        .16     .215E+01       .1           1.79          .66
        .18     .231E+01       .1           1.80          .65
        .19     .245E+01       .1           1.80          .65
        .20     .258E+01       .1           1.78          .66
        .21     .267E+01       .1           1.74          .68
        .22     .273E+01       .1           1.58          .74
                                 <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                            (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
```

```
    INFLOW : ID= 2 (0007)      .25       .06    4.00  64.84        .18       1.80
    OUTFLOW: ID= 1 (0011)      .25       .06    4.00  64.83        .19       1.79


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--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |    .0112       .0041
                          .0093      .0001    |    .0115       .0049
                          .0097      .0008    |    .0118       .0056
                          .0103      .0019    |    .0130       .0062
                          .0107      .0030    |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
                                (ha)      (cms)       (hrs)       (mm)
    INFLOW : ID= 2 (0011)       .247       .064        4.00       64.83
    OUTFLOW: ID= 1 (0006)       .247       .012        4.58       64.83

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 17.99
                  TIME SHIFT OF PEAK FLOW         (min)= 35.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0050

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--------------------
| ADD HYD   (0009) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):      .25    .012     4.58    64.83
        + ID2= 2 (0004):      .04    .007     4.08    55.50
          ====================================================
          ID = 3 (0009):      .29    .018     4.17    63.46

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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  ****************************
  ** SIMULATION NUMBER:   5 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 732.951
| Ptotal= 42.34 mm |                          B=   6.199
--------------------                          C=    .810
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  = 12.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33

                 TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
```

```
         hrs    mm/hr |   hrs    mm/hr |   hrs    mm/hr |   hrs    mm/hr
         .17      .72 |  3.17     2.81 |  6.17     2.12 |  9.17     1.00
         .33      .75 |  3.33     3.50 |  6.33     1.99 |  9.33      .97
         .50      .78 |  3.50     4.69 |  6.50     1.87 |  9.50      .95
         .67      .82 |  3.67     7.30 |  6.67     1.77 |  9.67      .93
         .83      .85 |  3.83    18.21 |  6.83     1.68 |  9.83      .90
        1.00      .89 |  4.00    76.81 |  7.00     1.60 | 10.00      .88
        1.17      .94 |  4.17    24.08 |  7.17     1.52 | 10.17      .86
        1.33      .99 |  4.33    12.36 |  7.33     1.46 | 10.33      .84
        1.50     1.04 |  4.50     8.32 |  7.50     1.40 | 10.50      .82
        1.67     1.11 |  4.67     6.30 |  7.67     1.34 | 10.67      .81
        1.83     1.18 |  4.83     5.09 |  7.83     1.29 | 10.83      .79
        2.00     1.27 |  5.00     4.29 |  8.00     1.24 | 11.00      .78
        2.17     1.37 |  5.17     3.72 |  8.17     1.20 | 11.17      .76
        2.33     1.49 |  5.33     3.29 |  8.33     1.16 | 11.33      .75
        2.50     1.63 |  5.50     2.95 |  8.50     1.13 | 11.50      .73
        2.67     1.82 |  5.67     2.68 |  8.67     1.09 | 11.67      .72
        2.83     2.05 |  5.83     2.46 |  8.83     1.06 | 11.83      .71
        3.00     2.37 |  6.00     2.28 |  9.00     1.03 | 12.00      .69

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD     (0003) |   Area     (ha)=     .06   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=    7.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=     .17

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


                       ---- TRANSFORMED HYETOGRAPH ----
        TIME     RAIN |  TIME     RAIN |  TIME     RAIN |  TIME     RAIN
         hrs    mm/hr |   hrs    mm/hr |   hrs    mm/hr |   hrs    mm/hr
        .083      .72 | 3.083     2.81 | 6.083     2.12 |  9.08     1.00
        .167      .72 | 3.167     2.81 | 6.167     2.12 |  9.17     1.00
        .250      .75 | 3.250     3.50 | 6.250     1.99 |  9.25      .97
        .333      .75 | 3.333     3.50 | 6.333     1.99 |  9.33      .97
        .417      .78 | 3.417     4.69 | 6.417     1.87 |  9.42      .95
        .500      .78 | 3.500     4.69 | 6.500     1.87 |  9.50      .95
        .583      .82 | 3.583     7.30 | 6.583     1.77 |  9.58      .93
        .667      .82 | 3.667     7.30 | 6.667     1.77 |  9.67      .93
        .750      .85 | 3.750    18.21 | 6.750     1.68 |  9.75      .90
        .833      .85 | 3.833    18.21 | 6.833     1.68 |  9.83      .90
        .917      .89 | 3.917    76.80 | 6.917     1.60 |  9.92      .88
       1.000      .89 | 4.000    76.81 | 7.000     1.60 | 10.00      .88
       1.083      .94 | 4.083    24.08 | 7.083     1.52 | 10.08      .86
       1.167      .94 | 4.167    24.08 | 7.167     1.52 | 10.17      .86
       1.250      .99 | 4.250    12.36 | 7.250     1.46 | 10.25      .84
       1.333      .99 | 4.333    12.36 | 7.333     1.46 | 10.33      .84
       1.417     1.04 | 4.417     8.32 | 7.417     1.40 | 10.42      .82
       1.500     1.04 | 4.500     8.32 | 7.500     1.40 | 10.50      .82
       1.583     1.11 | 4.583     6.30 | 7.583     1.34 | 10.58      .81
       1.667     1.11 | 4.667     6.30 | 7.667     1.34 | 10.67      .81
```

```
               1.750    1.18 | 4.750    5.09 | 7.750    1.29 | 10.75     .79
               1.833    1.18 | 4.833    5.09 | 7.833    1.29 | 10.83     .79
               1.917    1.27 | 4.917    4.29 | 7.917    1.24 | 10.92     .78
               2.000    1.27 | 5.000    4.29 | 8.000    1.24 | 11.00     .78
               2.083    1.37 | 5.083    3.72 | 8.083    1.20 | 11.08     .76
               2.167    1.37 | 5.167    3.72 | 8.167    1.20 | 11.17     .76
               2.250    1.49 | 5.250    3.29 | 8.250    1.16 | 11.25     .75
               2.333    1.49 | 5.333    3.29 | 8.333    1.16 | 11.33     .75
               2.417    1.63 | 5.417    2.95 | 8.417    1.13 | 11.42     .73
               2.500    1.63 | 5.500    2.95 | 8.500    1.13 | 11.50     .73
               2.583    1.82 | 5.583    2.68 | 8.583    1.09 | 11.58     .72
               2.667    1.82 | 5.667    2.68 | 8.667    1.09 | 11.67     .72
               2.750    2.05 | 5.750    2.46 | 8.750    1.06 | 11.75     .71
               2.833    2.05 | 5.833    2.46 | 8.833    1.06 | 11.83     .71
               2.917    2.37 | 5.917    2.28 | 8.917    1.03 | 11.92     .69
               3.000    2.37 | 6.000    2.28 | 9.000    1.03 | 12.00     .69

     Unit Hyd Qpeak  (cms)=    .012

     PEAK FLOW       (cms)=    .002 (i)
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  15.518
     TOTAL RAINFALL   (mm)=  42.344
     RUNOFF COEFFICIENT   =    .366

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0001) |   Area    (ha)=    .11
|ID= 1 DT= 5.0 min |   Total Imp(%)=  73.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .08           .03
     Dep. Storage     (mm)=       1.57          4.67
     Average Slope     (%)=       1.00          3.00
     Length            (m)=      26.70         26.00
     Mannings n           =       .013          .250

     Max.Eff.Inten.(mm/hr)=      76.81        163.01
                over (min)        5.00         10.00
     Storage Coeff.  (min)=       1.29 (ii)     5.18 (ii)
     Unit Hyd. Tpeak (min)=       5.00         10.00
     Unit Hyd. peak  (cms)=        .33           .16
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .01           .00            .016 (iii)
     TIME TO PEAK    (hrs)=       4.00          4.08            4.00
     RUNOFF VOLUME    (mm)=      40.77          8.57           27.89
     TOTAL RAINFALL   (mm)=      42.34         42.34           42.34
     RUNOFF COEFFICIENT   =        .96           .20             .66

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!
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```
       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20           K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0002) |   Area    (ha)=    .14
|ID= 1 DT= 5.0 min |   Total Imp(%)=  46.00   Dir. Conn.(%)=  42.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .06          .08
     Dep. Storage     (mm)=       1.57         4.67
     Average Slope     (%)=       1.00         3.00
     Length            (m)=      30.60        26.00
     Mannings n          =        .013         .250

     Max.Eff.Inten.(mm/hr)=      76.81        19.97
                over (min)        5.00        15.00
     Storage Coeff.  (min)=       1.40 (ii)   10.59 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=        .33          .09
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .01          .00         .013 (iii)
     TIME TO PEAK    (hrs)=       4.00         4.17         4.00
     RUNOFF VOLUME    (mm)=      40.77         3.39        19.09
     TOTAL RAINFALL   (mm)=      42.34        42.34        42.34
     RUNOFF COEFFICIENT   =        .96          .08          .45

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

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 76.20           K  (1/hr)=  4.14
            Fc    (mm/hr)= 13.20     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0004) |   Area    (ha)=    .04   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   7.10   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .002 (i)
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  14.785
     TOTAL RAINFALL   (mm)=  42.344
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```
    RUNOFF COEFFICIENT   =    .349

    (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (0005) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |    .0076       .0009
                          .0010      .0001    |    .0079       .0015
                          .0035      .0002    |    .0081       .0029
                          .0041      .0004    |    .0082       .0041
                          .0046      .0005    |    .1960       .0056
                          .0055      .0009    |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
                                (ha)      (cms)       (hrs)       (mm)
    INFLOW : ID= 2 (0001)       .107       .016        4.00       27.89
    OUTFLOW: ID= 1 (0005)       .107       .007        4.17       27.80

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 42.43
                  TIME SHIFT OF PEAK FLOW         (min)= 10.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0009

------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0010)|     PIPE Number     =   1.00
| IN= 2---> OUT= 1 |     Diameter    (mm)= 250.00
| DT=  5.0 min     |     Length       (m)=  19.50
--------------------     Slope      (m/m)=   .030
                         Manning n       =   .013

    <----------------- TRAVEL TIME TABLE --------------------->
      DEPTH       VOLUME     FLOW RATE    VELOCITY     TRAV.TIME
       (m)       (cu.m.)       (cms)        (m/s)         min
       .01      .193E-01        .0           .56           .58
       .03      .537E-01        .0           .87           .37
       .04      .970E-01        .0          1.12           .29
       .05      .147E+00        .0          1.33           .24
       .07      .201E+00        .0          1.51           .21
       .08      .259E+00        .0          1.68           .19
       .09      .320E+00        .0          1.82           .18
       .11      .383E+00        .0          1.94           .17
       .12      .447E+00        .0          2.05           .16
       .13      .511E+00        .1          2.14           .15
       .14      .574E+00        .1          2.22           .15
       .16      .637E+00        .1          2.29           .14
       .17      .698E+00        .1          2.34           .14
       .18      .756E+00        .1          2.37           .14
       .20      .811E+00        .1          2.39           .14
       .21      .860E+00        .1          2.39           .14
       .22      .904E+00        .1          2.36           .14
```

```
      .24     .938E+00        .1          2.30          .14
      .25     .957E+00        .1          2.10          .15
                                  <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                            (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
     INFLOW : ID= 2 (0005)   .11      .01    4.17  27.80       .04      1.18
     OUTFLOW: ID= 1 (0010)   .11      .01    4.17  27.80       .04      1.18


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

--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0010):      .11    .007     4.17    27.80
        + ID2= 2 (0002):      .14    .013     4.00    19.09
          ==================================================
          ID = 3 (0007):      .25    .018     4.00    22.85

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE PIPE (0011)|      PIPE Number      =   1.00
| IN= 2---> OUT= 1 |      Diameter    (mm)= 150.00
| DT=  5.0 min     |      Length       (m)=  70.50
--------------------      Slope      (m/m)=   .020
                          Manning n        =   .013

     <----------------- TRAVEL TIME TABLE -------------------->
       DEPTH      VOLUME    FLOW RATE    VELOCITY    TRAV.TIME
        (m)       (cu.m.)     (cms)        (m/s)        min
        .01     .251E-01        .0          .32         3.63
        .02     .699E-01        .0          .51         2.33
        .02     .126E+00        .0          .65         1.81
        .03     .191E+00        .0          .77         1.52
        .04     .262E+00        .0          .88         1.34
        .05     .337E+00        .0          .97         1.21
        .06     .417E+00        .0         1.06         1.11
        .06     .498E+00        .0         1.13         1.04
        .07     .581E+00        .0         1.19          .99
        .08     .665E+00        .0         1.25          .94
        .09     .748E+00        .0         1.29          .91
        .09     .829E+00        .0         1.33          .88
        .10     .908E+00        .0         1.36          .86
        .11     .984E+00        .0         1.38          .85
        .12     .105E+01        .0         1.39          .85
        .13     .112E+01        .0         1.39          .85
        .13     .118E+01        .0         1.37          .86
        .14     .122E+01        .0         1.34          .88
        .15     .125E+01        .0         1.22          .96
                                  <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
```

```
                              (ha)     (cms)    (hrs)    (mm)        (m)        (m/s)
     INFLOW : ID= 2 (0007)      .25        .02    4.00  22.85        .11        1.37
     OUTFLOW: ID= 1 (0011)      .25        .02    4.00  22.84        .11        1.38


--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .0112       .0041
                           .0093      .0001   |    .0115       .0049
                           .0097      .0008   |    .0118       .0056
                           .0103      .0019   |    .0130       .0062
                           .0107      .0030   |    .0000       .0000

                                   AREA      QPEAK       TPEAK       R.V.
                                   (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0011)         .247       .020        4.00       22.84
     OUTFLOW: ID= 1 (0006)         .247       .010        4.25       22.84

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 48.70
                   TIME SHIFT OF PEAK FLOW         (min)= 15.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0007

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

--------------------
| ADD HYD   (0009) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):       .25    .010     4.25    22.84
        + ID2= 2 (0004):       .04    .002     4.17    14.79
          ==================================================
          ID = 3 (0009):       .29    .011     4.17    21.66

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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 FINISH
================================================================================
```

Schematic Summary Table

Hydrograph No.	Model Type	Item Represented	Comment
13	NASDHYD	Catchment Area contributing Runoff to the West Side Yard Swale	Includes both offsite and onsite areas
15	NASDHYD	Catchment Area contributing Runoff to the East Side Yard Swale	Includes both offsite and onsite areas

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

       V    V   I   SSSSS  U   U    A    L
       V    V   I   SS     U   U   A A   L
        V  V    I    SS    U   U  AAAAA  L
        V  V    I     SS   U   U  A   A  L
         VV     I   SSSSS  UUUUU  A   A  LLLLL

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


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


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  ****************************
  ** SIMULATION NUMBER:   2 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 998.071
| Ptotal= 56.17 mm |                          B=   6.053
--------------------                          C=    .814
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  = 12.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33

                         TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                          hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                          .17     .94 |  3.17    3.68 |  6.17    2.77 |  9.17    1.30
                          .33     .98 |  3.33    4.58 |  6.33    2.60 |  9.33    1.27
                          .50    1.02 |  3.50    6.15 |  6.50    2.44 |  9.50    1.24
                          .67    1.06 |  3.67    9.61 |  6.67    2.31 |  9.67    1.20
                          .83    1.11 |  3.83   24.17 |  6.83    2.19 |  9.83    1.17
                         1.00    1.16 |  4.00  104.19 |  7.00    2.08 | 10.00    1.15
                         1.17    1.22 |  4.17   32.04 |  7.17    1.99 | 10.17    1.12
                         1.33    1.28 |  4.33   16.34 |  7.33    1.90 | 10.33    1.10
                         1.50    1.36 |  4.50   10.96 |  7.50    1.82 | 10.50    1.07
                         1.67    1.44 |  4.67    8.29 |  7.67    1.75 | 10.67    1.05
                         1.83    1.54 |  4.83    6.69 |  7.83    1.68 | 10.83    1.03
                         2.00    1.65 |  5.00    5.63 |  8.00    1.62 | 11.00    1.01
                         2.17    1.78 |  5.17    4.87 |  8.17    1.57 | 11.17     .99
                         2.33    1.94 |  5.33    4.30 |  8.33    1.51 | 11.33     .97
                         2.50    2.13 |  5.50    3.86 |  8.50    1.47 | 11.50     .95
                         2.67    2.37 |  5.67    3.51 |  8.67    1.42 | 11.67     .93
                         2.83    2.68 |  5.83    3.22 |  8.83    1.38 | 11.83     .92
                         3.00    3.10 |  6.00    2.98 |  9.00    1.34 | 12.00     .90
```

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-------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0013) |   Area     (ha)=    .06   Curve Number   (CN)= 81.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=  11.60   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

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

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.083	.94	3.083	3.68	6.083	2.77	9.08	1.30
.167	.94	3.167	3.68	6.167	2.77	9.17	1.30
.250	.98	3.250	4.58	6.250	2.60	9.25	1.27
.333	.98	3.333	4.58	6.333	2.60	9.33	1.27
.417	1.02	3.417	6.15	6.417	2.44	9.42	1.24
.500	1.02	3.500	6.15	6.500	2.44	9.50	1.24
.583	1.06	3.583	9.61	6.583	2.31	9.58	1.20
.667	1.06	3.667	9.61	6.667	2.31	9.67	1.20
.750	1.11	3.750	24.17	6.750	2.19	9.75	1.17
.833	1.11	3.833	24.17	6.833	2.19	9.83	1.17
.917	1.16	3.917	104.19	6.917	2.08	9.92	1.15
1.000	1.16	4.000	104.19	7.000	2.08	10.00	1.15
1.083	1.22	4.083	32.04	7.083	1.99	10.08	1.12
1.167	1.22	4.167	32.04	7.167	1.99	10.17	1.12
1.250	1.28	4.250	16.34	7.250	1.90	10.25	1.10
1.333	1.28	4.333	16.34	7.333	1.90	10.33	1.10
1.417	1.36	4.417	10.96	7.417	1.82	10.42	1.07
1.500	1.36	4.500	10.96	7.500	1.82	10.50	1.07
1.583	1.44	4.583	8.29	7.583	1.75	10.58	1.05
1.667	1.44	4.667	8.29	7.667	1.75	10.67	1.05
1.750	1.54	4.750	6.69	7.750	1.68	10.75	1.03
1.833	1.54	4.833	6.69	7.833	1.68	10.83	1.03
1.917	1.65	4.917	5.63	7.917	1.62	10.92	1.01
2.000	1.65	5.000	5.63	8.000	1.62	11.00	1.01
2.083	1.78	5.083	4.87	8.083	1.57	11.08	.99
2.167	1.78	5.167	4.87	8.167	1.57	11.17	.99
2.250	1.94	5.250	4.30	8.250	1.51	11.25	.97
2.333	1.94	5.333	4.30	8.333	1.51	11.33	.97
2.417	2.13	5.417	3.86	8.417	1.47	11.42	.95
2.500	2.13	5.500	3.86	8.500	1.47	11.50	.95
2.583	2.37	5.583	3.51	8.583	1.42	11.58	.93
2.667	2.37	5.667	3.51	8.667	1.42	11.67	.93
2.750	2.68	5.750	3.22	8.750	1.38	11.75	.92
2.833	2.68	5.833	3.22	8.833	1.38	11.83	.92
2.917	3.10	5.917	2.98	8.917	1.34	11.92	.90
3.000	3.10	6.000	2.98	9.000	1.34	12.00	.90

```
     Unit Hyd Qpeak  (cms)=    .015

     PEAK FLOW       (cms)=    .003 (i)
```

```
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  19.000
     TOTAL RAINFALL   (mm)=  56.170
     RUNOFF COEFFICIENT   =    .338

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| CALIB              |
| NASHYD    (0015) |   Area     (ha)=     .08   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=    9.50   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=     .17

     Unit Hyd Qpeak  (cms)=    .017

     PEAK FLOW       (cms)=    .005 (i)
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  22.828
     TOTAL RAINFALL   (mm)=  56.170
     RUNOFF COEFFICIENT   =    .406

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   4 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1735.071
| Ptotal= 93.87 mm |                          B=   6.014
--------------------                          C=    .820
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  = 12.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.17	1.52	3.17	6.04	6.17	4.54	9.17	2.12
.33	1.58	3.33	7.54	6.33	4.24	9.33	2.06
.50	1.65	3.50	10.16	6.50	3.99	9.50	2.01
.67	1.72	3.67	15.96	6.67	3.77	9.67	1.95
.83	1.80	3.83	40.64	6.83	3.57	9.83	1.91
1.00	1.88	4.00	178.50	7.00	3.40	10.00	1.86
1.17	1.98	4.17	54.03	7.17	3.24	10.17	1.82
1.33	2.08	4.33	27.31	7.33	3.09	10.33	1.78
1.50	2.21	4.50	18.23	7.50	2.97	10.50	1.74
1.67	2.34	4.67	13.73	7.67	2.85	10.67	1.70
1.83	2.50	4.83	11.05	7.83	2.74	10.83	1.67
2.00	2.69	5.00	9.28	8.00	2.64	11.00	1.63
2.17	2.90	5.17	8.02	8.17	2.55	11.17	1.60

```
              2.33    3.16 |  5.33    7.08 |  8.33    2.46 | 11.33    1.57
              2.50    3.48 |  5.50    6.34 |  8.50    2.38 | 11.50    1.54
              2.67    3.87 |  5.67    5.76 |  8.67    2.31 | 11.67    1.51
              2.83    4.39 |  5.83    5.28 |  8.83    2.24 | 11.83    1.48
              3.00    5.07 |  6.00    4.88 |  9.00    2.18 | 12.00    1.46


--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0013) |   Area    (ha)=    .06   Curve Number   (CN)= 81.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=  11.60   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

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


                             ---- TRANSFORMED HYETOGRAPH ----
              TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
               hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
              .083    1.52 | 3.083    6.04 | 6.083    4.54 |  9.08    2.12
              .167    1.52 | 3.167    6.04 | 6.167    4.54 |  9.17    2.12
              .250    1.58 | 3.250    7.54 | 6.250    4.24 |  9.25    2.06
              .333    1.58 | 3.333    7.54 | 6.333    4.24 |  9.33    2.06
              .417    1.65 | 3.417   10.16 | 6.417    3.99 |  9.42    2.01
              .500    1.65 | 3.500   10.16 | 6.500    3.99 |  9.50    2.01
              .583    1.72 | 3.583   15.96 | 6.583    3.77 |  9.58    1.95
              .667    1.72 | 3.667   15.96 | 6.667    3.77 |  9.67    1.95
              .750    1.80 | 3.750   40.64 | 6.750    3.57 |  9.75    1.91
              .833    1.80 | 3.833   40.64 | 6.833    3.57 |  9.83    1.91
              .917    1.88 | 3.917  178.49 | 6.917    3.40 |  9.92    1.86
             1.000    1.88 | 4.000  178.50 | 7.000    3.40 | 10.00    1.86
             1.083    1.98 | 4.083   54.03 | 7.083    3.24 | 10.08    1.82
             1.167    1.98 | 4.167   54.03 | 7.167    3.24 | 10.17    1.82
             1.250    2.08 | 4.250   27.31 | 7.250    3.09 | 10.25    1.78
             1.333    2.08 | 4.333   27.31 | 7.333    3.09 | 10.33    1.78
             1.417    2.21 | 4.417   18.23 | 7.417    2.97 | 10.42    1.74
             1.500    2.21 | 4.500   18.23 | 7.500    2.97 | 10.50    1.74
             1.583    2.34 | 4.583   13.73 | 7.583    2.85 | 10.58    1.70
             1.667    2.34 | 4.667   13.73 | 7.667    2.85 | 10.67    1.70
             1.750    2.50 | 4.750   11.05 | 7.750    2.74 | 10.75    1.67
             1.833    2.50 | 4.833   11.05 | 7.833    2.74 | 10.83    1.67
             1.917    2.69 | 4.917    9.28 | 7.917    2.64 | 10.92    1.63
             2.000    2.69 | 5.000    9.28 | 8.000    2.64 | 11.00    1.63
             2.083    2.90 | 5.083    8.02 | 8.083    2.55 | 11.08    1.60
             2.167    2.90 | 5.167    8.02 | 8.167    2.55 | 11.17    1.60
             2.250    3.16 | 5.250    7.08 | 8.250    2.46 | 11.25    1.57
             2.333    3.16 | 5.333    7.08 | 8.333    2.46 | 11.33    1.57
             2.417    3.48 | 5.417    6.34 | 8.417    2.38 | 11.42    1.54
             2.500    3.48 | 5.500    6.34 | 8.500    2.38 | 11.50    1.54
             2.583    3.87 | 5.583    5.76 | 8.583    2.31 | 11.58    1.51
             2.667    3.87 | 5.667    5.76 | 8.667    2.31 | 11.67    1.51
             2.750    4.39 | 5.750    5.28 | 8.750    2.24 | 11.75    1.48
             2.833    4.39 | 5.833    5.28 | 8.833    2.24 | 11.83    1.48
```

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              2.917    5.07 | 5.917    4.88 | 8.917    2.18 | 11.92    1.46
              3.000    5.07 | 6.000    4.88 | 9.000    2.18 | 12.00    1.46

     Unit Hyd Qpeak  (cms)=    .015

     PEAK FLOW       (cms)=    .009 (i)
     TIME TO PEAK    (hrs)=   4.083
     RUNOFF VOLUME    (mm)=  47.537
     TOTAL RAINFALL   (mm)=  93.867
     RUNOFF COEFFICIENT   =    .506

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0015) |   Area    (ha)=    .08   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   9.50   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

     Unit Hyd Qpeak  (cms)=    .017

     PEAK FLOW       (cms)=    .013 (i)
     TIME TO PEAK    (hrs)=   4.083
     RUNOFF VOLUME    (mm)=  53.422
     TOTAL RAINFALL   (mm)=  93.867
     RUNOFF COEFFICIENT   =    .569

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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  ****************************
  ** SIMULATION NUMBER:   5 **
  ****************************

--------------------
| CHICAGO STORM    |     IDF curve parameters: A= 732.951
| Ptotal= 42.34 mm |                           B=   6.199
--------------------                           C=    .810
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  = 12.00 hrs
                         Storm time step    = 10.00 min
                         Time to peak ratio =   .33

               TIME    RAIN | TIME    RAIN | TIME    RAIN | TIME    RAIN
                hrs   mm/hr |  hrs   mm/hr |  hrs   mm/hr |  hrs   mm/hr
                .17     .72 | 3.17    2.81 | 6.17    2.12 | 9.17    1.00
                .33     .75 | 3.33    3.50 | 6.33    1.99 | 9.33     .97
                .50     .78 | 3.50    4.69 | 6.50    1.87 | 9.50     .95
                .67     .82 | 3.67    7.30 | 6.67    1.77 | 9.67     .93
                .83     .85 | 3.83   18.21 | 6.83    1.68 | 9.83     .90
               1.00     .89 | 4.00   76.81 | 7.00    1.60 | 10.00    .88
               1.17     .94 | 4.17   24.08 | 7.17    1.52 | 10.17    .86
```

```
                    1.33      .99 |  4.33   12.36 |  7.33    1.46 | 10.33      .84
                    1.50     1.04 |  4.50    8.32 |  7.50    1.40 | 10.50      .82
                    1.67     1.11 |  4.67    6.30 |  7.67    1.34 | 10.67      .81
                    1.83     1.18 |  4.83    5.09 |  7.83    1.29 | 10.83      .79
                    2.00     1.27 |  5.00    4.29 |  8.00    1.24 | 11.00      .78
                    2.17     1.37 |  5.17    3.72 |  8.17    1.20 | 11.17      .76
                    2.33     1.49 |  5.33    3.29 |  8.33    1.16 | 11.33      .75
                    2.50     1.63 |  5.50    2.95 |  8.50    1.13 | 11.50      .73
                    2.67     1.82 |  5.67    2.68 |  8.67    1.09 | 11.67      .72
                    2.83     2.05 |  5.83    2.46 |  8.83    1.06 | 11.83      .71
                    3.00     2.37 |  6.00    2.28 |  9.00    1.03 | 12.00      .69


--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0013) |   Area     (ha)=     .06   Curve Number   (CN)= 81.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=  11.60   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

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


                             ---- TRANSFORMED HYETOGRAPH ----
                    TIME     RAIN |  TIME     RAIN |  TIME     RAIN |  TIME     RAIN
                     hrs    mm/hr |   hrs    mm/hr |   hrs    mm/hr |   hrs    mm/hr
                    .083      .72 | 3.083    2.81 | 6.083    2.12 |  9.08    1.00
                    .167      .72 | 3.167    2.81 | 6.167    2.12 |  9.17    1.00
                    .250      .75 | 3.250    3.50 | 6.250    1.99 |  9.25     .97
                    .333      .75 | 3.333    3.50 | 6.333    1.99 |  9.33     .97
                    .417      .78 | 3.417    4.69 | 6.417    1.87 |  9.42     .95
                    .500      .78 | 3.500    4.69 | 6.500    1.87 |  9.50     .95
                    .583      .82 | 3.583    7.30 | 6.583    1.77 |  9.58     .93
                    .667      .82 | 3.667    7.30 | 6.667    1.77 |  9.67     .93
                    .750      .85 | 3.750   18.21 | 6.750    1.68 |  9.75     .90
                    .833      .85 | 3.833   18.21 | 6.833    1.68 |  9.83     .90
                    .917      .89 | 3.917   76.80 | 6.917    1.60 |  9.92     .88
                   1.000      .89 | 4.000   76.81 | 7.000    1.60 | 10.00     .88
                   1.083      .94 | 4.083   24.08 | 7.083    1.52 | 10.08     .86
                   1.167      .94 | 4.167   24.08 | 7.167    1.52 | 10.17     .86
                   1.250      .99 | 4.250   12.36 | 7.250    1.46 | 10.25     .84
                   1.333      .99 | 4.333   12.36 | 7.333    1.46 | 10.33     .84
                   1.417     1.04 | 4.417    8.32 | 7.417    1.40 | 10.42     .82
                   1.500     1.04 | 4.500    8.32 | 7.500    1.40 | 10.50     .82
                   1.583     1.11 | 4.583    6.30 | 7.583    1.34 | 10.58     .81
                   1.667     1.11 | 4.667    6.30 | 7.667    1.34 | 10.67     .81
                   1.750     1.18 | 4.750    5.09 | 7.750    1.29 | 10.75     .79
                   1.833     1.18 | 4.833    5.09 | 7.833    1.29 | 10.83     .79
                   1.917     1.27 | 4.917    4.29 | 7.917    1.24 | 10.92     .78
                   2.000     1.27 | 5.000    4.29 | 8.000    1.24 | 11.00     .78
                   2.083     1.37 | 5.083    3.72 | 8.083    1.20 | 11.08     .76
                   2.167     1.37 | 5.167    3.72 | 8.167    1.20 | 11.17     .76
                   2.250     1.49 | 5.250    3.29 | 8.250    1.16 | 11.25     .75
                   2.333     1.49 | 5.333    3.29 | 8.333    1.16 | 11.33     .75
```

```
              2.417    1.63 | 5.417    2.95 | 8.417    1.13 | 11.42     .73
              2.500    1.63 | 5.500    2.95 | 8.500    1.13 | 11.50     .73
              2.583    1.82 | 5.583    2.68 | 8.583    1.09 | 11.58     .72
              2.667    1.82 | 5.667    2.68 | 8.667    1.09 | 11.67     .72
              2.750    2.05 | 5.750    2.46 | 8.750    1.06 | 11.75     .71
              2.833    2.05 | 5.833    2.46 | 8.833    1.06 | 11.83     .71
              2.917    2.37 | 5.917    2.28 | 8.917    1.03 | 11.92     .69
              3.000    2.37 | 6.000    2.28 | 9.000    1.03 | 12.00     .69

     Unit Hyd Qpeak  (cms)=    .015

     PEAK FLOW       (cms)=    .001 (i)
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  10.420
     TOTAL RAINFALL   (mm)=  42.344
     RUNOFF COEFFICIENT   =    .246

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB             |
| NASHYD    (0015) |   Area    (ha)=    .08   Curve Number   (CN)= 84.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   9.50   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .17

     Unit Hyd Qpeak  (cms)=    .017

     PEAK FLOW       (cms)=    .002 (i)
     TIME TO PEAK    (hrs)=   4.167
     RUNOFF VOLUME    (mm)=  13.226
     TOTAL RAINFALL   (mm)=  42.344
     RUNOFF COEFFICIENT   =    .312

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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 FINISH
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Appendix B: Product Information

- Hydrovex Selection Chart
- Brentwood Storage Tanks

HYDROVEX®

SVHV Vertical Vortex Flow Regulator

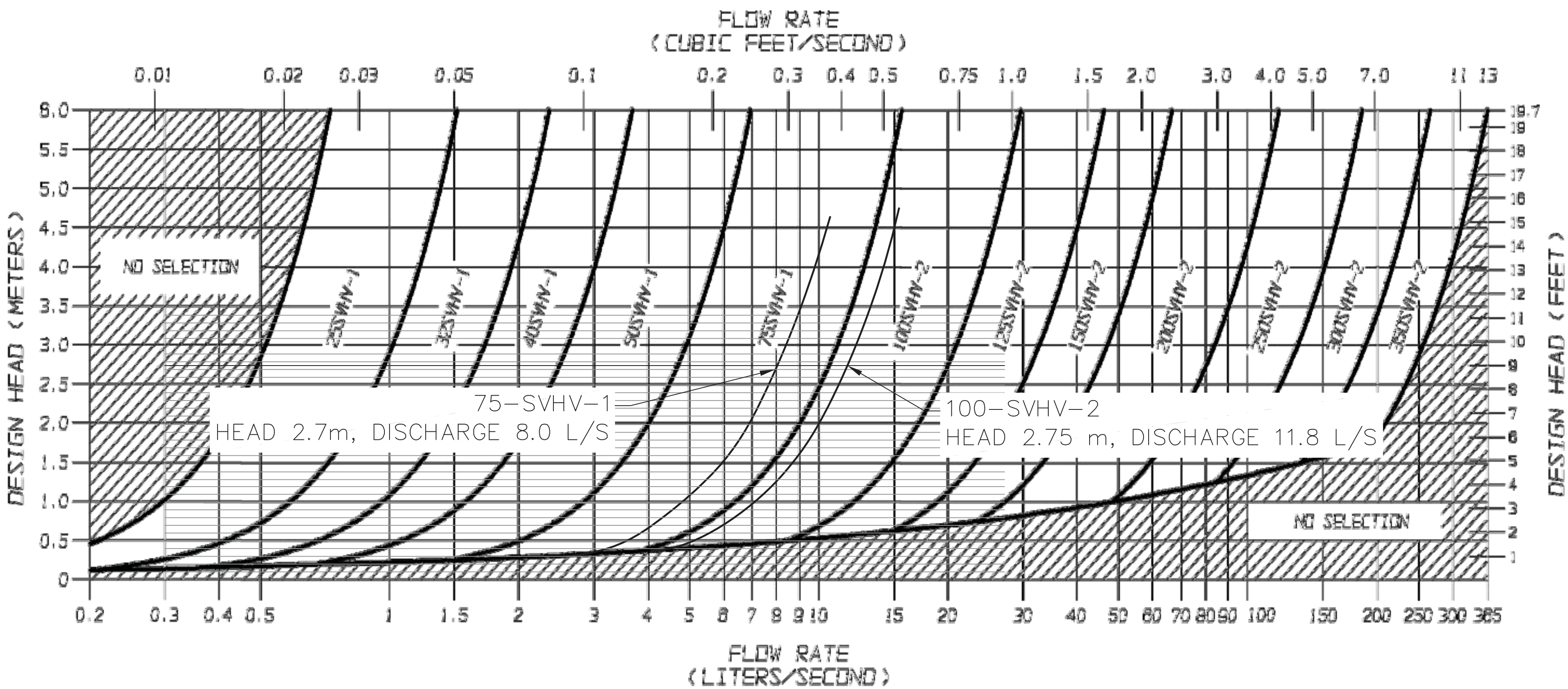
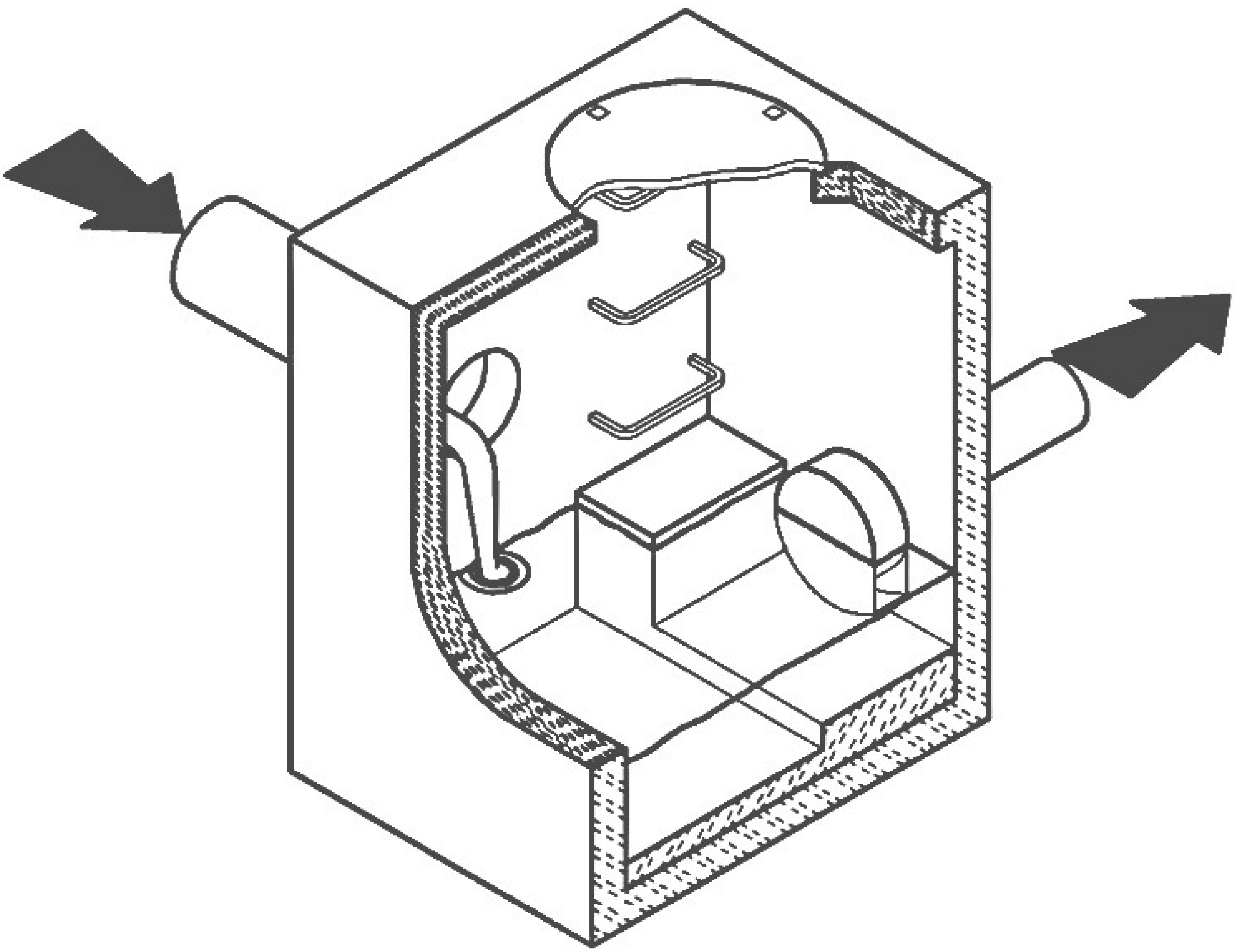


FIGURE 3 - SVHV

JOHN MEUNIER

® HYDROVEX® VHV / SVHV
Vertical Vortex Flow Regulator



JOHN MEUNIER

HYDROVEX® VHV / SVHV VERTICAL VORTEX FLOW REGULATOR

APPLICATIONS

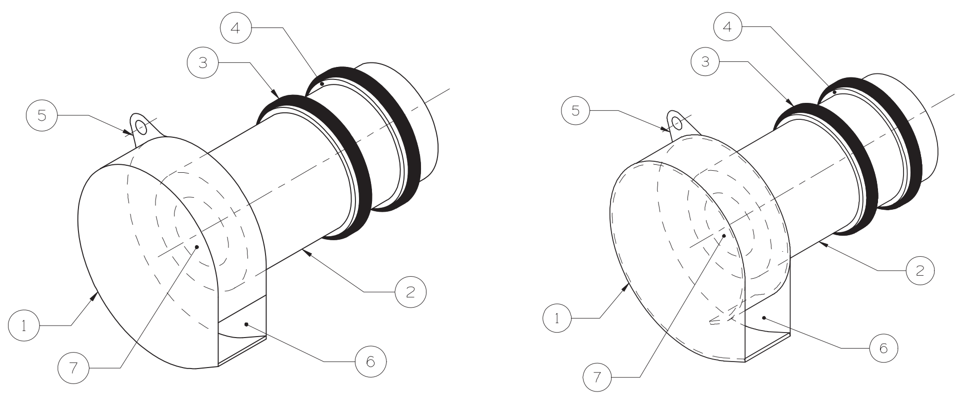
One of the major problems of urban wet weather flow management is the runoff generated after a heavy rainfall. During a storm, uncontrolled flows may overload the drainage system and cause flooding. Due to increased velocities, sewer pipe wear is increased dramatically and results in network deterioration. In a combined sewer system, the wastewater treatment plant may also experience significant increases in flows during storms, thereby losing its treatment efficiency.

A simple means of controlling excessive water runoff is by controlling excessive flows at their origin (manholes). **John Meunier Inc.** manufactures the **HYDROVEX® VHV / SVHV** line of vortex flow regulators to control stormwater flows in sewer networks, as well as manholes.

The vortex flow regulator design is based on the fluid mechanics principle of the forced vortex. This grants flow regulation without any moving parts, thus reducing maintenance. The operation of the regulator, depending on the upstream head and discharge, switches between orifice flow (gravity flow) and vortex flow. Although the concept is quite simple, over 12 years of research have been carried out in order to get a high performance.

The **HYDROVEX® VHV / SVHV** Vertical Vortex Flow Regulators **(refer to Figure 1)** are manufactured entirely of stainless steel, and consist of a hollow body (**1**) (in which flow control takes place) and an outlet orifice (**7**). Two rubber "O" rings (**3**) seal and retain the unit inside the outlet pipe. Two stainless steel retaining rings (**4**) are welded on the outlet sleeve to ensure that there is no shifting of the "O" rings during installation and use.

1. **BODY**
2. **SLEEVE**
3. **O-RING**
4. **RETAINING RINGS (SQUARE BAR)**
5. **ANCHOR PLATE**
6. **INLET**
7. **OUTLET ORIFICE**



VHV ***SVHV***

FIGURE 1: HYDROVEX® VHV-SVHV VERTICAL VORTREX FLOW REGULATORS

ADVANTAGES

- The **HYDROVEX® VHV / SVHV** line of flow regulators are manufactured entirely of stainless steel, making them durable and corrosion resistant.
- Having no moving parts, they require minimal maintenance.
- The geometry of the **HYDROVEX® VHV / SVHV** flow regulators allows a control equal to an orifice plate, having a cross section area 4 to 6 times smaller. This decreases the chance of blockage of the regulator, due to sediments and debris found in stormwater flows. **Figure 2** illustrates the comparison between a regulator model 100 SVHV-2 and an equivalent orifice plate. One can see that for the same height of water, the regulator controls a flow approximately four times smaller than an equivalent orifice plate.
- Installation of the **HYDROVEX® VHV / SVHV** flow regulators is quick and straightforward and is performed after all civil works are completed.
- Installation requires no special tools or equipment and may be carried out by any contractor.
- Installation may be carried out in existing structures.

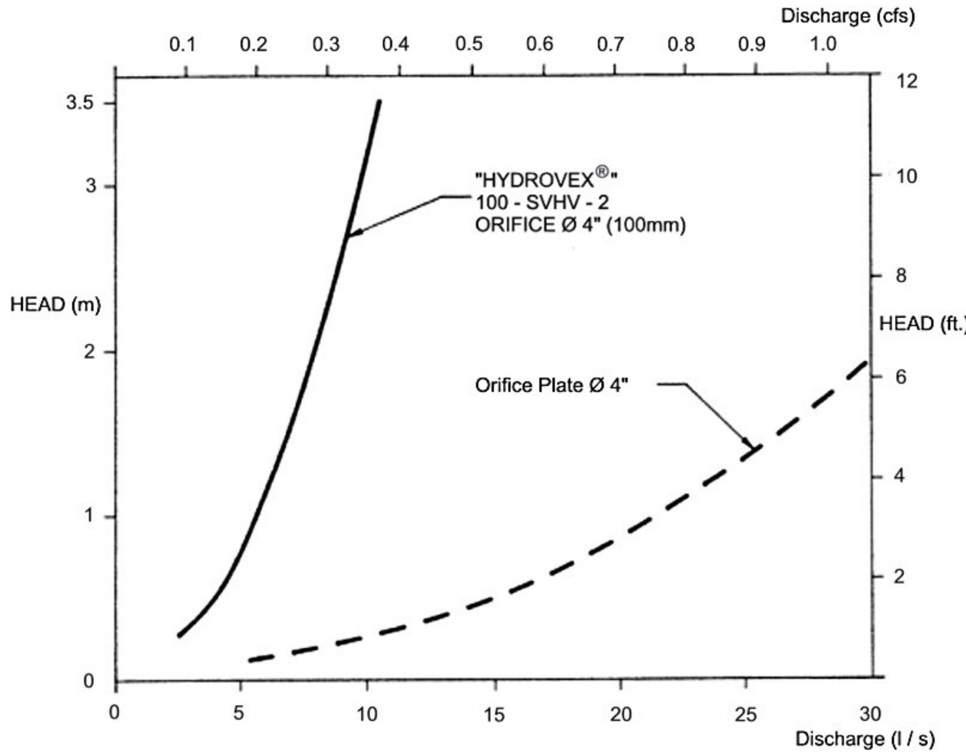


FIGURE 2: DISCHARGE CURVE SHOWING A HYDROVEX® FLOW REGULATOR VS AN ORIFICE PLATE

SELECTION

Selection of a **VHV or SVHV** regulator can be easily made using the selection charts found at the back of this brochure (see **Figure 3)**. These charts are a graphical representation of the maximum upstream water pressure (head) and the maximum discharge at the manhole outlet. The maximum design head is the difference between the maximum upstream water level and the invert of the outlet pipe. All selections should be verified by John Meunier Inc. personnel prior to fabrication.

Example:

✓ Maximum design head	2m (6.56 ft.)
✓ Maximum discharge	6 L/s (0.2 cfs)
✓ Using **Figure 3** - VHV	model required is a **75 VHV-1**

INSTALLATION REQUIREMENTS

All **HYDROVEX®** **VHV / SVHV** flow regulators can be installed in circular or square manholes. **Figure 4** gives the various minimum dimensions required for a given regulator. ***It is imperative to respect the minimum clearances shown to ensure easy installation and proper functioning of the regulator.***

SPECIFICATIONS

In order to specify a **HYDROVEX®** regulator, the following parameters must be defined:

- The model number (ex: 75-VHV-1)
- The diameter and type of outlet pipe (ex: 6" diam. SDR 35)
- The desired discharge (ex: 6 l/s or 0.21 CFS)
- The upstream head (ex: 2 m or 6.56 ft.) *
- The manhole diameter (ex: 36" diam.)
- The minimum clearance "H" (ex: 10 inches)
- The material type (ex: 304 s/s, 11 Ga. standard)

***** *Upstream head is defined as the difference in elevation between the maximum upstream water level and the invert of the outlet pipe where the HYDROVEX® flow regulator is to be installed.***

PLEASE NOTE THAT WHEN REQUESTING A PROPOSAL, WE SIMPLY REQUIRE THAT YOU PROVIDE US WITH THE FOLLOWING:

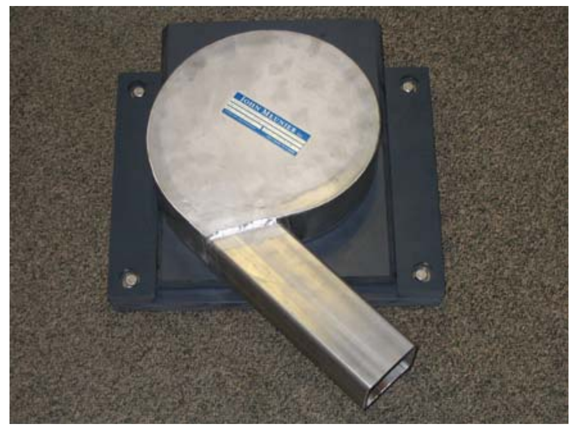
- ***project design flow rate***
- ***pressure head***
- ***chamber's outlet pipe diameter and type***

Typical VHV model in factory

OPTIONS

FV – SVHV (mounted on sliding plate)

VHV-1-O (standard model with odour control inlet)



FV – VHV-O (mounted on sliding plate with odour control inlet)

VHV with Gooseneck assembly in existing chamber without minimum release at the bottom

VHV with air vent for minimal slopes

® HYDROVEX®

SVHV Vertical Vortex Flow Regulator

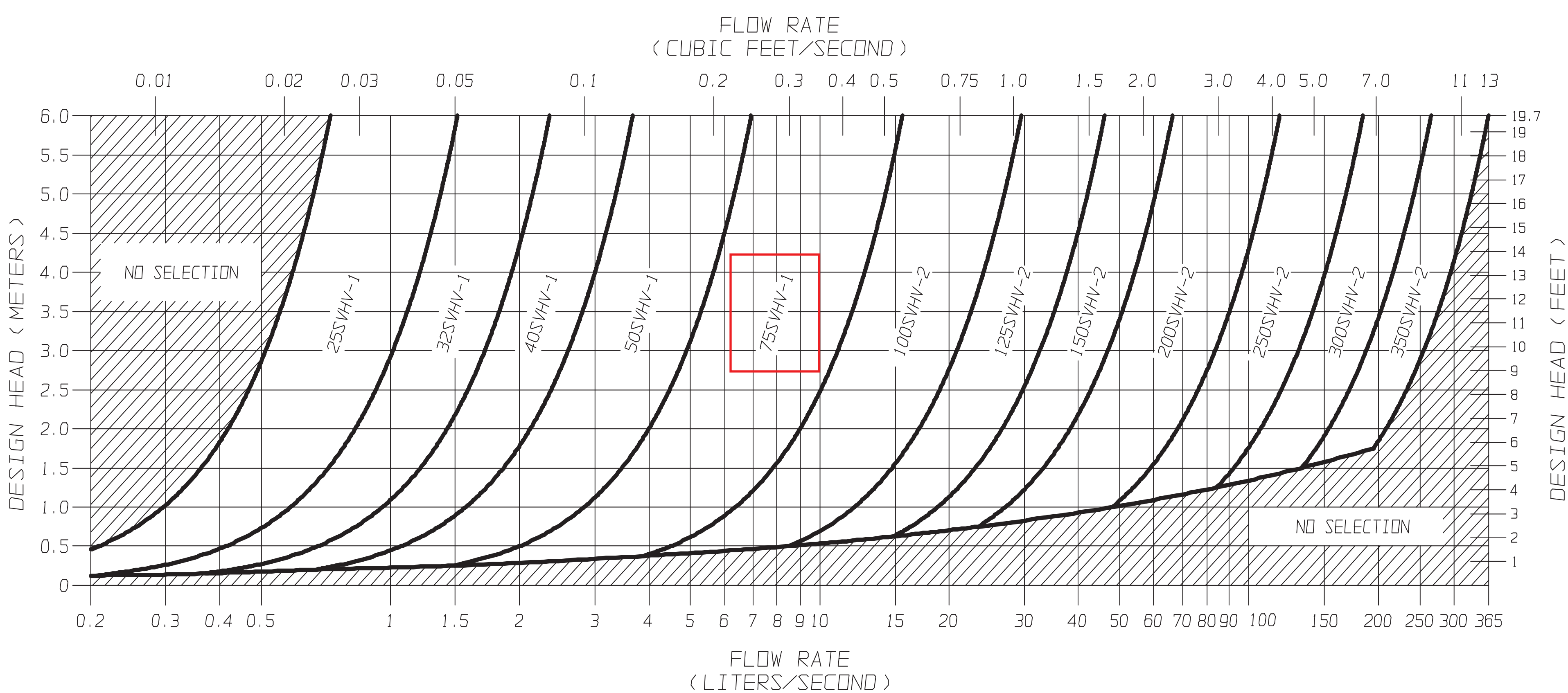


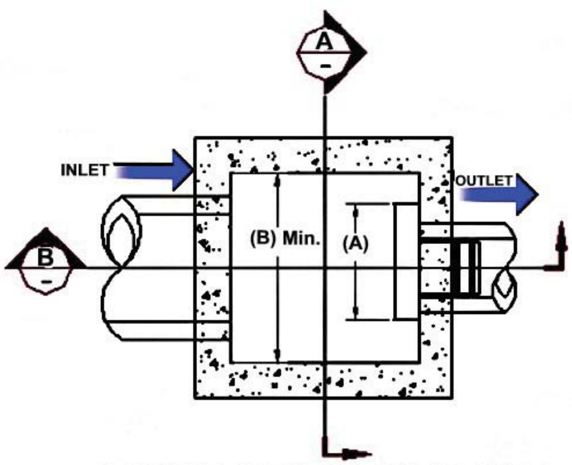
FIGURE 3 - SVHV

JOHN MEUNIER

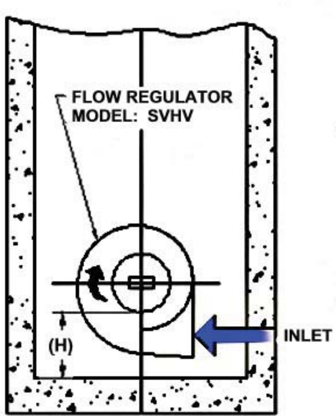
FLOW REGULATOR TYPICAL INSTALLATION IN SQUARE MANHOLE
FIGURE 4 (MODEL SVHV)

Model Number	Regulator Diameter		Minimum Chamber Width		Minimum Outlet Pipe Diameter		Minimum Clearance	
	A (mm)	**A** (in.)	**B** (mm)	**B** (in.)	**C** (mm)	**C** (in.)	**H** (mm)	**H** (in.)
25 SVHV-1	125	5	600	24	150	6	150	6
32 SVHV-1	150	6	600	24	150	6	150	6
40 SVHV-1	200	8	600	24	150	6	150	6
50 SVHV-1	250	10	600	24	150	6	150	6
75 SVHV-1	375	15	600	24	150	6	275	11
100 SVHV-2	275	11	600	24	150	6	250	10
125 SVHV-2	350	14	600	24	150	6	300	12
150 SVHV-2	425	17	600	24	150	6	350	14
200 SVHV-2	575	23	900	36	200	8	450	18
250 SVHV-2	700	28	900	36	250	10	550	22
300 SVHV-2	850	34	1200	48	250	10	650	26
350 SVHV-2	1000	40	1200	48	250	10	700	28

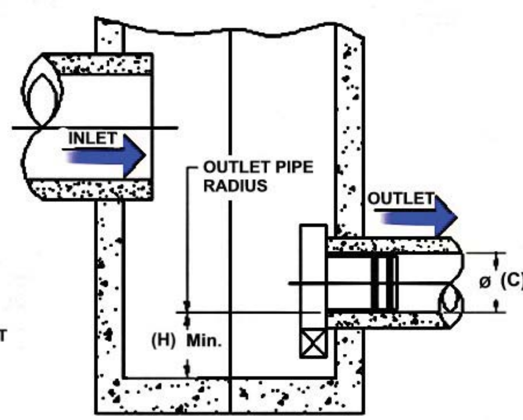
NOTE: ***In the case of a square manhole, the outlet flow pipe must be centered on the wall to ensure enough clearance for the unit.***



SQUARE / RECTANGULAR WELL



SECTION A



SECTION B

INSTALLATION

The installation of a **HYDROVEX®** regulator may be undertaken once the manhole and piping is in place. Installation consists of simply fitting the regulator into the outlet pipe of the manhole. **John Meunier Inc.** recommends the use of a lubricant on the outlet pipe, in order to facilitate the insertion and orientation of the flow controller.

MAINTENANCE

HYDROVEX® regulators are manufactured in such a way as to be maintenance free; however, a periodic inspection (every 3-6 months) is suggested in order to ensure that neither the inlet nor the outlet has become blocked with debris. The manhole should undergo periodically, particularly after major storms, inspection and cleaning as established by the municipality

GUARANTY

The **HYDROVEX®** line of **VHV / SVHV** regulators are guaranteed against both design and manufacturing defects for a period of 5 years. Should a unit be defective, **John Meunier Inc.** is solely responsible for either modification or replacement of the unit.

John Meunier Inc.
ISO 9001 : 2008

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VEOLIA WATER Solutions & Technologies

Revised: 2011-05-03

DESIGN GUIDE

STORMTANK® Module

BRENTWOOD®

Contents

General Notes

1. Brentwood recommends that the installing contractor contact either Brentwood or the local distributor prior to installation of the system to schedule a pre-construction meeting. This meeting will ensure that the installing contractor has a firm understanding of the installation instructions.
2. All systems must be designed and installed to meet or exceed Brentwood's minimum requirements. Although Brentwood offers support during the design, review, and construction phases of the Module system, it is the ultimate responsibility of the Engineer of Record to design the system in full compliance with all applicable engineering practices, laws, and regulations.
3. Brentwood requires a minimum cover of 24" (610 mm) and/or a maximum Module invert of 11' (3.35 m). Additionally, a minimum 6" (152 mm) leveling bed, 12" (305 mm) side backfill, and 12" (305 mm) top backfill are required on every system.
4. Brentwood recommends a minimum bearing capacity and subgrade compaction for all installations. If site conditions are found not to meet any design requirements during installation, the Engineer of Record must be contacted immediately.
5. All installations require a minimum two layers of geotextile fabric. One layer is to be installed around the Modules, and another layer is to be installed between the stone/soil interfaces.
6. Stone backfilling is to follow all requirements of the most current installation instructions.
7. The installing contractor must apply all protective measures to prevent sediment from entering the system during and after installation per local, state, and federal regulations.
8. The StormTank® Module carries a Limited Warranty, which can be accessed at www.brentwoodindustries.com.

1.0 Introduction

About Brentwood

Brentwood is a global manufacturer of custom and proprietary products and systems for the construction, consumer, medical, power, transportation, and water industries. A focus on plastics innovation, coupled with diverse production capabilities and engineering expertise, has allowed Brentwood to build a strong reputation for thermoplastic molding and solutions development.

Brentwood's product and service offerings continue to grow with an ever-increasing manufacturing presence. By emphasizing customer service and working closely with clients throughout the design, engineering, and manufacturing phases of each project, Brentwood develops forward-thinking strategies to create targeted, tailored solutions.

StormTank® Module

The StormTank Module is a strong, yet lightweight, alternative to other subsurface systems and offers the largest void space (up to 97%) of any subsurface stormwater storage unit on the market. The Modules are simple to assemble on site, limiting shipping costs, installation time, and labor. Their structural PVC columns pressure fit into the polypropylene top/bottom platens, with side panels inserted around the perimeter of the system. This open design and lack of internal walls make the Module system easy to clean compared to other subsurface box structures. When properly designed, applied, installed, and maintained, the Module system has been engineered to achieve a 50-year lifespan.

Technical Support

Brentwood's knowledgeable distributor network and in-house associates emphasize customer service and support by parterning with customers to extend the process beyond physical material supply. These trained specialists are available to assist in the review of proposed systems, conversions of alternatively designed systems, or to resolve any potential concerns before, during, and after the design process. To provide the best assistance, it is recommended that associates be provided with a site plan and cross-sections that include grading, drainage structures, dimensions, etc.

2.0 Product Information

Applications

The Module system can be utilized for detention, infiltration, capture and reuse, and specialty applications across a wide range of industries, including the commercial, residential, and recreational segments. The product's modular design allows the system to be configured in almost any shape (even around utilities) and to be located under almost any pervious or impervious surface.

Module Selection

Brentwood manufactures the Module in five different heights (Table 1) that can be stacked uniformly up to two Modules high. This allows for numerous height configurations up to 6' (1.83 m) tall. The Modules can be buried up to a maximum invert of 11' (3.35 m) and require a minimum cover of 24" (610 mm) for load rating. When selecting the proper Module, it is important to consider the minimum required cover, any groundwater or limiting zone restrictions, footprint requirements, and all local, state, and federal regulations.

Table 1: Nominal StormTank® Module Specificiations

	ST-18	ST-24	ST-30	ST-33	ST-36
Height	18" (457 mm)	24" (610 mm)	30" (762 mm)	33" (838 mm)	36" (914 mm)
Void Space	95.5%	96.0%	96.5%	96.9%	97.0%
Module Storage Capacity	6.54 ft^3 (0.18 m^3)	8.64 ft^3 (0.24 m^3)	10.86 ft^3 (0.31 m^3)	11.99 ft^3 (0.34 m^3)	13.10 ft^3 (0.37 m^3)
Min. Installed Capacity*	9.15 ft^3 (0.26 m^3)	11.34 ft^3 (0.32 m^3)	13.56 ft^3 (0.38 m^3)	14.69 ft^3 (0.42 m^3)	15.80 ft^3 (0.45 m^3)
Weight	22.70 lbs (10.30 kg)	26.30 lbs (11.93 kg)	29.50 lbs (13.38 kg)	31.3 lbs (14.20 kg)	33.10 lbs (15.01 kg)

Min. Installed Capacity includes the leveling bed, Module, and top backfill storage capacity for one Module. Stone storage capacity is based on 40% void space.* ***Side backfill storage is not included.*

3.0 Manufacturing Standards

Brentwood selects material based on long-term performance needs. To ensure long-term performance and limit component deflection over time (creep), Brentwood selected polyvinyl chloride (PVC) for the Module's structural columns and a virgin polypropylene (PP) blend for the top/bottom and side panels. PVC provides the largest creep resistance of commonly available plastics, and therefore, provides the best performance under loading conditions. Materials like polyethylene (HDPE) and recycled PP have lower creep resistance and are not recommended for load-bearing products and applications.

Materials:

Brentwood's proprietary PVC and PP copolymer resins have been chosen specifically for utilization in the StormTank® Module. The PVC is blended in house by experts and is a 100% blend of post-manuacturing/pre-consumer recycled material. Both materials exhibit structural resilience and naturally resist the chemicals typically found in stormwater runoff.

Methods:

Injection Molding

The Module's top/bottom platens and side panels are injection molded, using proprietary molds and materials. This allows Brentwood to manufacture a product that meets structural requirements while maintaining dimensional control, molded-in traceability, and quality control.

Extrusion

Brentwood's expertise in PVC extrusion allows the structural columns to be manufactured in house. The column extrusion includes the internal structural ribs required for lateral support.

Quality Control

Brentwood maintains strict quality control in order to ensure that materials and the final product meet design requirments. This quality assurance program includes full material property testing in accordance with American Society for Testing and Materials (ASTM) standards, full-part testing, and process testing in order to quantify product performance during manufacturing. Additionally, Brentwood conducts secondary finshed-part testing to verify that design requirements continue to be met post-manufacturing.

All Module parts are marked with traceability information that allows for tracking of manufacturing. Brentwood maintains equipment at all manufacturing locations, as well as at its corporate testing lab, to ensure all materials and products meet all requirements.

4.0 Structural Response

Structural Design

The Module has been designed to resist loads calculated in accordance with the American Association of State Highway and Transportation Official's (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design manual. This fully factored load includes a multiple presence factor, dynamic load allowance, and live load factor to account for real-world situations. This loading was considered when Brentwood developed both the product and installation requirements. The developed minimum cover ensures the system maintains an adequate resistance factor for the design truck (HS-20) and HS-25 loads.

Full-Scale Product Testing

Engineers at Brentwood's in-house testing facility have completed full-scale vertical and lateral tests on the Module to evaluate product response. To date, Brentwood continues in-house testing in order to evaluate long-term creep effects.

Fully Installed System Testing

Brentwood's dedication to providing a premier product extends to fully installed testing. Through a partnership with Queen's University's GeoEngineering Centre in Kingston, Ontario, Brentwood has conducted full-scale installation tests of single- and double-stacked Module systems to analyze short- and long-term performance. Testing includes short-term ultimate limit state testing under fully factored AASHTO loads and minimum installation cover, lateral load testing, long-term performance and lifecycle testing utilizing time-temperature superposition, and load resistance development. Side backfill material tests were also performed to compare the usage of sand, compacted stone, and uncompacted stone.

5.0 Foundation

The foundation (subgrade) of the subsurface storage structure may be the most important part of the Module system installation as this is the location where the system applies the load generated at the surface. If the subgrade lacks adequate support or encounters potential settlement, the entire system could be adversely affected. Therefore, when implementing an underground storage solution, it is imperative that a geotechnical investigation be performed to ensure a strong foundation.

Considerations & Requirements:

Bearing Capacity

The bearing capacity is the ability of the soil to resist settlement. In other words, it is the amount of weight the soil can support. This is important versus the native condition because the system is replacing earth, and even though the system weighs less than the earth, the additional load displacement of the earth is not offset by the difference in weight.

Using the Loading and Resistance Factor Design (LRFD) calculation for bearing capacity, Brentwood has developed a conservative minimum bearing capacity table (see Appendix). The Engineer of Record shall reference this table to assess actual cover versus the soil bearing required for each unit system.

Soil Profile

Limiting Zones

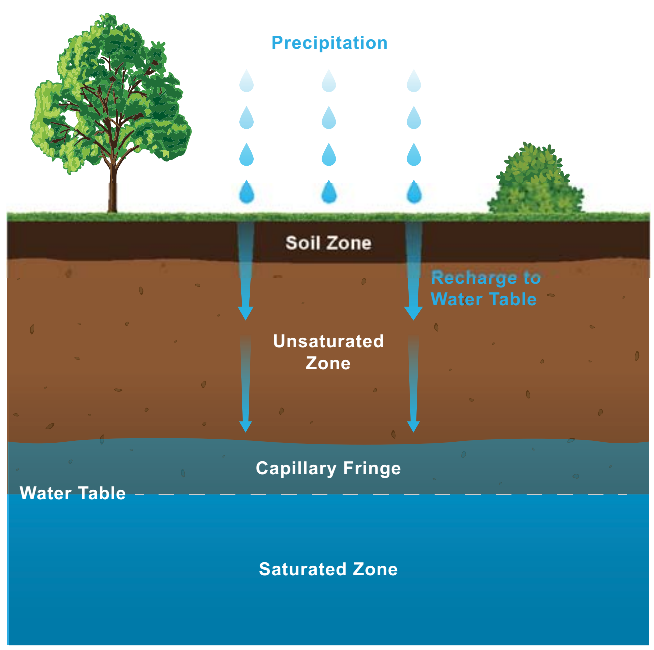
Limiting zones are conditions in the underlying soils that can affect the maximum available depth for installation and can reduce the strength and stability of the underlying subgrade. The three main forms of limiting zones are water tables, bedrock, and karst topography. It is recommended that a system be offset a minimum of 12" (305 mm) from any limiting zones.

Compaction

Soil compaction occurs as the soil particles are pressed together and pore space is eliminated. By compacting the soils to 95% (as recommended by Brentwood), the subgrade strength will increase, in turn limiting both the potential for the soil to move once installed and for differential settlement to occur throughout the system. If designing the specific compaction requirement, settlement should be limited to less than 1" (25 mm) through the entire subgrade and should not exceed a 1/2" (13 mm) of differential settlement between any two adjacent units within the system over time.

Mitigation

If a minimum subgrade bearing capacity cannot be achieved because of weak soil, a suitable design will need to be completed by a Geotechnical Engineer. This design may include the over-excavation of the subgrade and an engineered fill or slurry being placed. Additional material such as geogrid or other products may also be required. Please contact a Geotechnical Engineer prior to selecting products or designing the subgrade.



Water Table Zones

6.0 System Materials

Geotextile Fabric

The 6-ounce geotextile fabric is recommended to be installed between the soil and stone interfaces around the Modules to prevent soil migration.

Leveling Bed

The leveling bed is constructed of 6"-thick (152 mm) angular stone (Table 2). The bed has not been designed as a structural element but is utilized to provide a level surface for the installation of the system and provide an even distribution of load to the subgrade.

Stone Backfill

The stone backfill is designed to limit the strain on the product through displacement of load and ensure the product's longevity. Therefore, a minimum of 12"-wide (305 mm) angular stone must be placed around all sides of the system. In addition, a minimum layer of 12" (305 mm) angular stone is required on top of the system. All material is to be placed evenly in 12" (305 mm) lifts around and on top of the system and aligned with a vibratory plate compactor.

Table 2: Approved Backfill Material

Material Location	Description	AASHTO M43 Designation	ASTM D2321 Class	Compaction/Density
Finished Surface	Topsoil, hardscape, stone, concrete, or asphalt per Engineer of Record	N/A	N/A	Prepare per engineered plans
Suitable Compactable Fill	Well-graded granular soil/aggregate, typically road base or earthen fill (maximum 4" particle size)	56, 57, 6, 67, 68	I & II III (Earth Only)	Place in maximum 12" lifts to a minimum 90% standard proctor density
Top Backfill	Crushed angular stone placed between Modules and road base or earthen fill	56, 57, 6, 67, 68	I & II	Plate vibrate to provide evenly distributed layers
Side Backfill	Crushed angular stone placed between earthen wall and Modules	56, 57, 6, 67, 68	I & II	Place and plate vibrate in uniform 12" lifts around the system
Leveling Bed	Crushed angular stone placed to provide level surface for installation of Modules	56, 57, 6, 67, 68	I & II	Plate vibrate to achieve level surface

Impermeable Liner

In designs that prevent runoff from infiltrating into the surrounding soil (detention or reuse applications) or groundwater from entering the system, an impermeable liner is required. When incorporating a liner as part of the system, Brentwood recommends using a manufactured product such as a PVC liner. This can be installed around the Modules themselves or installed around the excavation (to gain the benefit of the void space in the stone) and should include an underdrain system to ensure the basin fully drains. This liner is installed with a layer of geotextile fabric on both sides to prevent puncture, in accordance with manufacturer recommendations.

7.0 Connections

Stormwater runoff must be able to move readily in and out of the StormTank® Module system. Brentwood has developed numerous means of connecting to the system, including inlet/outlet ports and direct abutment to a catch basin or endwall. All methods of connection should be evaluated as each one may offer a different solution. Brentwood has developed drawings to assist with specific installation methods, and these are available at www.brentwoodindustries.com.

Inlet/Outlet and Pipe Connections

To facilitate easy connection to the system, Brentwood manufactures two inlet/outlet ports. They are 12" (305 mm) and 14" (356 mm), respectfully, and utilize a flexible coupling connection to the adjoining pipe.

Another common installation method is to directly connect the pipe to the system. In order to do this, an opening is cut into the side panels, the pipe is inserted, and then the system is wrapped in geotextile fabric. When utilizing this connection method, the pipe must be located a minimum of 3" (76 mm) from the bottom of the system. This provides adequate clearance for the bottom platen and the required strength in the remaining side panel. To maintain the required clearances or reduce pipe size, it may be necessary to connect utilizing a manifold system.

Direct Abutment

The system can also be connected by directly abutting Modules to a concrete catch basin or endwall. This allows for a seamless connection of structures in close proximity to the system and eliminates the need for numerous pipe connections. When directly abutting one of these structures, remove any side panels that fully abut the structure, and make sure it is flush with the system to prevent material migration into the structure.

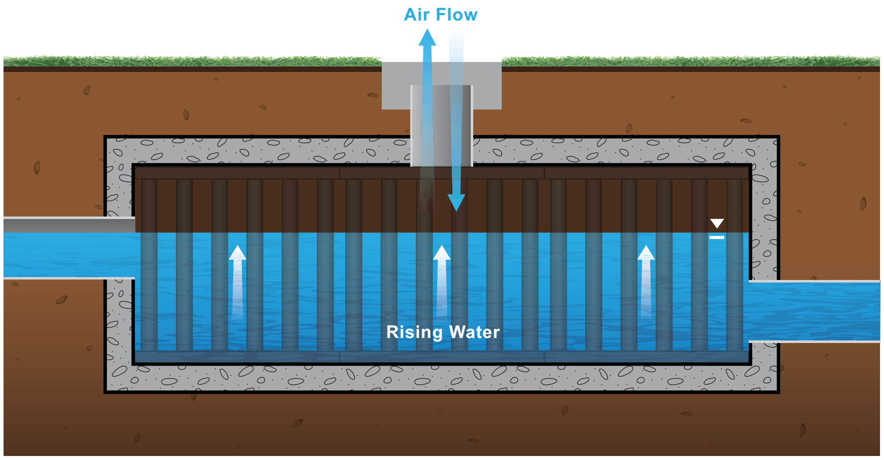
Underdrain

Underdrains are typically utilized in detention applications to ensure the system fully drains since infiltration is limited or prohibited. The incorporation of an underdrain in a detention application will require an impermeable liner between the stone-soil interface.

Cleanout Ports

Brentwood understands the necessity to inspect and clean a subsurface system and has designed the Module without any walls to allow full access. Brentwood offers three different cleanout/observation ports for utilization with the system. The ports are made from PVC, provide an easy means of connection, and are available in 6" (152 mm), 8" (203 mm) and 10" (254 mm) diameters. The 10" (254 mm) port is sized to allow access to the system by a vacuum truck suction hose for easy debris removal.

It is recommended that ports be located a maximum of 30' (9.14 m) on center to provide adequate access, ensure proper airflow, and allow the system to completely fill.



Ventilation and Air Flow

8.0 Pretreatment

Removing pollutants from stormwater runoff is an important component of any stormwater management plan. Pretreatment works to prevent water quality deterioration and also plays an integral part in allowing the system to maintain performance over time and increase longevity. Treatment products vary in complexity, design, and effectiveness, and therefore, should be selected based on specific project requirements.

Typical Stormwater System

Catch Basin → Inlet Piping → Pretreatment → Storage Basin → Outlet Structure

Pretreatment: STORMTANK® Shield

Storage Basin: STORMTANK® Module, STORMTANK® Pack

StormTank® Shield

Brentwood's StormTank Shield provides a low-cost solution for stormwater pretreatment. Designed to improve sumped inlet treatment, the Shield reduces pollutant discharge through gross sediment removal and oil/water separation. For more information, please visit www.brentwoodindustries.com.

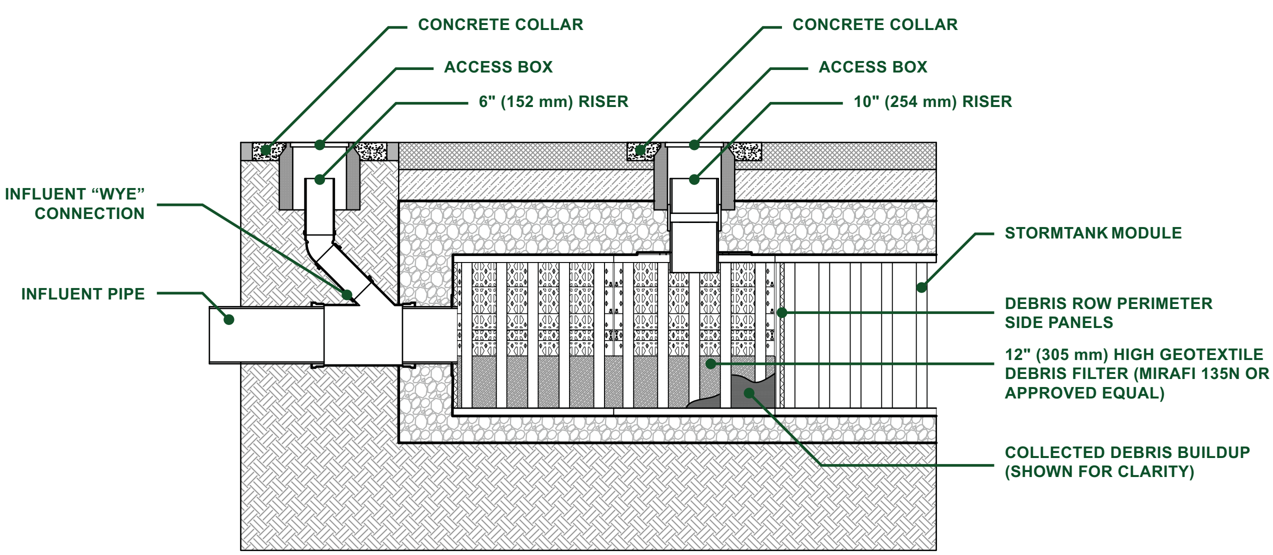
Debris Row (Easy Cleanout)

An essential step of designing, installing, and maintaining a subsurface system is preventing debris from entering the storage. This can be done by incorporating debris rows (or bays) at the inlets of the system to prevent debris from entering the rest of the system.

The debris row is built into the system utilizing side panels with a 12" (305 mm) segment of geotextile fabric. This allows for the full basin capacity to be utilized while storing any debris in an easy-to-remove location. To calculate the number of side panels required to prevent backing up, the opening area of the side panels on the area above the geotextile fabric has been calculated and compared to the inflow pipe diameter.

Debris row cleanout is made easy by including 10" (254 mm) suction ports, based on the length of the row, and a 6" (152 mm) saddle connection to the inflow pipe. If the system is directly abutting a catch basin, the saddle connection is not required, and the flush hose can be inserted through the catch basin. Debris is then flushed from the inlet toward the suction ports and removed.

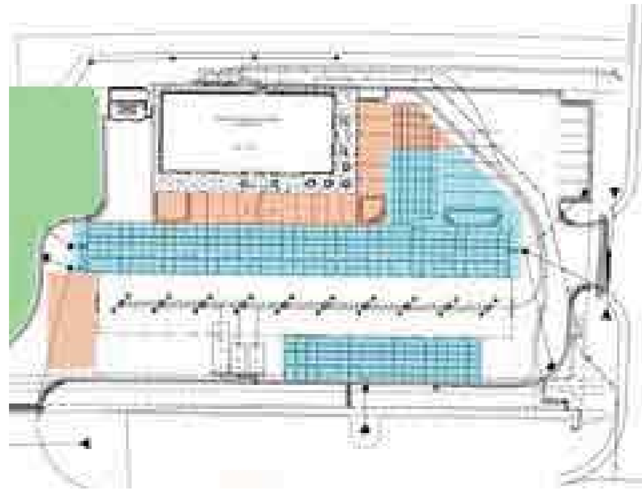
Brentwood has developed drawings and specifications that are available at www.brentwoodindustries.com to illustrate the debris row configuration and layouts.



Debris Row Section Detail

9.0 Additional Considerations

Many variable factors, such as the examples below, must be taken into consideration when designing a StormTank® Module system. As these considerations require complex calculations and proper planning, please contact Brentwood or your local distributor to discuss project-specific requirements.



Site Plan Module Layout Adaptability
(StormTank Modules shown in blue)

Adaptability

The Modules can be arranged in custom configurations to meet tight site constraints and to provide different horizontal and edge configurations. Modules can also be stacked, to a maximum 2 units tall, to meet capacity needs and can be buried to a maximum invert of 11' (3.35 m) to allow for a stacked system or deeper burial.

Adjacent Structures

The location of adjacent structures, especially the location of footings and foundations, must be taken into consideration as part of system design. The foundation of a building or retaining wall produces a load that is transmitted to a footing and then applied to the surface below. The footing is intended to distribute the line load of the wall over a larger area without increasing the larger wall's thickness. The reason this is important is because the load the footing is applying to the earth is distributed through the earth and could potentially affect a subsurface system as either a vertical load to the top of the Module or a lateral load to the side of the Module.

Based on this increased loading, it is recommended that the subsurface system either maintain a distance away from the foundation, footing equal to the height between the Module invert and structure invert of the system, or the foundation or footing extend at a minimum to the invert of the subsurface system. By locating the foundation away from the system or equal to the invert, the loading generated by the structure does not get transferred onto the system. It is recommended that all adjacent structures be completed prior to the installation of the Modules to prevent construction loads from being imparted on the system.

Adjacent Excavation

The subsurface system must be protected before, during, and after the installation. Once a system is installed, it is important to remember that excavation adjacent to the system could potentially cause the system to become unstable. The uniform backfilling will evenly distribute the lateral loads to the system and prohibit the system from becoming unstable and racking from unequal loads. However, it is recommended that any excavation adjacent to a system remain a minimum distance away from the system equal to the invert. This will provide a soil load that is equal to the load applied by the opposite side of the installation. If the excavation is to exceed the invert of the system, additional analysis may be necessary.

Sloped Finished Grade

Much like adjacent excavation, a finished grade with a differential cover could potentially cause a subsurface system to become disproportionately loaded. For example, if one side of the system has 10' (3.05 m) of cover and the adjacent side has 24" (610 mm) of cover, the taller side will generate a higher lateral load, and the opposite side may not have an equal amount of resistance to prevent a racking of the system. Additional evaluation may be required when working on sites where the final grade around a system exceeds 5%.

10.0 Inspection & Maintenance

Description

Proper inspection and maintenance of a subsurface stormwater storage system are vital to ensuring proper product functioning and system longevity. It is recommended that during construction the contractor takes the necessary steps to prevent sediment from entering the subsurface system. This may include the installation of a bypass pipe around the system until the site is stabilized. The contractor should install and maintain all site erosion and sediment per Best Management Practices (BMP) and local, state, and federal regulations.

Once the site is stabilized, the contractor should remove and properly dispose of erosion and sediment per BMP and all local, state, and federal regulations. Care should be taken during removal to prevent collected sediment or debris from entering the stormwater system. Once the controls are removed, the system should be flushed to remove any sediment or construction debris by following the maintenance procedure outlined below.

During the first service year, a visual inspection should be completed during and after each major rainfall event, in addition to semi-annual inspections, to establish a pattern of sediment and debris buildup. Each stormwater system is unique, and multiple criteria can affect maintenance frequency. For example, whether or not a system design includes inlet protection or a pretreatment device has a substantial effect on the system's need for maintenance. Other factors include where the runoff is coming from (hardscape, gravel, soil, etc.) and seasonal changes like autumn leaves and winter salt.

During and after the second year of service, an established annual inspection frequency, based on the information collected during the first year, should be followed. At a minimum, an inspection should be performed semi-annually. Additional inspections may be required at the change of seasons for regions that experience adverse conditions (leaves, cinders, salt, sand, etc).

Maintenance Procedures

Inspection:

1. Inspect all observation ports, inflow and outflow connections, and the discharge area.
2. Identify and log any sediment and debris accumulation, system backup, or discharge rate changes.
3. If there is a sufficient need for cleanout, contact a local cleaning company for assistance.

Cleaning:

1. If a pretreatment device is installed, follow manufacturer recommendations.
2. Using a vacuum pump truck, evacuate debris from the inflow and outflow points.
3. Flush the system with clean water, forcing debris from the system.
4. Repeat steps 2 and 3 until no debris is evident.

11.0 System Sizing

System Sizing Calculation

This section provides a brief description of the process required to size the StormTank® Module system. If you need additional assistance in determining the required number of Modules or assistance with the proposed configuration, it is recommended that you contact Brentwood or your local distributor. Additionally, Brentwood's volume calculator can help you to estimate the available storage volumes with and without stone storage. This tool is available at www.brentwoodindustries.com.

1. Determine the required storage volume (Vs):
It is the sole responsibility of the Engineer of Record to calculate the storage volume in accordance with all local, state, and federal regulations.

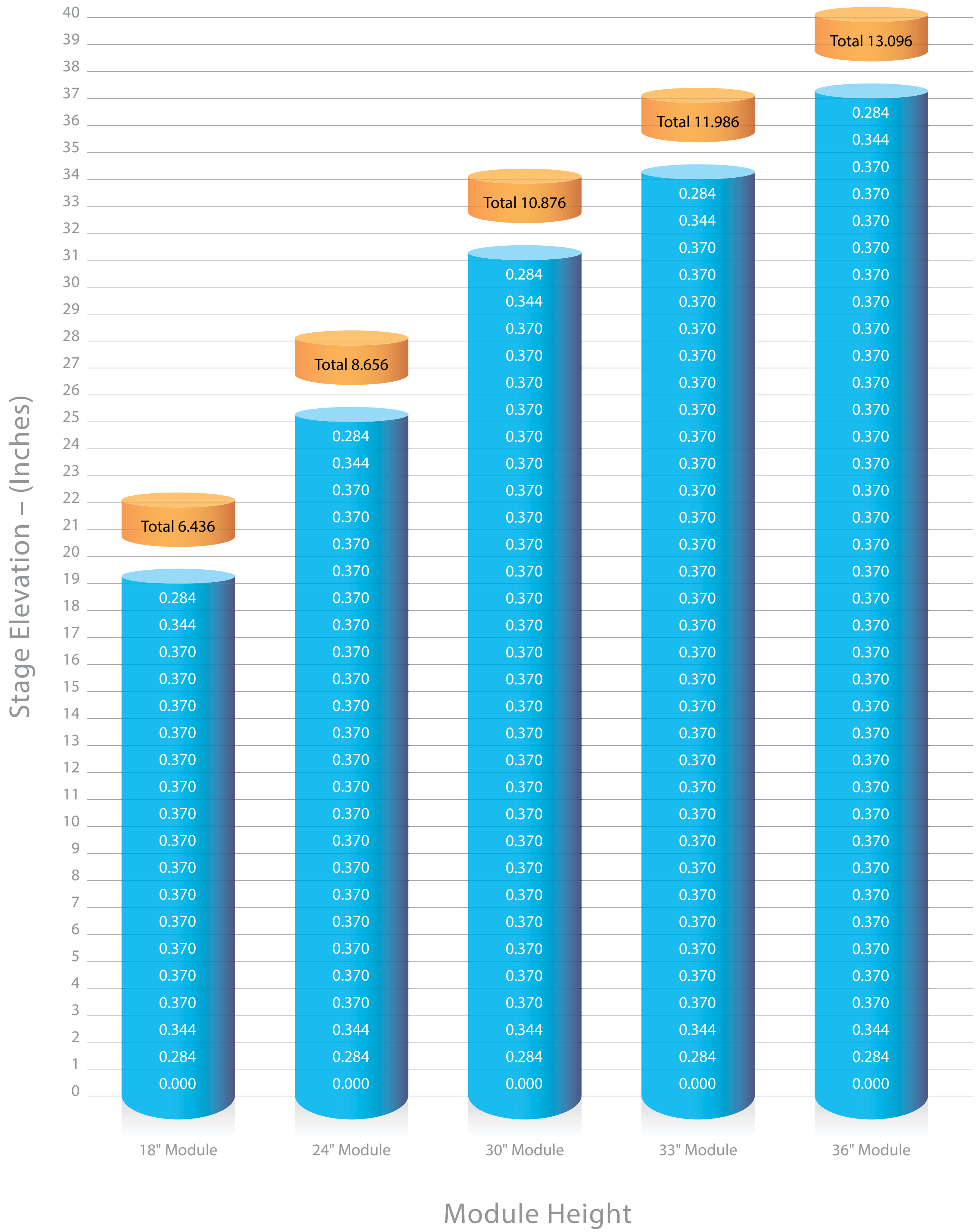
2. Determine the required number of Modules (N):
If the storage volume does not include stone storage, take the total volume divided by the selected Module storage volume. If the stone storage is to be included, additional calculations will be required to determine the available stone storage for each configuration.

3. Determine the required volume of stone (Vstone):
The system requires a minimum 6" (152 mm) leveling bed, 12" (305 mm) backfill around the system, and 12" (305 mm) top backfill utilizing 3/4" (19 mm) angular clean stone. Therefore, take the area of the system times the leveling bed and the top backfill. Once that value is determined, add the volume based on the side backfill width times the height from the invert of the Modules to the top of the Modules.

4. Determine the required excavation volume (Vexcv):
Utilizing the area of the system, including the side backfill, multiply by the depth of the system including the leveling bed. It is noted that this calculation should also include any necessary side pitch or benching that is required for local, state, or federal safety standards.

5. Determine the required amount of geotextile (G):
The system utilizes a multiple layer system of geotextile fabric. Therefore, two calculations are required to determine the necessary amount of geotextile. The first layer surrounds the entire system (including all backfill), and the second layer surrounds the Module system only. It is recommended that an additional 20% be included for waste and overlap.

11.1 Storage Volume



11.2 Material Quantity Worksheet

Project Name:	By:
Location:	**Date:**

System Requirements

Required Storage		ft^3 (m^3)
Number of Modules		Each
Module Storage		ft^3 (m^3)
Stone Storage		ft^3 (m^3)
Module Footprint		ft^2 (m^2) Number of Modules x 4.5 ft^2 (0.42 m^2)
System Footprint w/ Stone		ft^2 (m^2) Module Footprint + 1 ft (0.3048 m) to each edge
Stone		Tons (kg) Leveling Bed + Side Backfill + Top Backfill
Volume of Excavation		yd^3 (m^3) System Footprint w/ Stone x Total Height
Area of Geotextile		yd^2 (m^2) Wrap around Modules + Wrap around Stone/Soil Interface

System Cost

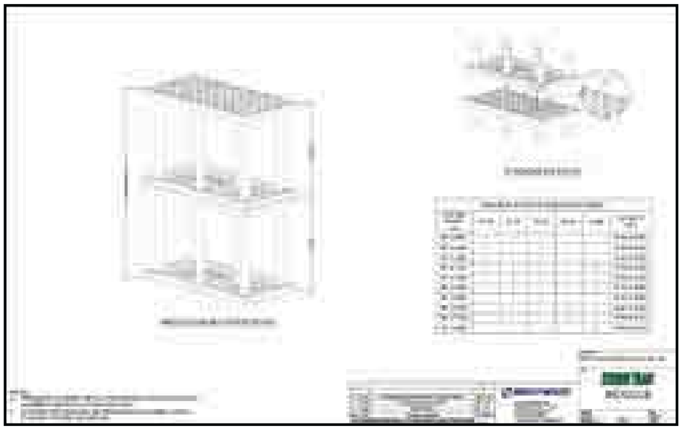
	Quantity			Unit Price			Total
Modules		ft^3 (m^3)	X	$	ft^3 (m^3)	=	$
Stone		Tons (kg)	X	$	Tons (kg)	=	$
Excavation		yd^3 (m^3)	X	$	yd^3 (m^3)	=	$
Geotextile		yd^2 (m^2)	X	$	yd^2 (m^2)	=	$
					Subtotal =		$
					Tons =		$

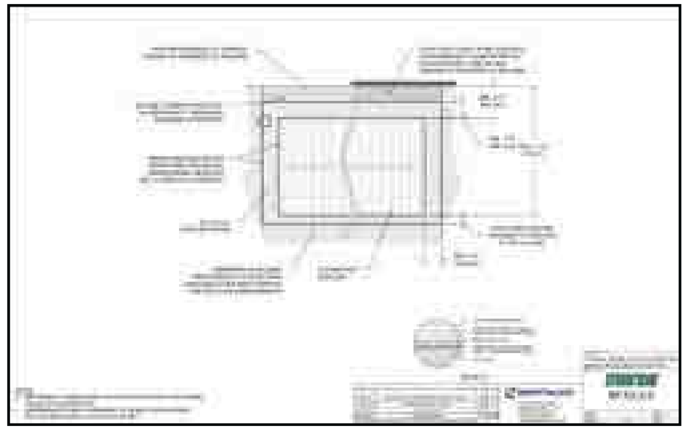
Material costs may not include freight.

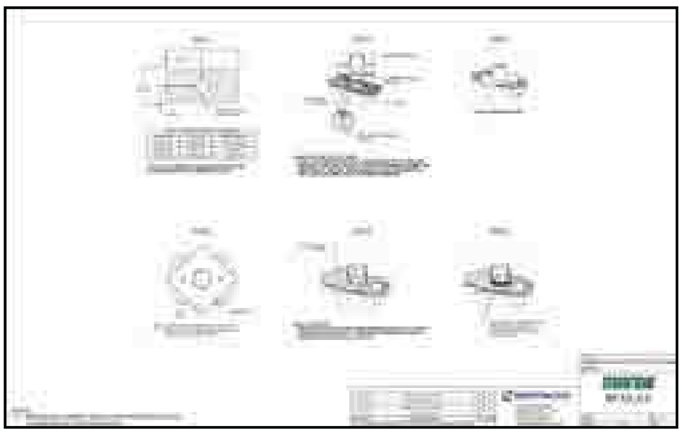
Please contact Brentwood or your local distributor for this information.

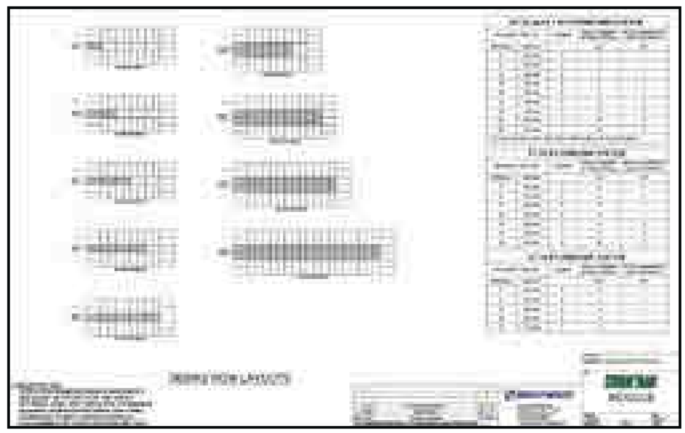
12.0 Detail Drawings

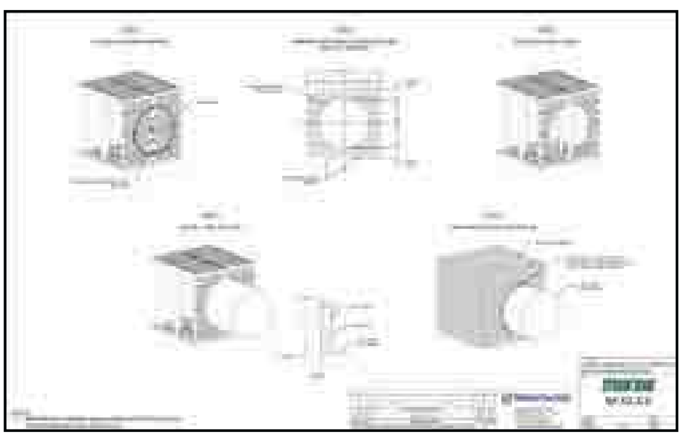
Brentwood has developed numerous drawings for utilization when specifying a StormTank® Module system. Below are some examples of drawings available at www.brentwoodindustries.com.

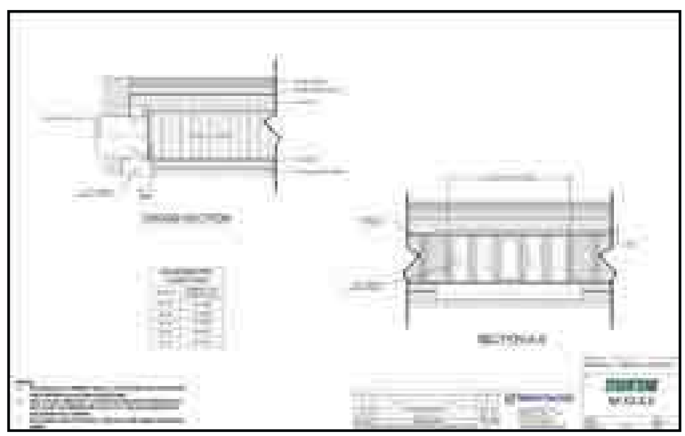












13.0 Specifications

1) General

a) This specification shall govern the implementation, performance, material, and fabrication pertaining to the subsurface stormwater storage system. The subsurface stormwater storage system shall be manufactured by Brentwood Industries, Inc., 500 Spring Ridge Drive, Reading, PA 19610 (610.374.5109), and shall adhere to the following specification at the required storage capacities.

b) All work is to be completed per the design requirements of the Engineer of Record and to meet or exceed the manufacturer's design and installation requirements.

2) Subsurface Stormwater Storage System Modules

a) The subsurface stormwater storage system shall be constructed from virgin polypropylene and 100% recycled PVC to meet the following requirements:

i) High-Impact Polypropylene Copolymer Material

(1) Injection molded, polypropylene, top/bottom platens and side panels formed to a dimension of 36" (914 mm) long by 18" (457 mm) wide [nominal].

ii) 100% Recycled PVC Material

(1) PVC conforming to ASTM D-1784 Cell Classification 12344 b-12454 B.

(2) Extruded, rigid, and 100% recycled PVC columns sized for applicable loads as defined by Section 3 of the AASHTO LRFD Bridge Design Specifications and manufactured to the required length per engineer-approved drawings.

iii) Platens and columns are assembled on site to create Modules, which can be uniformly stacked up to two Modules high, in vertical structures of variable height (custom for each project).

iv) Modular stormwater storage units must have a minimum 95% void space and be continuously open in both length and width, with no internal walls or partitions.

3) Submittals

a) Only systems that are approved by the engineer will be allowed.

b) At least 10 days prior to bid, submit the following to the engineer to be considered for pre-qualification to bid:

i) A list of materials to be provided for work under this article, including the name and address of the materials producer and the location from which the materials are to be obtained.

ii) Three hard copies of the following:

(1) Shop drawings.

(2) Specification sheets.

(3) Installation instructions.

(4) Maintenance guidelines.

c) Subsurface Stormwater Storage System Component Samples for review:

i) Subsurface stormwater storage system Modules provide a single 36" (914 mm) long by 18" (457 mm) wide, height as specified, unit of the product for review.

ii) Sample to be retained by owner.

d) Manufacturers named as acceptable herein are not required to submit samples.

4) Structural Design

a) The structural design, backfill, and installation requirements shall ensure the loads and load factors specified in the AASHTO LRFD Bridge Design Specifications, Section 3 are met.

b) Product shall be tested under minimum installation criteria for short-duration live loads that are calculated to include a 20% increase over the AASHTO Design Truck standard with consideration for impact, multiple vehicle presences, and live load factor.

c) Product shall be tested under maximum burial criteria for long-term dead loads.

d) The engineer may require submission of third-party test data and results in accordance with items 4b and 4c to ensure adequate structural design and performance.

14.0 Appendix - Bearing Capacity Tables

Cover		HS-25 (Unfactored)		HS-25 (Factored)	
English (in)	Metric (mm)	English (ksf)	Metric (kPa)	English (ksf)	Metric (kPa)
24	610	1.89	90.45	4.75	227.43
25	635	1.82	86.96	4.53	216.90
26	660	1.75	83.78	4.34	207.80
27	686	1.69	80.88	4.16	199.18
28	711	1.63	78.24	3.99	191.04
29	737	1.58	75.82	3.84	183.86
30	762	1.54	73.62	3.70	177.16
31	787	1.50	71.60	3.57	170.93
32	813	1.46	69.75	3.45	165.19
33	838	1.42	68.06	3.34	159.92
34	864	1.39	66.51	3.24	155.13
35	889	1.36	65.10	3.14	150.34
36	914	1.33	63.80	3.05	146.03
37	940	1.31	62.62	2.97	142.20
38	965	1.29	61.54	2.90	138.85
39	991	1.26	60.55	2.83	135.50
40	1,016	1.25	59.65	2.76	132.15
41	1,041	1.23	58.54	2.70	129.28
42	1,067	1.21	58.09	2.67	127.84
43	1,092	1.20	57.42	2.60	124.49
44	1,118	1.19	56.81	2.55	122.09
45	1,143	1.18	56.26	2.50	119.70
46	1,168	1.16	55.77	2.46	117.79
47	1,194	1.16	55.33	2.42	115.87
48	1,219	1.15	54.94	2.39	114.43
49	1,245	1.14	54.59	2.36	113.00
50	1,270	1.13	54.29	2.33	111.56
51	1,295	1.13	54.03	2.30	110.12
52	1,321	1.12	53.80	2.27	108.69
53	1,346	1.12	53.62	2.25	107.73
54	1,372	1.12	53.46	2.23	106.77
55	1,397	1.11	53.34	2.21	105.82
56	1,422	1.11	53.24	2.19	104.86
57	1,448	1.11	53.18	2.17	103.90
58	1,473	1.11	53.14	2.16	103.42
59	1,499	1.11	53.12	2.14	102.46
60	1,524	1.11	53.13	2.13	101.98
61	1,549	1.11	53.16	2.12	101.51
62	1,575	1.11	53.21	2.11	101.03
63	1,600	1.11	53.28	2.10	100.55
64	1,626	1.11	53.37	2.09	100.07
65	1,651	1.12	53.48	2.08	99.59
66	1,676	1.12	53.61	2.08	99.59
67	1,702	1.12	53.75	2.07	99.11
68	1,727	1.13	53.91	2.07	99.11
69	1,753	1.13	54.08	2.06	98.63

Cover		HS-25 (Unfactored)		HS-25 (Factored)	
English (in)	Metric (mm)	English (ksf)	Metric (kPa)	English (ksf)	Metric (kPa)
70	1,778	1.13	54.26	2.06	98.63
71	1,803	1.14	54.46	2.06	98.63
72	1,829	1.14	54.67	2.06	98.63
73	1,854	1.15	54.90	2.06	98.63
74	1,880	1.15	55.13	2.06	98.63
75	1,905	1.16	55.38	2.06	98.63
76	1,930	1.16	55.64	2.06	98.63
77	1,956	1.17	55.90	2.06	98.63
78	1,981	1.17	56.18	2.06	98.63
79	2,007	1.18	56.46	2.07	99.11
80	2,032	1.19	56.76	2.07	99.11
81	2,057	1.19	57.06	2.07	99.11
82	2,083	1.20	57.37	2.08	99.59
83	2,108	1.20	57.69	2.08	99.59
84	2,134	1.21	58.02	2.09	100.07
85	2,159	1.22	58.35	2.09	100.07
86	2,184	1.23	58.69	2.10	100.55
87	2,210	1.23	59.04	2.11	101.03
88	2,235	1.24	59.39	2.11	101.03
89	2,261	1.25	59.75	2.12	101.51
90	2,286	1.26	60.11	2.13	101.98
91	2,311	1.26	60.48	2.13	101.98
92	2,337	1.27	60.86	2.14	102.46
93	2,362	1.28	61.24	2.15	102.94
94	2,388	1.29	61.62	2.16	103.42
95	2,413	1.30	62.01	2.17	103.90
96	2,438	1.30	62.41	2.18	104.38
97	2,464	1.31	62.81	2.19	104.86
98	2,489	1.32	63.21	2.20	105.34
99	2,515	1.33	63.62	2.21	105.82
100	2,540	1.34	64.03	2.22	106.29
101	2,565	1.35	64.45	2.23	106.77
102	2,591	1.35	64.87	2.24	107.25
103	2,616	1.36	65.29	2.25	107.73
104	2,642	1.37	65.72	2.27	108.69
105	2,667	1.38	66.15	2.28	109.17
106	2,692	1.39	66.58	2.29	109.65
107	2,718	1.40	67.02	2.30	110.12
108	2,743	1.41	67.45	2.31	110.60
109	2,769	1.42	67.90	2.33	111.56
110	2,794	1.43	68.34	2.34	112.04
111	2,819	1.44	68.79	2.35	112.52
112	2,845	1.45	69.24	2.36	113.00
113	2,870	1.46	69.69	2.38	113.96
114	2,896	1.47	70.15	2.39	114.43

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Appendix C: Sanitary Sewer Calculation Sheet and Water Pressure Loss Calculation Sheet

Sanitary Sewer Design Calculations

6173 Renaud Road, City Of Ottawa, Ontario

Location			Residential Flow								Commercial/Institutional			Infiltration		Flow	Sanitary Sewer Design						
STREET	From	To	No. of Single Dwellings	No. of Row/Semi Dwellings	Pop.	Area, A	Cumulative Pop.	Cumulative Area	Peaking Factor	Res. Flow, $Q_{(p)}$	Area	Tributary Area, A	Com. Flow, $Q_{(p)}$	Total Tributary Area	Infiltration Flow	Peak Design Flow	Length, L	Diameter, d_{nom}*	Slope, s	Pipe Capacity, Q_f	Full Flow Velocity, v_f	Design peak Velocity Vp	Percent of Capacity
	MH	MH			[no.]	[ha]	[no.]	[ha]		[L/s]	[ha]	[Sq.m]	[L/s]	[ha]	[L/s]	[L/s]	[m]	[mm]	[%]	[L/s]	[m/s]	[m/s]	[%]
After Development																							
Trailsedge	Upstream	6173	NA	350	945	9.20	945	9.20	3.25	9.95	0.00	0.00	0.000	9.20	3.04	12.99		200	0.33%	18.84	0.60	0.64	68.9%
Building 1	Building	SANMH1			38	0.17	38	0.17	3.67	0.45	0.00	0.00	0.000	0.17	0.05	0.51	52	200	2.53%	52.17	1.66	0.40	1.0%
Building 2	Building	SANMH2			38	0.17	76	0.33	3.62	0.89	0.00	0.00	0.000	0.33	0.11	1.00	13	200	2.53%	52.17	1.66	0.56	1.9%
Contour Street				100	270	2.80	270	2.80	3.48	3.04	0.00	0.00	0.000	2.80	0.92	3.97		200	0.33%	18.84	0.60	0.48	21.1%
Trailsedge	Down stream	Trunk Sewer					1291	12.33	3.18	13.31	0.00	0.00	0.000	12.33	4.07	17.38		300	0.50%	68.38	0.97	0.81	25.4%

Notes:

Q = Average daily flow per capita — 280 L/day per capita

$Q_{ext.}$ = Unit peak extraneous flow — 0.33 L/s per gross ha.

Pop. Single Family — 3.4 Persons

Pop. Semi-Detached & Row House — 2.7 Persons

Estimated Residential Density — 38 Row house Unit/ha

Project: Teak Developments

Location 6173 Renuad Road
City Of Ottawa, Ontario

Design by: SD **Date:** June 30, 2020

Checked by: SD **Rev.** 0

Min Velocity of flow > 0.6m/s

Max Velocity of flow > 3m/s

Kollaard Associates File #: 190867

APPENDIX C: WATER PRESSURE LOSS CALCULATION SHEET

Client: Teak Developments
Job No.: 190867
Location: 6173 Renaud Road, Ottawa
Date: April 13, 2022

Average Daily Water Demand		0.230 L/s	0.000230 m^3/s		13.8 L/min
Max Daily Demand		0.580 L/s	0.000580 m^3/s		34.8 L/min
Max Hourly Demand		1.270 L/s	0.001270 m^3/s		76.2 L/min
Fire demand		90 L/s	0.090000 m^3/s		5400 L/min
	Water Density	999.7 kg/m3			
g	Gravity	9.806 m/s2			
S		9.8030582 kN/m2			
v	=	1.31E-06	$[m^2/s]$	Kinematic Viscosity of Water @ 10˚ C	
Roughness Factor		0.0015 mm			

Water Flow Analysis

Pipe Sections			Grade Elevation		Hydraulic Grade line							
Start	Along	End	Start m	End* m	Start** m	End m	Ps kPa	Pe kPa	Q m^3/sec	V m/sec	D m	A m^2
Calculation of Available Pressure Using 50 mm Diameter Pipe Starting at Minimum HGL and Max Hourly Demand												
Trailsedge Way	Service	3 Storey Residential	84.9	88.15	126.5	126.3	408	374	0.0013	0.647	0.05	0.0020
Trailsedge Way	Service	3 Storey Residential	84.9	94.5	126.5	126.3	408	312	0.0013	0.647	0.05	0.0020
Calculation of Available Pressure Using 100 mm Diameter Pipe Starting at Minimum HGL and Max Hourly Demand												
Trailsedge Way	Service	3 Storey Residential	84.9	88.15	126.5	126.5	408	376	0.0013	0.162	0.10	0.0079
Trailsedge Way	Service	3 Storey Residential	84.9	94.5	126.5	126.5	408	314	0.0013	0.162	0.10	0.0079
Calculation of Maximum Pressure Using 100 mm Diameter Pipe Resulting From Maximum HGL and Average Daily Flow Demand												
Trailsedge Way	Service	3 Storey Residential	84.9	88.15	130.6	130.6	448	416	0.0002	0.029	0.10	0.0079
Trailsedge Way	Service	3 Storey Residential	84.9	94.5	130.6	130.6	448	354	0.0002	0.029	0.10	0.0079
Calculation of Available Pressure Using 100 mm Diameter Pipe Starting at Minimum HGL and Average Daily Flow Demand												
Trailsedge Way	Service	3 Storey Residential	84.9	88.15	119.5	119.5	339	307	0.0002	0.029	0.10	0.0079
Trailsedge Way	Service	3 Storey Residential	84.9	94.5	119.5	119.5	339	245	0.0002	0.029	0.10	0.0079

Start Elevation Corresponds to Approximate Elevation Street = 84.9 metres.
*End Elevation Correspond as follows: 88.15- Ground Floor
94.5- Fixtures in 3rd floor

Ps	Pressure at Start	= (HGL - Start Elevation) x Specific Gravity of Water
Pe	Pressure at End	= (HGL - End Elevation) x Specific Gravity of Water
Q	Flow Rate	
V	Flow Velocity	
D	Pipe Diameter	
A	Pipe Area	

Kollaard Associates
Engineers

Appendix D: Fire Flow Calculations and Boundary Conditions

- Fire Flow Requirements – FUS (Technical Bulletin ISTB-2018-02)

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Kemptville, Ontario K0G 1J0

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Structural • Environmental •
Materials Testing •

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FAX: (613) 258-0475

January 10, 2020

Mike Thivierge P.Eng., PE
Sr. Engineer, Development Review East Branch
Planning Infrastructure & Economic Development Department
Planning Services.

Re: Boundary Conditions 6173 Renaud Road

Kollaard Associates Inc has been retained by Mr. George Elias to complete the Site Servicing Plan and Site Servicing Report for the proposed residential development at 6173 Renaud Road in the City of Ottawa.

Could you provide us with the boundary conditions for the property based on the following information:

Type of Development: Residential (Two 4-storey, 16-unit apartment buildings)
Location of Services: Trailsedge Way
Amount of Fire Flow: 166.7 L/s (See attached fire flow requirements)
Average daily water demand: 0.40 L/s
Maximum daily water demand: 1.00 L/s
Maximum Hourly water demand: 2.21 L/s
Peak sanitary flow: 1.27 L/s

Please note:

- The sanitary calculations have been completed using Technical Bulletin ISTB-2018-01. The water demand calculations have not been updated to reflect the changes in sanitary demand calculations.
- Fire flow is based on FUS calculations and takes into account the methodology provided in Technical Bulletin ISTB-2018-02
- Design calculation spread sheets for FUS, Water and Sanitary are attached
- Servicing Sketch is attached showing proposed connection location

If there are any questions related to the above please contact the undersigned.

Sincerely,
KOLLAARD ASSOCIATES INC.

Steven deWit, P.Eng.

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Kemptville, Ontario K0G 1J0

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Hydrogeology

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APPENDIX C: CALCULATION OF FIRE FLOW REQURIEMENTS - 854 Grenon Avenue

Calculation Based on Fire Underwriters Survey, 1999 and Ottawa Technical Bulletin ISTB-2018-02

Proposed Building:

Two 4 storey wood frame 16-unit residential buildings.

1) An estimate of the Fire Flow required for a given fire area may be estimated by:

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

where F = required fire flow in litres per minute

A = total floor area in m^2 (including all storeys, but excluding basements at least 50% below grade)

C = coefficient related to the type of construction:

- 1.5 for wood construction (structure essentially combustible)
- 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
- 0.8 for noncombustible construction (unprotected metal structural components, masonary or metal walls)
- 0.6 for fire-resistive construction (fully protected frame, floors, roof)

No. of Floors = 3 (FUS excludes basements that are at least 50% below grade)

Area (per floor) = 400 m^2

A = 1200 m^2

C = 1.5

F = 11,432 L/min -----------------------> Rounded to nearest 1000 = **11,000** L/min

2) The value obtained in 1) may be reduced by as much as 25% for occupancies having a low

Non-combustible =	-25%
Limited Combustible =	-15%
Combustible =	0%
Free Burning =	15%
Rapid Burning =	25%

Reduction due to low occupancy hazard = -15% x 11,000 = **9,350** L/min

3) The value above my be reduced by up to 50% for automatic sprinlker system

Reduction due to automatic sprinker system = 0% x 9,350 = **0** L/min

4) The value obtained in 2. may be increased for structures exposed within 45 metres by the fire

Separation (metres)	Condtion	Max Charge*
0m to 3.0m	1	25%
3.1m to 10.0m	2	20%
10.1m to 20.0m	3	15%
20.1m to 30.0m	4	10%
30.1m to 45.0m	5	5%
45.1m to	6	0%

Charge for separation has been modified by Technical Bulletin ISTB-2018-02 based on construction and Lenght-Height Factor

Lenght*Height (L * H) = Exposed wall length in feet x height of building in stories

No of Stories = 3

Exposures	Distance(m)	Length (ft)	L * H	Condition		Charge
Back (north)	35.2	84	252	5	-------->	5%
Front (south)	24.6	84	252	4	-------->	10%
Side 1 (west)	18.1	51	153	3	-------->	13%
Side 2 (east)	9.0	51	153	2	-------->	18%
						46%

Increase due to separation = 46% x 9,350 = **4,301** L/min

The fire flow requirement is = **9,350**

Reduction due to Sprinkler = **0**

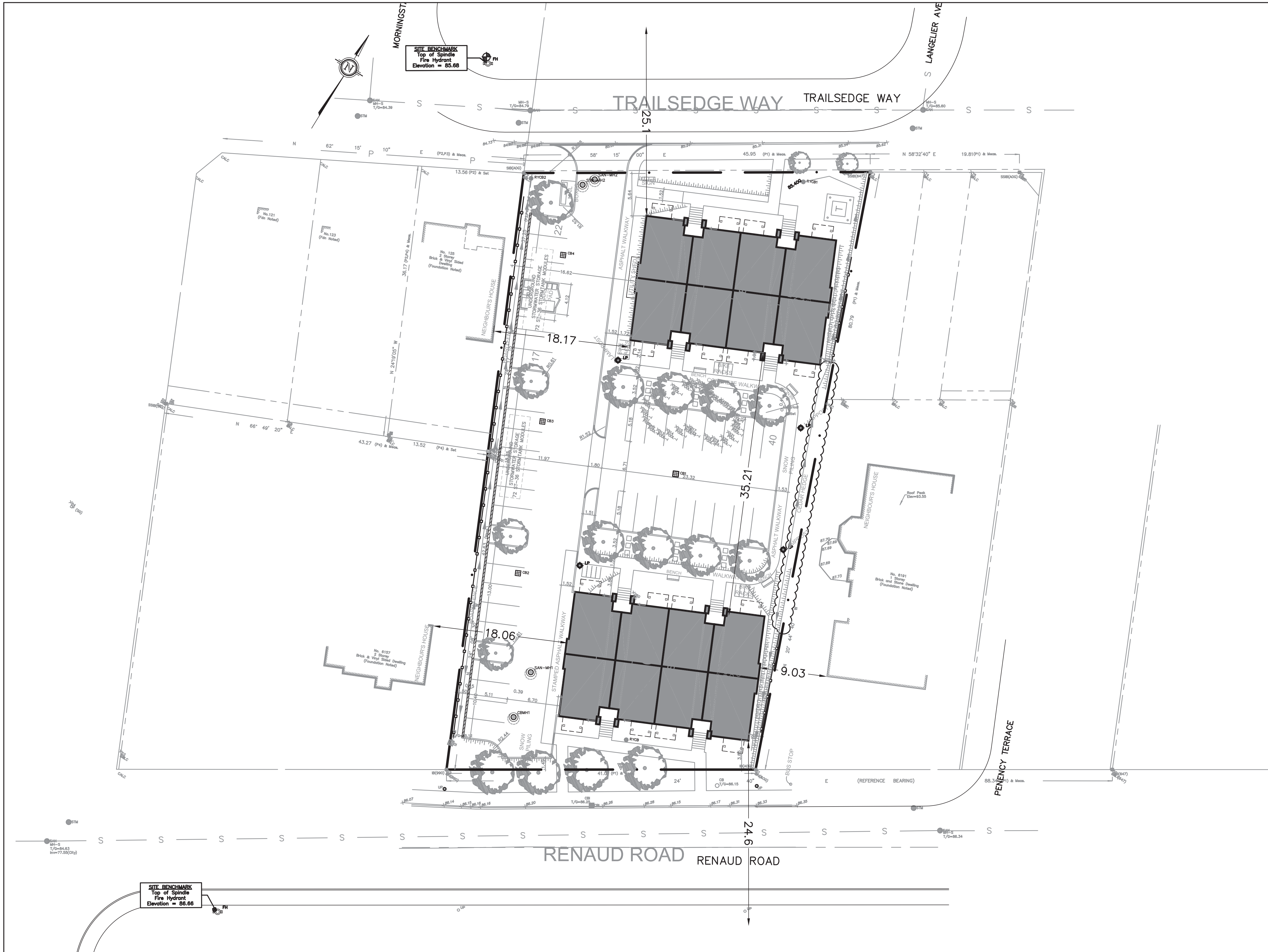
Increase due to Separation = 4,301

13,651

City of Ottawa Cap = 10,000 L/min

The Total fire flow requirement is = 10,000 L/min

or **166.7** **L/sec**



DRAWING NUMBER:
190867–FUS

FUS EXPOSURE DISTANCES

REV	BY	DATE	DESCRIPTION

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http://www.kollaard.ca
(613) 860–0923
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CLIENT:
TEAK DEVELOPMENTS
31 WOODVIEW CRESCENT
OTTAWA, ON K1B 3B1

PROJECT:
PROPOSED RESIDENTIAL
DEVELOPMENT

LOCATION:
6173 RENAUD ROAD
CITY OF OTTAWA, ON
K1W 0K9

DESIGNED BY: SD	*DATE:* MAY 20, 2020
DRAWN BY: ML	*SCALE:* AS NOTED

KOLLAARD FILE NUMBER:
190867

Boundary Conditions
6173 Renaud Road

Provided Information

Date Provided January-20

Scenario	Demand	
	L/min	L/s
Average Daily Demand	24	0.40
Maximum Daily Demand	60	1.00
Peak Hour	76	1.27
Fire Flow Demand #1	10,000	166.67

Location

Results

Connection 1 – Trailsedge Way

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	130.6	64.8
Peak Hour	126.5	58.9
Max Day plus Fire 1	119.5	48.9

[1] Ground Elevation = 85.1 m

Notes:

1. Providing a second connection on Renaud Road is required to decrease vulnerability of the water system in case of breaks.

Disclaimer

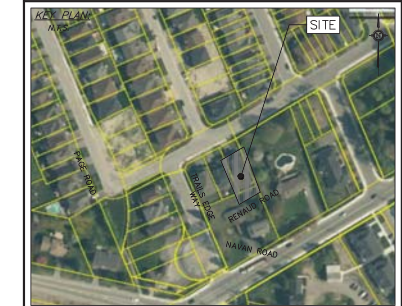
The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.

Appendix E: Drawings

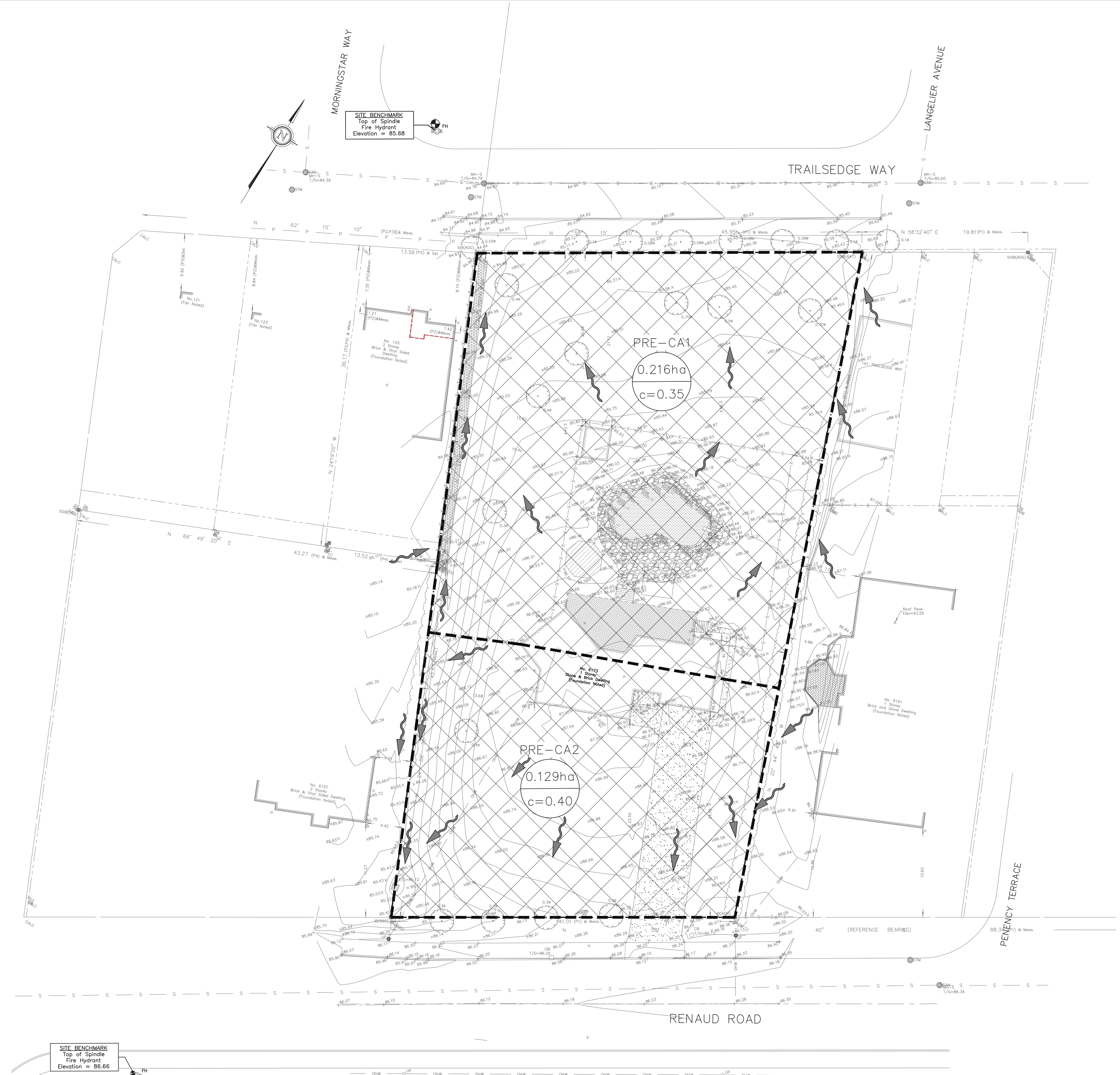
190867– PRE – PRE-DEVELOPMENT DRAINAGE
190867– POST – POST-DEVELOPMENT DRAINAGE
190867– SER – Site Servicing Plan
190867– GRD – Site Grading and Erosion Control Plan
190867– DET – Details
Plan of Survey

DRAWING NUMBER:

190867–PRE



SCALE:



PRE–DEVELOPMENT DRAINAGE

SCALE = 1:250

LEGEND (STORM WATER MANAGEMENT)

- CONTROLLED — CATCHMENT LABEL
- 0.39 — CATCHMENT AREA (HECTARES)
- c=0.34 — RUNOFF COEFFICIENT
- CATCHMENT AREA BOUNDARY
- DIRECTION OF FLOW
- PROPERTY LINE
- TOP OF SLOPE
- CONTROLLED AREA
- UNCONTROLLED AREA
- PRE–DEVELOPMENT DRAINAGE PATTERN

No.	REVISION	DATE	BY
4	REVISIONS PER NEW SITE PLAN AND REVIEW COMMENTS	APR. 13/2022	ML
3	ISSUED FOR SPC RE–SUBMISSION	APR. 22/2021	ML
2	REVISIONS PER SITE PLAN AND 1ST REVIEW COMMENTS	MAR. 29/2021	ML
1	ISSUED FOR SPC APPLICATION	JUNE 30/2020	ML
#	REVISION ITEM / DESCRIPTION	REV. DATE	INT.

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http://www.kollaard.ca

CLIENT:

TEAK DEVELOPMENTS
31 WOODVIEW CRESCENT
OTTAWA, ON K1B 3B1

PROJECT:

PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:

6173 RENAUD ROAD
CITY OF OTTAWA, ON
K1W 0K9

LICENSED PROFESSIONAL ENGINEER
APRIL 13, 2022
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGNED BY:	CHECKED BY:
––	––
DRAWN BY:	APPROVED BY:
––	––

DATE: NOV. 11, 2019

KOLLAARD FILE NUMBER: 190867

DRAWING NUMBER:

190867–PRE

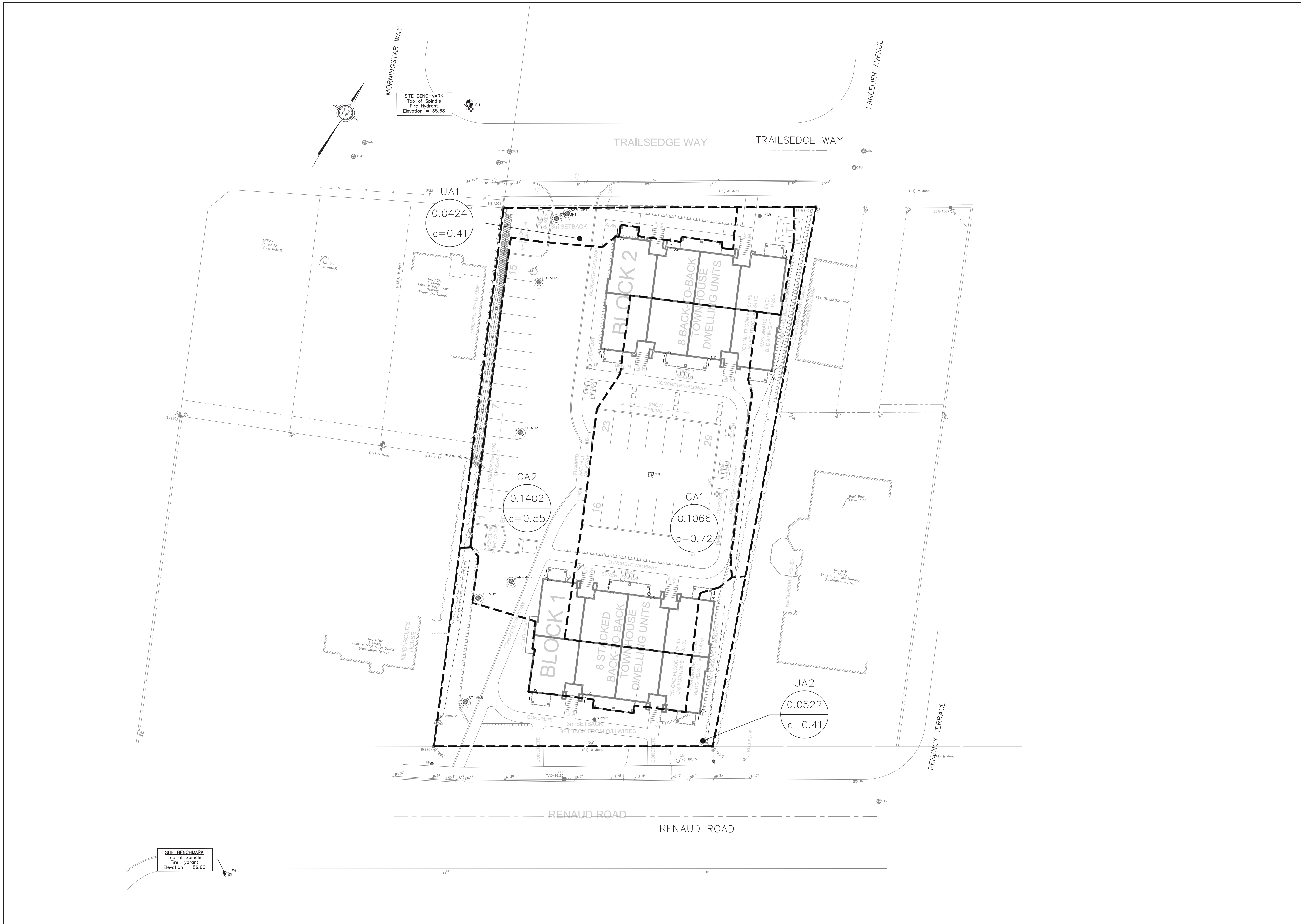
DRAWING NAME:

PRE–DEVELOPMENT DRAINAGE

ARCH D

D07-12-20-0094

#18196

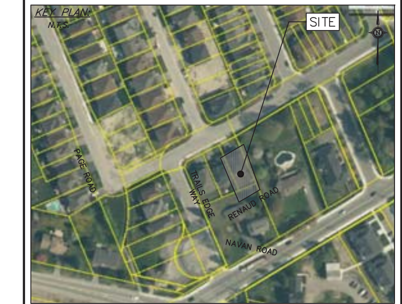


POST-DEVELOPMENT DRAINAGE

SCALE = 1:250

DRAWING NUMBER:
190867-POST

KEY PLAN:



SCALE:

LEGEND (STORM WATER MANAGEMENT)

CONTROLLED — CATCHMENT LABEL
0.39 — CATCHMENT AREA (HECTARES)
c=0.34 — RUNOFF COEFFICIENT
CATCHMENT AREA BOUNDARY
DIRECTION OF FLOW
PROPERTY LINE
TOP OF SLOPE
CONTROLLED AREA
UNCONTROLLED AREA
PRE-DEVELOPMENT DRAINAGE PATTERN

No.	REVISION	DATE	BY
4	REVISIONS PER NEW SITE PLAN AND REVIEW COMMENTS	APR. 13/2022	ML
3	ISSUED FOR SPC RE-SUBMISSION	APR. 22/2021	ML
2	REVISIONS PER SITE PLAN AND 1ST REVIEW COMMENTS	MAR. 29/2021	ML
1	ISSUED FOR SPC APPLICATION	JUNE 30/2020	ML
#	REVISION ITEM / DESCRIPTION	REV. DATE	INT.

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LICENSED PROFESSIONAL ENGINEER
APRIL 13, 2022
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGNED BY:	CHECKED BY:
--	--
DRAWN BY: --	APPROVED BY: --

DATE: NOV. 11, 2019

KOLLAARD FILE NUMBER: 190867

DRAWING NUMBER:
190867-POST

DRAWING NAME:
POST-DEVELOPMENT DRAINAGE

ARCH D

D07-12-20-0094

#18196

LEGEND

EXISTING ELEVATION	EXISTING UTILITY POLE
PROPOSED/EXISTING ELEVATIONS	FIRE HYDRANT
PROPOSED CURB ELEVATION	PROPOSED LAMP POST
PROPOSED ELEVATION AT BUILDING ENTRANCE	PROPOSED CURB STOP
DRAINAGE SLOPE	PROPOSED WATER METER
PROPOSED BOARD FENCE	PROPOSED REMOTE WATER METER
WATERMAIN	SANITARY SERVICE CONNECTION
STORM SEWER	SUMP LOCATION
SANITARY SEWER	PROPOSED DOWNSPOUT LOCATION
TOP OF SLOPE	EXISTING WATER STAND POST
PROPERTY LINE	EXISTING WATER VALVE
OVERHEAD WIRES	EXISTING STORM MANHOLE
SILT FENCE	EXISTING SANITARY MANHOLE
PROPOSED DEPRESSED CURB	EXISTING CATCH BASIN
PROPOSED SWALE	PROPOSED REAR-YARD CB
OVERLAND FLOW ROUTE	PROPOSED CATCH BASIN/MANHOLE
LOCATION OF EXISTING SERVICE CONNECTIONS	PROPOSED CATCH BASIN
TEMPORARY BENCHMARK	PROPOSED STORM MANHOLE
	PROPOSED SANITARY MANHOLE

WATER CROSSING TABLE

1	BOTTOM OF W/M = 82.45 STM OBV = 82.05± SAN OBV = 80.55±
2	BOTTOM OF W/M = 82.70 STM OBV = 82.44
3	SAN INV = 83.40 TOP OF W/M = 82.90
4	BOTTOM OF W/M = 83.40 STM OBV = 83.14
5	SAN INV = 84.40 TOP OF W/M = 83.85
6	STM INV = 84.00 TOP OF W/M = 83.50

NOTE: REFER TO CITY OF OTTAWA STANDARD DWG NO. W25 FOR INSTANCES WHERE WATERMAIN CROSSING IS BELOW SEWER

INLET CONTROL DEVICE TABLE

STRUCTURE	STM-MH2	CB1
MODEL	HYDROVEX 75-SVHV-1	HYDROVEX 75-SVHV-1
PIPE OUTLET	250 mm PVC SDR 35	250 mm PVC SDR 35
DISCHARGE	12.4 L/s	7.0L/s
UPSTREAM HEAD	3.4m	2.7m
HWL ABOVE ICD	85.25m	86.00m
ICD INVERT ELEVATION	82.00m	83.30m
STRUCTURE SIZE	1.2m DIA.	0.6mx0.6m
MINIMUM CLEARANCE		

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	SPEC. No.	CITY STD DWG No.
CATCH BASIN (600mm x 600mm)	OPSD 705.010	S2
STORM/SANITARY MANHOLE (1200ø)	OPSD 701.010	
SEWER SERVICE CONNECTION		S11 & S11.1
SANITARY BENCHING	OPSD 701.021	
CATCH BASIN & MANHOLE ADJUSTMENTS	OPSD 704.010	
STORM MANHOLE FRAME & COVER	OPSD 401.010	S24.1 & S25
CATCH BASIN FRAME & COVER	OPSD 400.020	S19, S22 & S23
SEWER TRENCH		S6 & S7
SANITARY MANHOLE FRAME & COVER	OPSD 401.030	S24 & S25

SEWER TRENCH:
SITE SERVICES EXCAVATION, BEDDING & BACKFILL AS PER THE RECOMMENDATIONS OF THE GEOTECHNICAL INVESTIGATION PREPARE BY KOLLAARD ASSOCIATES INC.

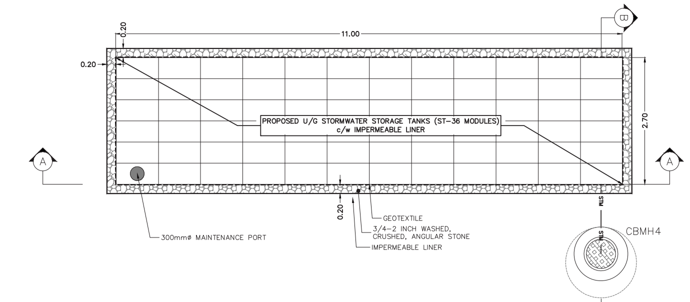
3. INSULATE ALL SEWER PIPES THAT HAVE LESS THAN 2m COVER WITH THERMAL INSULATION. PROVIDE 150mm CLEARANCE BETWEEN PIPE AND INSULATION.
4. PIPE BEDDING, COVER AND BACKFILL ARE TO BE COMPACTED TO AT LEAST 98% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY.
5. FLEXIBLE CONNECTIONS ARE REQUIRED FOR CONNECTION 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. THE OWNER SHALL REQUIRE THAT THE SITE SERVICING CONTRACTOR PERFORM FIELD TESTS FOR QUALITY CONTROL OF ALL SANITARY SEWERS. LEAKAGE TESTING SHALL BE COMPLETED IN ACCORDANCE WITH OPSS 410.07.16, 410.07.16.04 AND 407.07.24. DYE TESTING IS TO BE COMPLETED ON ALL SANITARY SERVICES TO CONFIRM PROPER CONNECTION TO THE SANITARY SEWER MAIN. THE FIELD TESTS SHALL BE PERFORMED IN THE PRESENCE OF A CERTIFIED PROFESSIONAL ENGINEER WHO SHALL SUBMIT A CERTIFIED COPY OF THE TEST RESULTS.
7. STORM MANHOLES AND CBMHS ARE TO HAVE 300mm SUMPS (AS PER SUMP DETAIL ON OPSD 701.010), UNLESS OTHERWISE INDICATED.
8. BUILDING CONTRACTOR TO PROVIDE TEMPORARY ADDITIONAL GRANULAR BACKFILL ABOVE SHALLOW CULVERTS AND STORM SEWERS TO SUPPORT HEAVY CONSTRUCTION EQUIPMENT.
9. 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.
10. WHERE THE SANITARY SEWER CROSSES ABOVE THE WATERMAIN, THE CONTRACTOR IS TO PROVIDE A MINIMUM OF 0.50m VERTICAL SEPARATION, ADEQUATE STRUCTURAL SUPPORT OF THE SEWER TO PREVENT SETTLING AND EXCESSIVE JOINT DEFLECTION AND ENSURE THAT THE LENGTH OF THE WATER PIPE BE CENTERED AT THE POINT OF CROSSING SO THAT THE JOINTS ARE EQUIDISTANT AND AS FAR AS POSSIBLE FROM THE SEWER.

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:

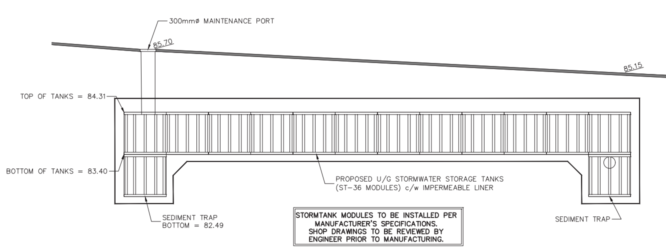
ITEM	SPEC. No.	CITY STD DWG No.
WATERMAIN BEDDING AND BACKFILL	OPSD 802.010/802.031	W17 (trench detail)
CATHODIC PROTECTION	OPSD 1109.010	W40
PRESSURE TESTING	AWWA C-605-5	
CHLORINATION	AWWA C-651-05	
WATERMAIN MATERIAL	PVC DR18 (CLASS 150)	

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.



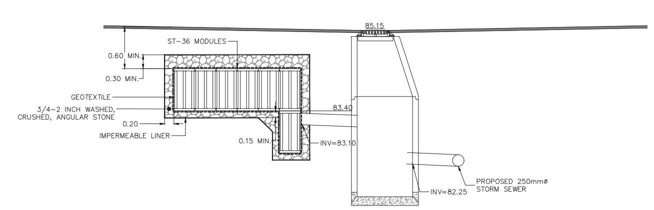
STORMWATER STORAGE TANKS PLAN VIEW (TYPICAL)

NOT TO SCALE



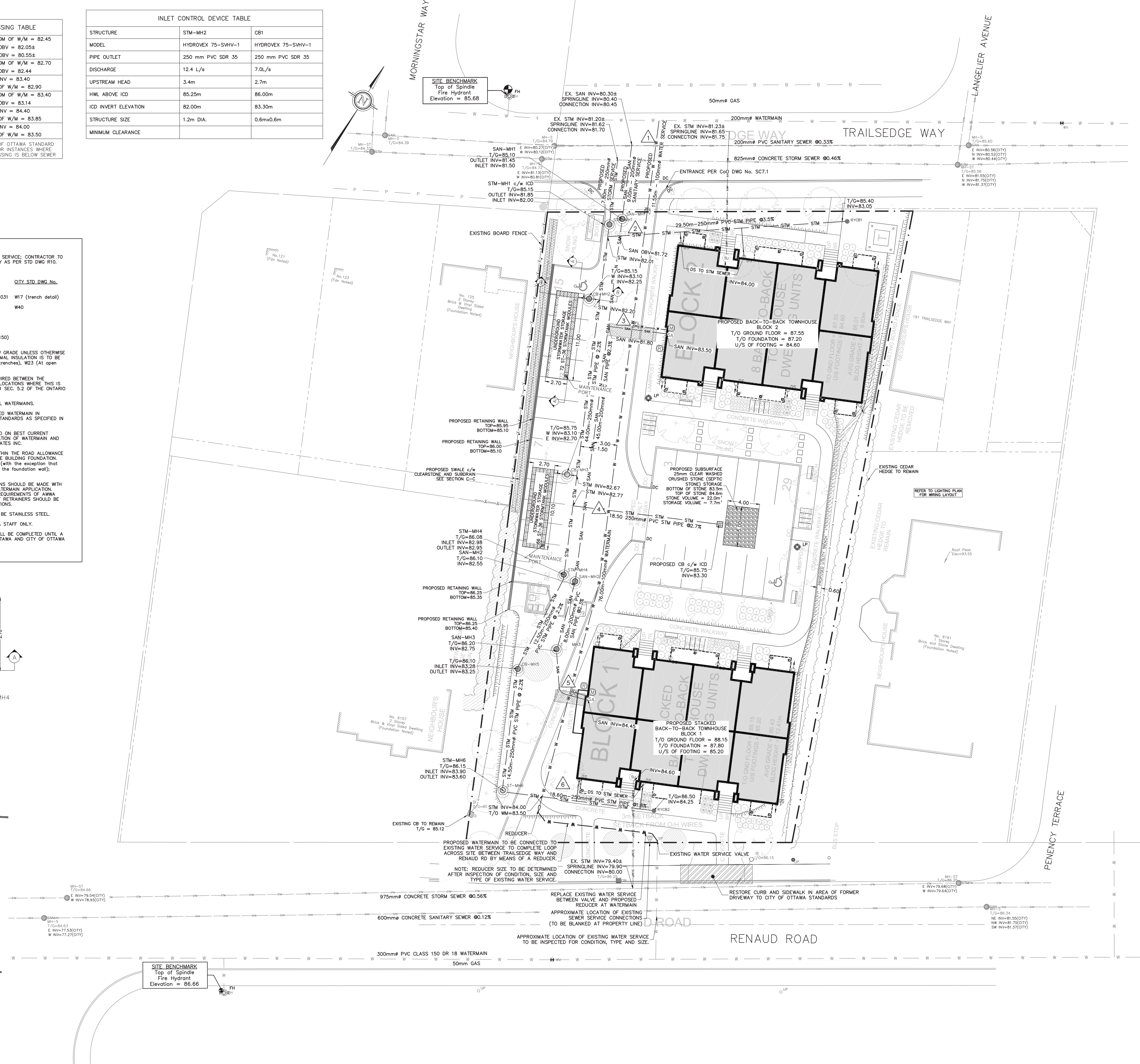
SECTION A-A (TYPICAL)

NOT TO SCALE



SECTION B-B (TYPICAL)

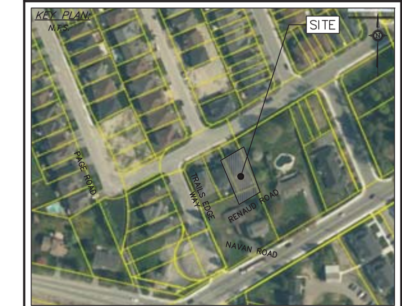
NOT TO SCALE



SITE SERVICING PLAN

SCALE = 1:250

DRAWING NUMBER: 190867-SER



SCALE: 1:250 METRES

GENERAL NOTES:

1. All dimensions are in metres; all elevations are in metres and are geodetic. TBM = top of spindle of existing fire hydrant on Trailsedge Way. Elevation = 85.68. Geodetic datum reference: CGVD-1928;1978.
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9. A geotechnical engineer should be retained to provide recommendations with respect to the sub-grade conditions prior to footing installation.
10. The owner agrees to prepare and implement an erosion and sediment control plan 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 such as, and not limited to installing filter cloths across manhole/catchbasin lids to prevent sediments from entering structures and install and maintain a light duty silt fence barrier as required.
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12. Reference to Kollaard File No. 190867 for Servicing and Stormwater Management Design and Geotechnical Reports.

No.	REVISION	DATE	BY
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KEMPTVILLE, ONTARIO
K0G 1J0 FAX (613) 258-0475
http://www.kollaard.ca

CLIENT:
TEAK DEVELOPMENTS
31 WOODVIEW CRESCENT
OTTAWA, ON K1B 3B1

PROJECT:
PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:
6173 RENAUD ROAD
CITY OF OTTAWA, ON
K1W 0K9

LICENSED PROFESSIONAL ENGINEER
APRIL 13, 2022
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGNED BY:	CHECKED BY:
SD	SD
DRAWN BY:	APPROVED BY:
ML	SD

DATE: NOV. 11, 2019

KOLLAARD FILE NUMBER: 190867

DRAWING NUMBER: 190867-SER

DRAWING NAME:

D07-12-20-0094

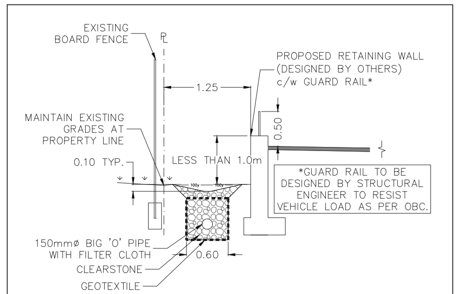
#18196

LEGEND

x68.75	EXISTING ELEVATION	UP	EXISTING UTILITY POLE
68.75 / 68.75	PROPOSED/EXISTING ELEVATIONS	FH	FIRE HYDRANT
68.75C	PROPOSED CURB ELEVATION	LP	PROPOSED LAMP POST
68.75	PROPOSED ELEVATION AT BUILDING ENTRANCE	WV	PROPOSED CURB STOP
0.0%	DRAINAGE SLOPE	M	PROPOSED WATER METER
	PROPOSED BOARD FENCE	R	PROPOSED REMOTE WATER METER
W	WATERMAIN	SA	SANITARY SERVICE CONNECTION
STM	STORM SEWER	SU — SUMP	SUMP LOCATION
SAN	SANITARY SEWER	DS	PROPOSED DOWNSPOUT LOCATION
	TOP OF SLOPE	SP	EXISTING WATER STAND POST
	PROPERTY LINE	WV	EXISTING WATER VALVE
OHW	OVERHEAD WIRES	STM	EXISTING STORM MANHOLE
	SILT FENCE	SAN	EXISTING SANITARY MANHOLE
DC	PROPOSED DEPRESSED CURB	CB	EXISTING CATCH BASIN
	PROPOSED SWALE	RYCB	PROPOSED REAR-YARD CB
	OVERLAND FLOW ROUTE	CBMH	PROPOSED CATCH BASIN/MANHOLE
S	LOCATION OF EXISTING SERVICE CONNECTIONS	CB	PROPOSED CATCH BASIN
	TEMPORARY BENCHMARK	STM	PROPOSED STORM MANHOLE
		SAN	PROPOSED SANITARY MANHOLE

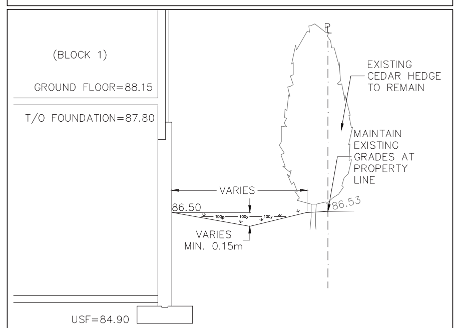
GRADING NOTES:

1. ALL TREES ON THE RIGHT-OF-WAY ARE TO BE MAINTAINED BEFORE AND AFTER THE CONSTRUCTION AND ALL EXISTING TREES WITHIN THE PROPERTY SHALL BE PROTECTED AS PER "MUNICIPAL TREES AND NATURAL AREAS PROTECTION BY-LAW" AND THE "URBAN TREES CONSERVATION BY-LAW" AS AMENDED FROM TIME TO TIME.
2. NO EXCESS DRAINAGE WILL BE DIRECTED TOWARDS THE NEIGHBOURING PROPERTIES DURING AND AFTER CONSTRUCTION.
3. ALL RETAINING WALLS TO HAVE MINIMUM 0.15 METRE CLEARANCE FROM PROPERTY LINE.
4. EAVES TROUGHS TO BE DIRECTED TO THE GROUND SURFACE 1.2 METRES FROM FOUNDATION EXCEPT AS INDICATED.
5. THERE IS TO BE NO ALTERATION TO THE EXISTING GRADE AND DRAINAGE PATTERNS ON THE PROPERTY LINES.



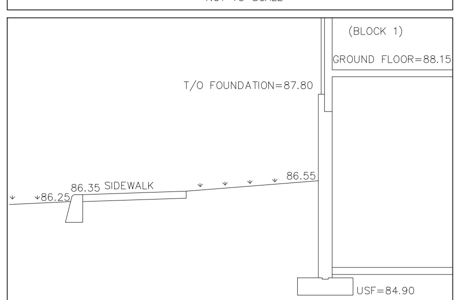
SECTION C-C (TYPICAL)

NOT TO SCALE



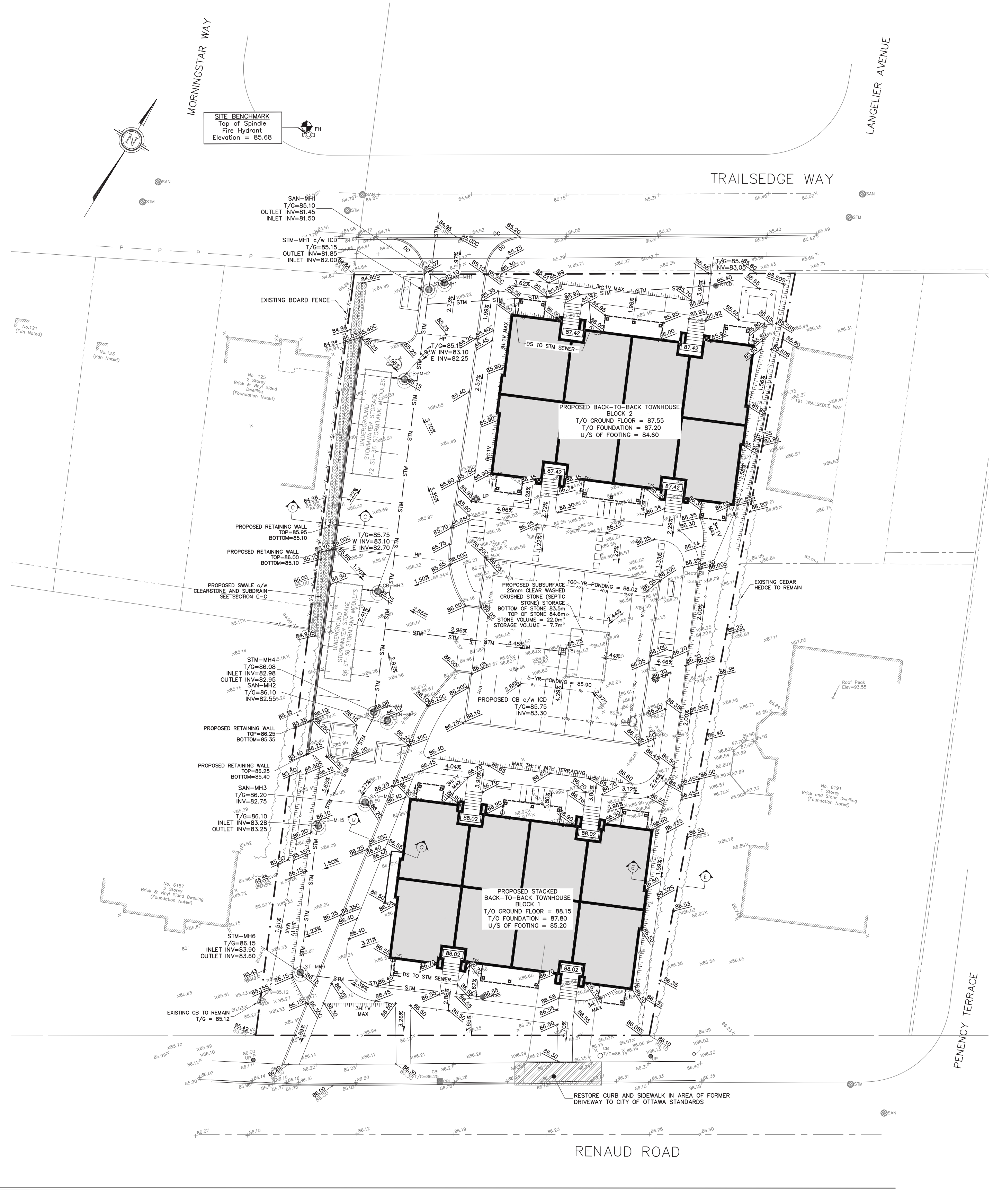
SECTION E-E (TYPICAL)

NOT TO SCALE



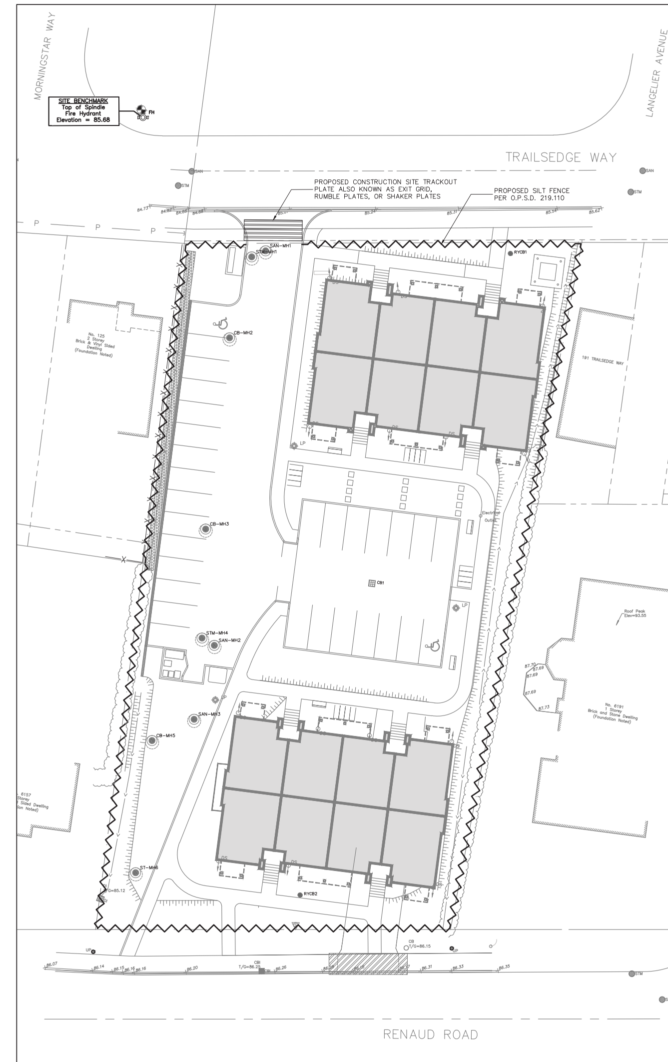
SECTION G-G (TYPICAL)

NOT TO SCALE



SITE GRADING PLAN

SCALE = 1:250



EROSION & SEDIMENT CONTROL PLAN

SCALE = 1:400

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.

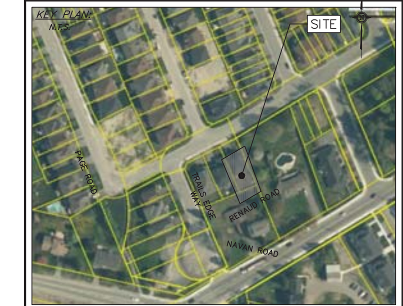
MINIMUM EROSION AND SEDIMENT CONTROL PLAN REQUIREMENTS:

- 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 SOCKS IN ALL EXISTING AND PROPOSED CATCH BASINS AND CATCH BASIN MANHOLES PRIOR TO CONSTRUCTION.

DRAWING NUMBER:

190867-GEC

KEY PLAN



SCALE:

0 1 2 3 4 5 10 15 20

1:250 METRES

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No.	REVISION	DATE	BY

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

CLIENT:

TEAK DEVELOPMENTS
31 WOODVIEW CRESCENT
OTTAWA, ON K1B 3B1

PROJECT:

PROPOSED RESIDENTIAL DEVELOPMENT

LOCATION:

6173 RENAUD ROAD
CITY OF OTTAWA, ON
K1W 0K9

LICENSED PROFESSIONAL ENGINEER
APRIL 13, 2022
S.E. deWit
100079612
PROVINCE OF ONTARIO

DESIGNED BY: SD	CHECKED BY: SD
DRAWN BY: ML	APPROVED BY: SD
DATE: NOV. 11, 2019	
KOLLAARD FILE NUMBER: 190867	

DRAWING NUMBER:

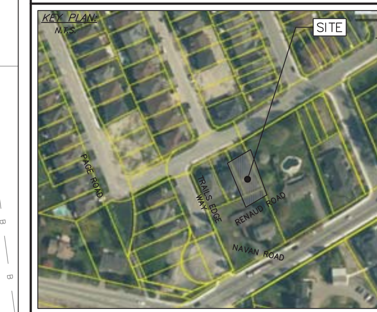
190867-GEC

DRAWING NAME: SITE GRADING AND EROSION CONTROL PLAN

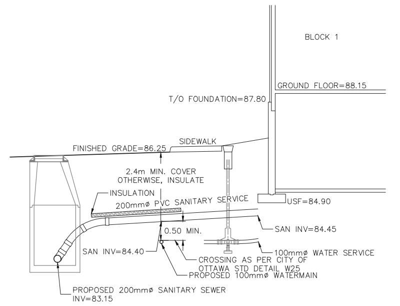
ARCH D

D07-12-20-0094

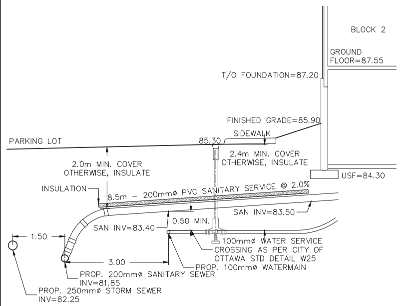
#18196



SCALE: 0 1 2 3 4 5 10 15 20
1:250 METRES



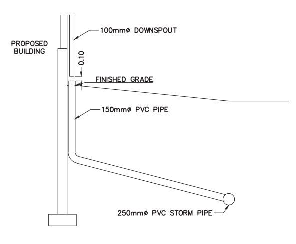
SERVICE CONNECTION DETAIL (BLOCK 1)
NOT TO SCALE



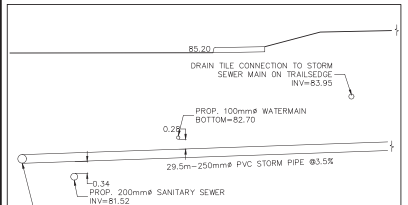
SERVICE CONNECTION DETAIL (BLOCK 2)
NOT TO SCALE

WATER CROSSING TABLE	
1	BOTTOM OF W/M = 82.45 STM OBV = 82.05± SAN OBV = 80.55±
2	BOTTOM OF W/M = 82.70 STM OBV = 82.44
3	SAN INV = 83.40 TOP OF W/M = 82.90
4	BOTTOM OF W/M = 83.40 STM OBV = 83.14
5	SAN INV = 84.40 TOP OF W/M = 83.85
6	STM INV = 84.00 TOP OF W/M = 83.50

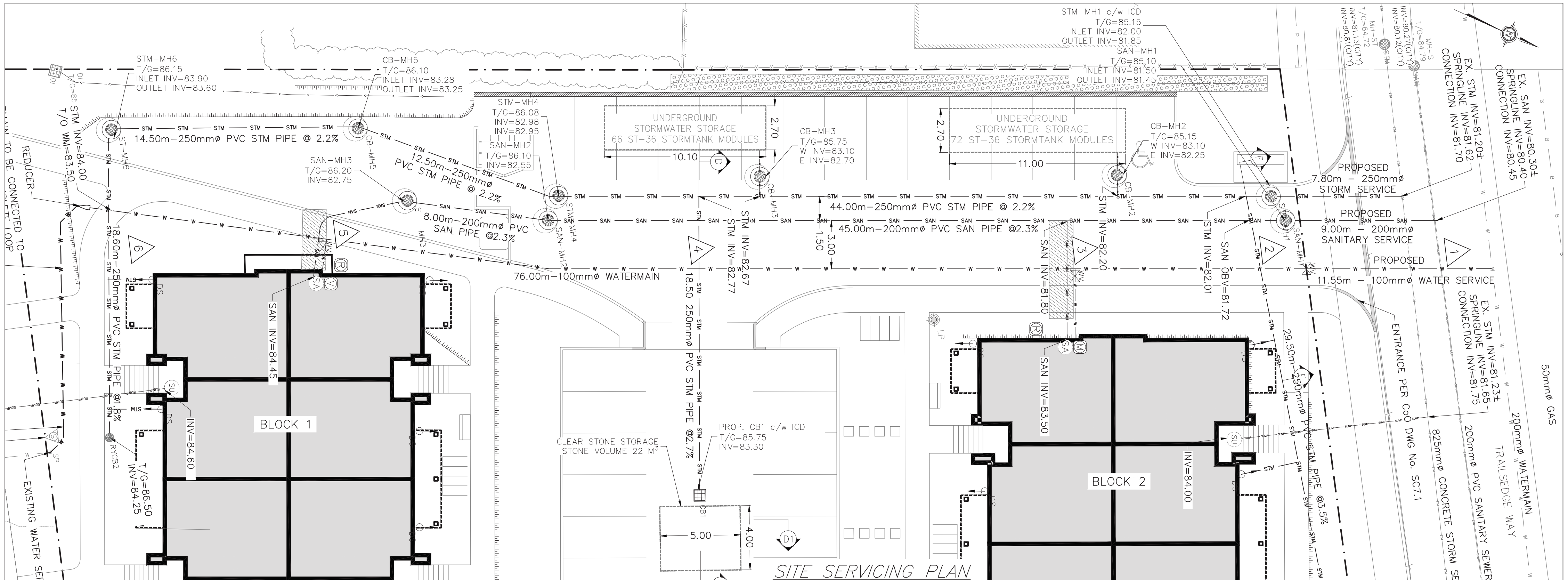
NOTE: REFER TO CITY OF OTTAWA STANDARD DWG NO. W25 FOR INSTANCES WHERE WATERMAIN CROSSING IS BELOW SEWER



DOWNSPOUT CONNECTION (TYPICAL)
NOT TO SCALE



SECTION F–F
NOT TO SCALE

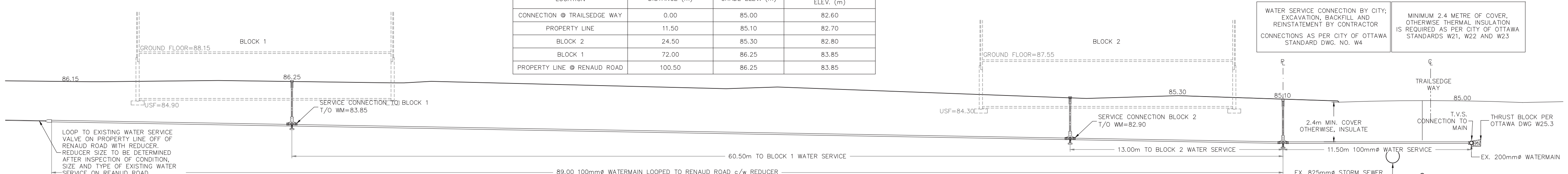


SITE SERVICING PLAN
SCALE = 1:150

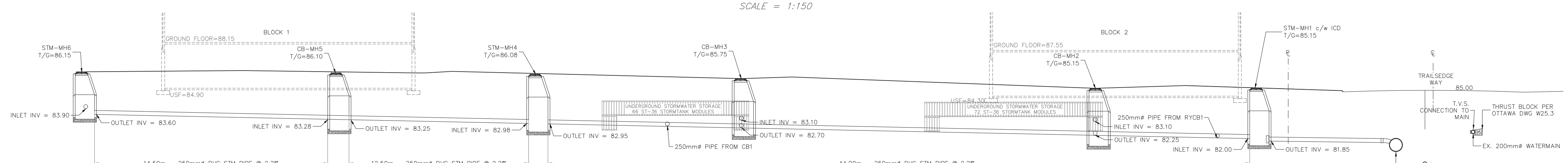
WATER TABLE [6173 RENAUD ROAD – SERVICE CONNECTION ON TRAILSEDGE WAY]			
LOCATION	DISTANCE (m)	GRADE ELEV. (m)	TOP OF WATER SERVICE ELEV. (m)
CONNECTION @ TRAILSEDGE WAY	0.00	85.00	82.60
PROPERTY LINE	11.50	85.10	82.70
BLOCK 2	24.50	85.30	82.80
BLOCK 1	72.00	86.25	83.85
PROPERTY LINE @ RENAUD ROAD	100.50	86.25	83.85

WATER SERVICE CONNECTION BY CITY; EXCAVATION, BACKFILL AND REINSTATEMENT BY CONTRACTOR
CONNECTIONS AS PER CITY OF OTTAWA STANDARD DWG. NO. W4

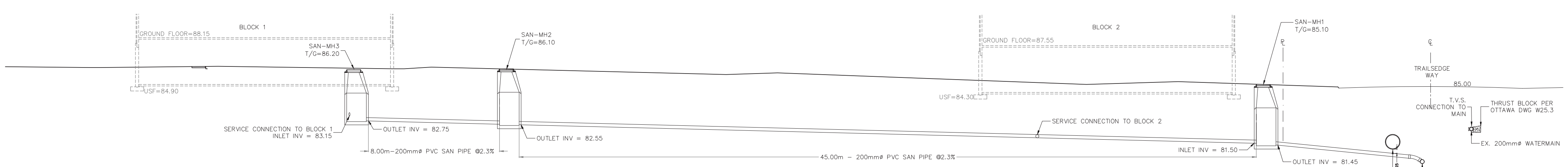
MINIMUM 2.4 METRE OF COVER, OTHERWISE THERMAL INSULATION IS REQUIRED AS PER CITY OF OTTAWA STANDARDS #21, #22 AND W23



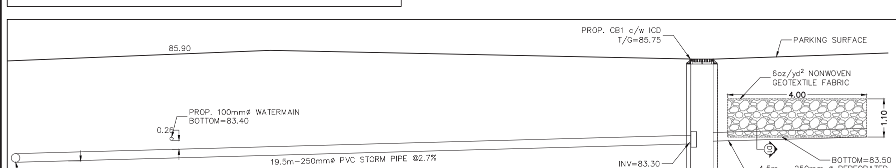
WATERMAIN DETAIL
SCALE = 1:150



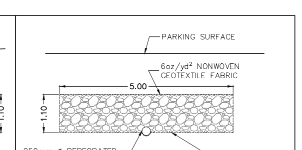
STORM SEWER DETAIL
SCALE = 1:150



SANITARY SEWER DETAIL
SCALE = 1:150



SECTION D–D
NOT TO SCALE



SECTION D1
NOT TO SCALE

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LICENSED PROFESSIONAL ENGINEER
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PROVINCE OF ONTARIO

DESIGNED BY: SD
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APPROVED BY: SD
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DRAWING NUMBER: 190867–DET
DRAWING NAME: DETAILS

ARCH D

D07-12-20-0094

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