PROPOSED EMERGENCY SERVICES DEVELOPMENT
VAUGHAN FIRE STATION 7-12
9541 Weston Road
Woodbridge, ON M6M 4P8

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

Prepared for:

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Prepared by:

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File No. 2022-007

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Appendices

1. Introduction

MGM Consulting Inc. (MGM) has been retained by Thomas Brown Architects to prepare a Functional Servicing and Stormwater Management Report in support of the Site Plan Application for a proposed Fire Station development located at 9541 Weston Road, Woodbridge, Ontario.

This report has been prepared to address the municipal servicing and stormwater management requirements for the subject lands including sanitary drainage, water supply and distribution, stormwater management objectives, upstream drainage conveyance, site grading and temporary on site works to accommodate internal and external construction phasing. The overall design objectives will be in accordance with current City of Vaughan design standards and as required to achieve the objectives identified in the York Region & Toronto and Region Conservation Authority (TRCA) Guidelines.

2. Existing Conditions

The total area of the subject property is approximately 2.0 hectares in area of which approximately 0.5 hectares will be re-developed. The proposed development is the south west portion of the land owned by City of Vaughan. The legal description of the property is Block 163 Registered Plan 65M-3348 and Part of North Half Lot 17 Concession 5, Geographic Township of Vaughan, County of York now in the City of Vaughan Regional Municipality of York.

The development area is fronting on Weston Road to the west, Vellore Tract Woodlot to the east, multiple community buildings to the north & Johnswood Crescent to the north east.

The subject land currently occupies the following buildings;

- a one-storey brick building Vellore Hall Park on the north side (9541 Weston Road),
- a two-storey brick church building on the north side (9545 Weston Road),
- a one-storey wood shed on the northeast,
- a wooden Gazebo with paving stones and concrete walkway to the Johnswood Crescent on the northeast,
- An asphalt parking lot south of the buildings facing Weston Road.

2.1. Existing Topography and Drainage

Drainage from the site is currently draining away from the existing buildings in sheet flow manner from west to east.

Existing elevations of the site area of the property range from 226.64 metres in the northwest side to 224.23 metres at the northeast area of the subject area near the edge of the concrete walkway to the Johnswood Crescent. The existing topography is indicated on **Figure No. 1 & 2**.

2.2. Existing Servicing

Information on the existing municipal servicing in the vicinity of the proposed site development was obtained from historical records provided by the City of Vaughan & York Region. Based on the available information sources, the existing municipal services relevant to the current proposal includes the following:

Within Weston Road, north to south:
-1800mm dia. C.P.P. feedermain
-600mm dia. C.P.P. watermain
-300mm dia. PVC storm sewer
-32mm gas main
-underground hydro
-underground telephone line
-underground cable line
-underground fibre optic line

East to west:
-300mm dia. PVC storm sewer

Within Subject Property, east to west:
- 200mm dia. PVC sanitary sewer
- 150mm dia. PVC watermain
- underground hydro
- 200mm dia. PVC storm sewer
- 150mm dia. PVC storm sewer
- 32mm gas main

Within Johnswood Crescent, north to south:
- 375mm ultra rib pvc storm
- 450mm ultra rib pvc storm
- 675mm dia. conc storm sewer
- 600mm dia. conc storm sewer
- 200mm dia. PVC sanitary sewer

- 200mm dia. PVC watermain
- underground hydro
- underground cable line
- underground telephone line
- 32mm gas main

East to west:
-200mm PVC SAN storm sewer
-300mm ultra-rib PVC storm sewer

Majority of the drainage from the subject property is currently draining easterly towards two existing catchbasins which drains to an existing infiltration pit on the east side which makes a by-pass connection to an existing catchbasin on the north east corner of the land where the drainage flows overland towards the existing ditch inlet catchbasin located at the south west corner of Johnswood Crescent outletting to the existing 600mm municipal storm sewer within Johnswood Crescent right of way.

3. Proposed Site Development

The project involves the demolition of the existing parking lot on the southside and construction of a new one-storey emergency services fire station with parking lot expansion to the east side.

The proposed conditions are shown in **Figure No.4**.

4. Proposed Grading

Based on the current site plan, there are several constraints associated with the overall grading design. Some of them are the proposed driveway entrance from Weston Road, continuous pedestrian access from the northwest side and existing grades along the woodlot buffer on south side.

Based on the above constraints, the finished floor elevation of the proposed building is proposed at 225.70 meters.

Additional constraints include providing sufficient cover over the proposed storm sewers which will convey storm drainage to the existing storm service connection on the east side and the costs associated with importing material for filling purposes.

Proposed grading within the hardscaped area of the site will be in the range of 1%-5% and will be designed to direct surface drainage to proposed storm structures to ensure site draining is routed through the private storm system towards the ultimate site outlet.

Proposed site grading is indicated on the Site Grading Plan, **Drawing CV-2**.

5. Private Servicing Design

5.1. Minor Storm Servicing

The minor storm sewer network is designed to convey the flows from the site based on a 5-year storm event, without surcharging, in accordance with the City of Vaughan requirements.

Storm drainage from the proposed development area will consist of roof drainage from the new building and drainage from the parking lot. Roof drainage will be directly conveyed to a newly proposed infiltration pit on the north side, which will then have a by-pass connection to the internal storm sewer network. The downstream of the internal storm system will be connected to the existing infiltration pit at the east side via an OGS unit prior to outletting to the existing storm service connection through a by-pass connection from the existing infiltration pit.

The existing storm infrastructure within the site will be decommissioned and removed as required to accommodate the proposed redevelopment layout.

The minor system is designed based on the Rational Method using a time of concentration of 7 minutes and the City of Vaughan's IDF curve with a 5-year return period. The storm sewer design sheet has been included in **Appendix B**. The existing and proposed drainage areas can be found in **Figure 3** and **Figure 5** respectively.

Proposed storm servicing for the subject development is indicated on the Site Servicing Plan, **Drawing CV-3**.

5.2. Major Storm Servicing

In the event of a major storm or the storm system becoming blocked, the proposed grading will be designed to ensure emergency overland flow is directed through the proposed parking lot expansion on the east side towards the concrete walkway east of the existing Gazebo draining to the Johnswood Crescent right of Way, located at the northeast corner of the property at an elevation of 224.23 m; 1.47 m below the proposed finished floor elevation.

5.3. Water Servicing

Water servicing as required for a domestic supply and fire protection for the building is proposed to be provided with a new 150mm fire line connection & a new 100mm domestic connection from the existing 150mm watermain located on north side of the property along the concrete walkway. This existing 150mm watermain is connected to the existing 200mm municipal watermain located within Johnswood Crescent right of way.

The proposed water service connection is indicated on the Site Servicing Plan, **Drawing CV-3**. Calculations for water demand are provided in **Appendix D**.

A preliminary calculation for the required fire flow to protect the new construction is included in **Appendix C** based on the criteria provided by the Fire Underwriters Survey (FUS). The analysis indicates that a flow of 7,000 L/min is required at a residual pressure of 140 kPa to provide fire protection for the new construction.

Flow and pressure test of the existing 150mm watermain has been conducted by The Ontario Clean Water Agency. Hydrant test results are provided in **Appendix H** and the results indicate a flow of 19,616 L/min (5182 gpm) (which is higher than the required) is available at a residual pressure of 20 psi (140 kPa). Based on the above, it can be concluded that the site can be adequately serviced with water servicing.

Final confirmation of adequate flow and pressure for fire protection will be provided by the mechanical engineer during the detailed design phase.

5.4. Sanitary Servicing

Sanitary servicing as required for the proposed development is to be provided by a new 200mm service connecting to the existing 200mm sanitary sewer located on north side of the property along the concrete walkway. This existing 200mm sanitary pipe is connected to the existing 200mm municipal sanitary sewer located within Johnswood Crescent right of way.

The proposed sanitary service connection is indicated on the Site Servicing Plan, **Drawing CV-3**. Calculations for sanitary flow projections are provided in **Appendix E**.

6. Proposed Stormwater Management

6.1. Stormwater Management Design Criteria

Stormwater management for the site is to be consistent with the policies and guidelines identified in the City of Vaughan's Development Standard and TRCA's Equation G - Sub Basin 46 of Humber River Stormwater Management Quantity Control Release Rates drawing of the TRCA SWM criteria (2012) document.

The Guideline recommends the objectives identified as follows.

- Rate controls as required to control flows to a pre-determined, or existing condition,
- Water balance objectives as required to provide for groundwater recharge and contain small frequent flows within the site boundaries, and
- Water quality as required to contain sediments and oils that may be transported to adjacent storm conveyance systems.

Preliminary Stormwater Management calculations are provided in **Appendix A.**

6.2. Stormwater Quantity/Rate Controls

Stormwater peak rate controls are required to control flows during the 2 to 100-year storm events to unitary flow rates per TRCA's Equation G - Sub Basin 46 of Humber River Stormwater Management Quantity Control Release Rates drawing of the TRCA SWM criteria (2012) document. The proposed strategy for stormwater management includes an internal storm sewer system, stormbrixx cells and a control orifice tube located at the control manhole which outlets to the existing storm service connection which ultimately outlets to the existing 600 mm municipal sewer within Johnswood Crescent right of way.

Because of site characteristics that include external drainage areas, controlling flows during the 2 to 100-year storm events to unitary flow rates per TRCA's Equation G would not be practical, the installation of the proposed 75 mm orifice tube (located at the outlet pipe of the control manhole MH 2) and the installation of flow control roof drains on new building roof areas are used to achieve the required 100-year storm storage based on the unitary flow rates per TRCA's Equation G. The roof controls will provide a maximum ponding to a depth of 0.15m. The roof flows were calculated as per the manufacturer's (Zurn) standard flow rating specification for a maximum ponding depth of 0.15m.

Stormwater management calculations are included in **Appendix A** which provide a summary of existing drainage characteristics within the site, the allowable site flow based on controlling flows per TRCA's Equation G - Sub Basin 46 of Humber River Stormwater Management Quantity Control Release Rates, and the required site storage which is provided within the rooftop, stormbrixx cells and within storm sewer pipes.

Pre and Post development drainage areas within the site are included as **Figures 3 and 5**, respectively.

As indicated in **Appendix A**, the stormbrixx cells, in combination with the proposed orifice tube, roof control drains and the internal storm sewers will provide the required storage based on the unitary flow rates per TRCA's Equation G - Sub Basin 46 of Humber River Stormwater Management Quantity Control Release Rates. A summary of post development flows (attenuated flows + unattenuated flows) and storage during the 100-year storm event, follows:

Table 1 - Summary of Post Development Flows

Item	Storage Required (cu.m)	Max Storage Provided (cu.m)	Actual Discharge (cms)
Water balance	-	53.3	-
100 Year Flood	114.9	115.1	0.0183

6.3. Stormwater Quality Controls

The subject property encompasses a significant portion of asphalt parking area which requires enhanced level of treatment (80% TSS removal) of the runoff from the site.

Shields at all catch basin/catch basin maintenance hole locations combined with a treatment unit (HydroStorm HS 4) proposed downstream of the internal storm sewer system, will provide the required quality control for the drainage coming from the new parking lots as indicated on Drawing CV-3. The design of the HydroStorm unit is attached in **Appendix F**.

The proposed treatment unit and shields will be designed to achieve 80% TSS removal during 90% of the average annual rainfall events on an annual loading basis prior to discharging stormwater runoff to the existing municipal sewer. The combined TSS removal calculations are attached in **Appendix G.**

6.4. Water Balance

Since addition of a new building will alter the balance of runoff, infiltration and evaporation, water balance must be restored in the post-development conditions back to the pre-development levels. This drainage alteration to the natural drainage conditions requires remediation through retention of the incoming stormwater runoff and releasing at a controlled rate to ensure the appropriate amount of water balance is achieved within the subject site.

To accomplish this, clean water from rooftops areas will be diverted towards a newly proposed infiltration trench located north of the new building. There is also an existing infiltration pit east of the new building which was constructed in 2017. It is proposed to protect this existing infiltration pit and utilize it for water balance purpose.

As per the water balance calculations attached in **Appendix A**, the existing infiltration pit along with the proposed infiltration pit and softscapes over the area bounded by the construction limit provides an estimated depth of mm, which is equivalent to capturing an

estimated 80% of the average annual rainfall depths. A 36.5 cu.m. volume of water within the proposed infiltration pit & a 17 cu.m volume of water within the existing infiltration pit are required to achieve this objective.

For the purpose of confirmation that this volume of water can be infiltrated within the pit at a 72 hour timeframe, a Percolation Test was carried out and the test results provided in Appendix A confirms that the water can be infiltrated within a 72 hour timeframe.

The proposed infiltration pit is provided with relief outlet back to the internal storm network in the event it is clogged or overwhelmed during major storm events. The invert levels of this outlet will be set high enough to ensure the internal storm sewer network is hydraulically disconnected from the pit itself thereby preventing untreated stormwater runoff from the site from entering the pit.

6.5. Low Impact Development (LID) Measures

To achieve a sustainable development, LID measures will be adopted in the SWM design.

Application of LIDs will help reduce runoff volumes and promote infiltration, in combination with increased extended detention storage and drawdown times within end-of-pipe facilities. The main purpose of LID measures is to eliminate the direct connection between impervious surfaces such as roads, parking lots, driveways, roofs, and the storm drainage system. LID measures take into account more frequent storm events (5mm rainfall events) of lower volumes as opposed to the less frequent storm events (e.g. 100 year storm) with higher volumes. The LID measures which promote infiltration or filtration through a granular medium also provide thermal mitigation for storm runoff. They can be implemented to varying degrees based upon the available area given and the proposed land use and development form and the soil infiltration capacity.

The SMW design will consider the predevelopment hydrology by using design techniques that infiltrate, filter, store, evaporate, and detain runoff close to its source. A treatment train approach including possible LID measures will be considered to provide quality control.

An infiltration trench has been incorporated into the SWM design. It will be comprised of a trench lined with filter fabric and filled with clear stone, where storm water will percolate into the native soil. This feature will be used to infiltrate clean runoff from roofs.

A green roof is incorporated as one of the LID features in the design. This best management practice will be used to reduce clean runoff volume from roofs and promote absorption and reuse of stormwater.

Reduced lot grading has also been considered as one of the LIDs incorporated into the site plan. To maximize infiltration and evapotranspiration of stormwater runoff at the lot level, grades within the hardscaped areas are limited to a maximum of 5% and a minimum of 1%.

7. Erosion and Sediment Controls

All erosion and sediment controls required during construction are to be in accordance with the Greater Golden Horseshoe Area Conservation Authorities ESC Guideline entitled Erosion and Sediment Control Guidelines for Urban Construction.

The following principles should be adhered to for all development within the subject lands *(Ref. Erosion and Sediment Control Guidelines for Urban Construction):*

- Adopt a multi-barrier approach to provide erosion and sediment control through erosion controls first,
- Retain existing ground cover and stabilize exposed soils with vegetation where possible,
- Limit the duration of soil exposure and phase construction where possible,
- Limit the size of disturbed areas by minimizing nonessential clearing and grading,
- Minimize slope length and gradient of disturbed areas,
- Maintain overland sheet flow and avoid concentrated flows,
- Store/stockpile soil away (e.g. greater than 30m) from watercourses, drainage features and top of steep slopes,
- Ensure contractors and all involved in the ESC practices are trained in ESC Plan, implementation, inspections, maintenance, and repairs,
- Adjust ESC Plan at construction site to adapt to site features,
- Assess all ESC practices before and after all rainfall and significant snowmelt events, and
- Maintain record of inspection on site.

7.1. Control Measures

- Install all silt fences prior to any other activities on site,
- Any dewatering of the site should incorporate sediment and debris screens at the inlet of the pumping system and sediment settling facilities at discharge points, to prevent sediment discharges to the municipal drainage system.
- Construct temporary construction access including mud mat at construction access points,
- Construct cut-off swales as shown on the Sediment and Erosion Control Plan,
- Install sediment socks as indicated on the Erosion and Sediment Control Plan,

- Contractor to use haulage roads to minimize disturbances to existing vegetated areas,
- During the servicing construction, limit open trench lengths to minimize erosion potential,
- During work stoppages or inclement weather, plug ends of open sewers to prevent downstream sedimentation,
- Provide catchbasin sediment protection on all catchbasins for the duration of construction,
- Provide dust control during dry periods as directed by the site engineer,
- Sweep external streets as directed by the engineer,
- Following base course asphalt, catchbasins to be rewrapped with geotextile, and
- Periodic street cleaning and catchbasin cleanout to be performed as required.

7.2. Inspection and Maintenance

The following is a minimum inspection schedule that should be adhered to for the full length of the construction period. *(Ref. Erosion and Sediment Control Guidelines for Urban Construction)*

- All erosion and siltation measures are to be inspected on a weekly basis by the Consultant's resident inspector.
- All erosion and siltation measures are to be inspected after every rainfall event by the Consultant's resident inspector.
- All erosion and siltation measures are to be inspected after significant snowfall events by the Consultant's resident inspector.
- All erosion and siltation measures are to be inspected daily during extended rain or snowmelt periods by the Consultant's resident inspector.
- Maintain record of inspection on site.

During inactive construction periods, where the site is left alone for 30 days or longer, monthly inspections should be conducted.

8. Conclusions

Based on our analysis, the conceptual development scenario can be adequately serviced with storm, sanitary and water servicing and can be designed to meet the City of Vaughan objectives.

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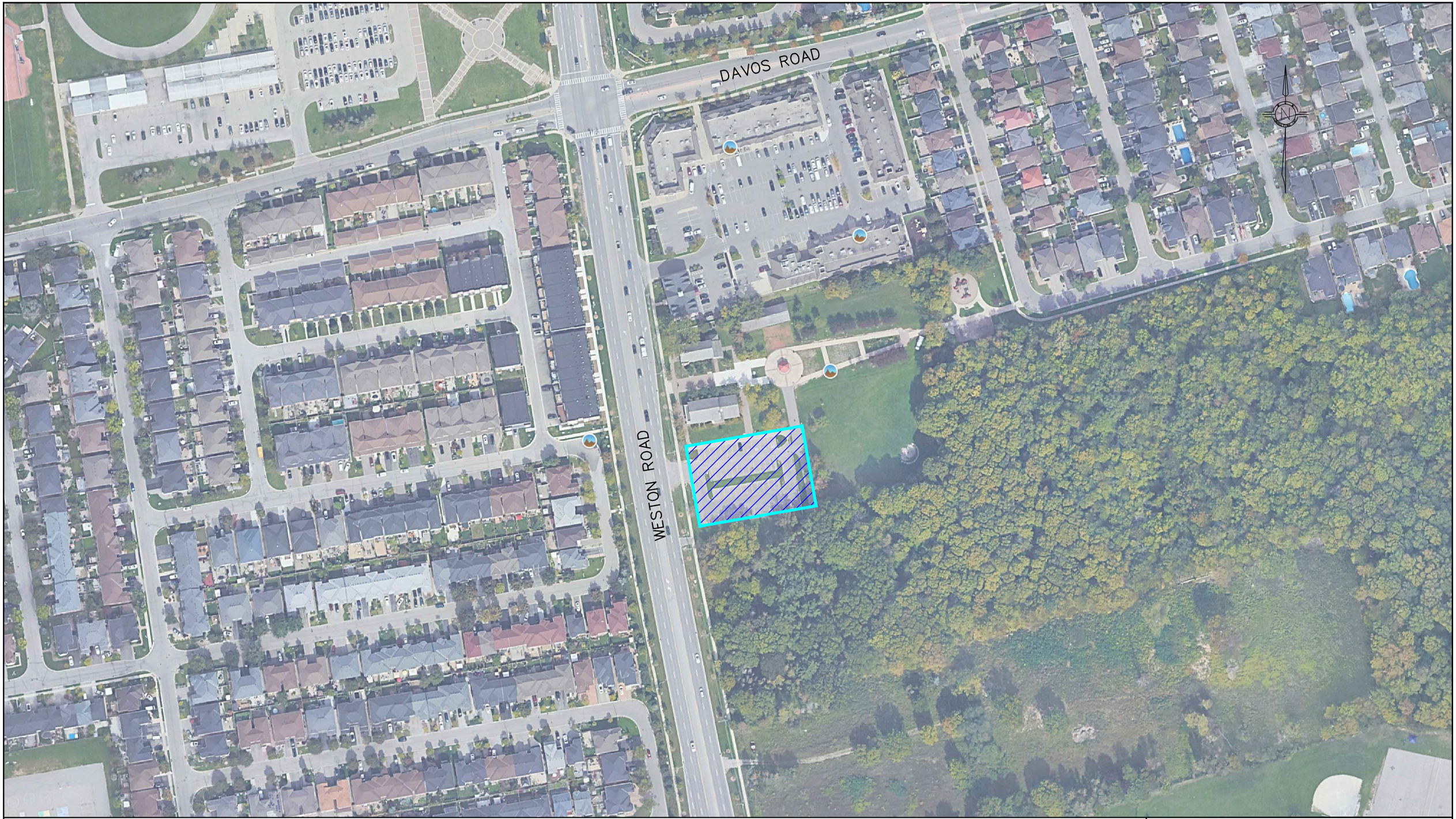
FIGURES



VAUGHAN FIRE STATION 7-12
9541 WESTON ROAD, WOODBRIDGE, ONTARIO

LOCATION PLAN

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FIGURE #1

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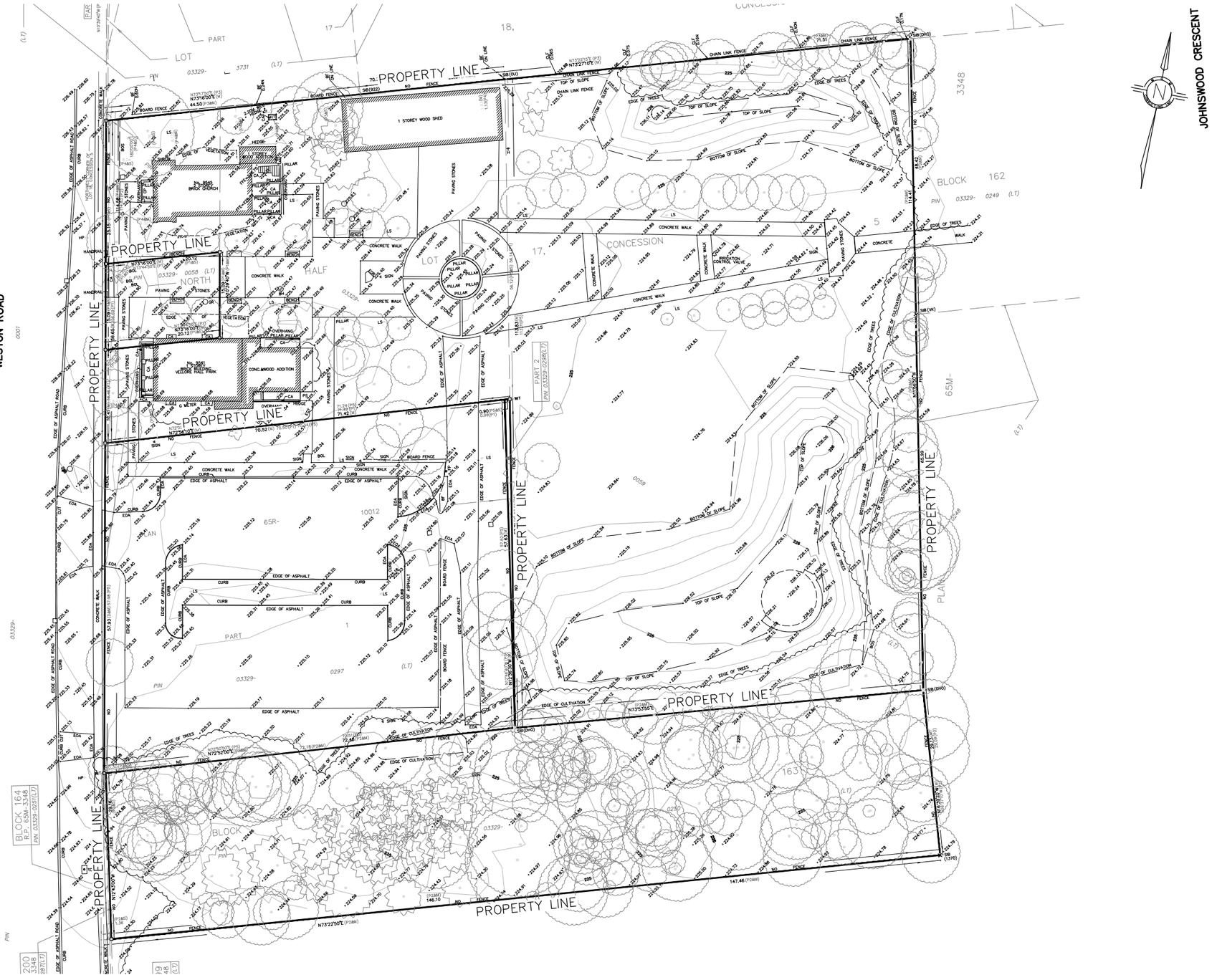
SCALE: N.T.S.
DWG#: 2022-007 C1



VAUGHAN FIRE STATION 7-12
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EXISTING CONDITIONS

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FIGURE #2

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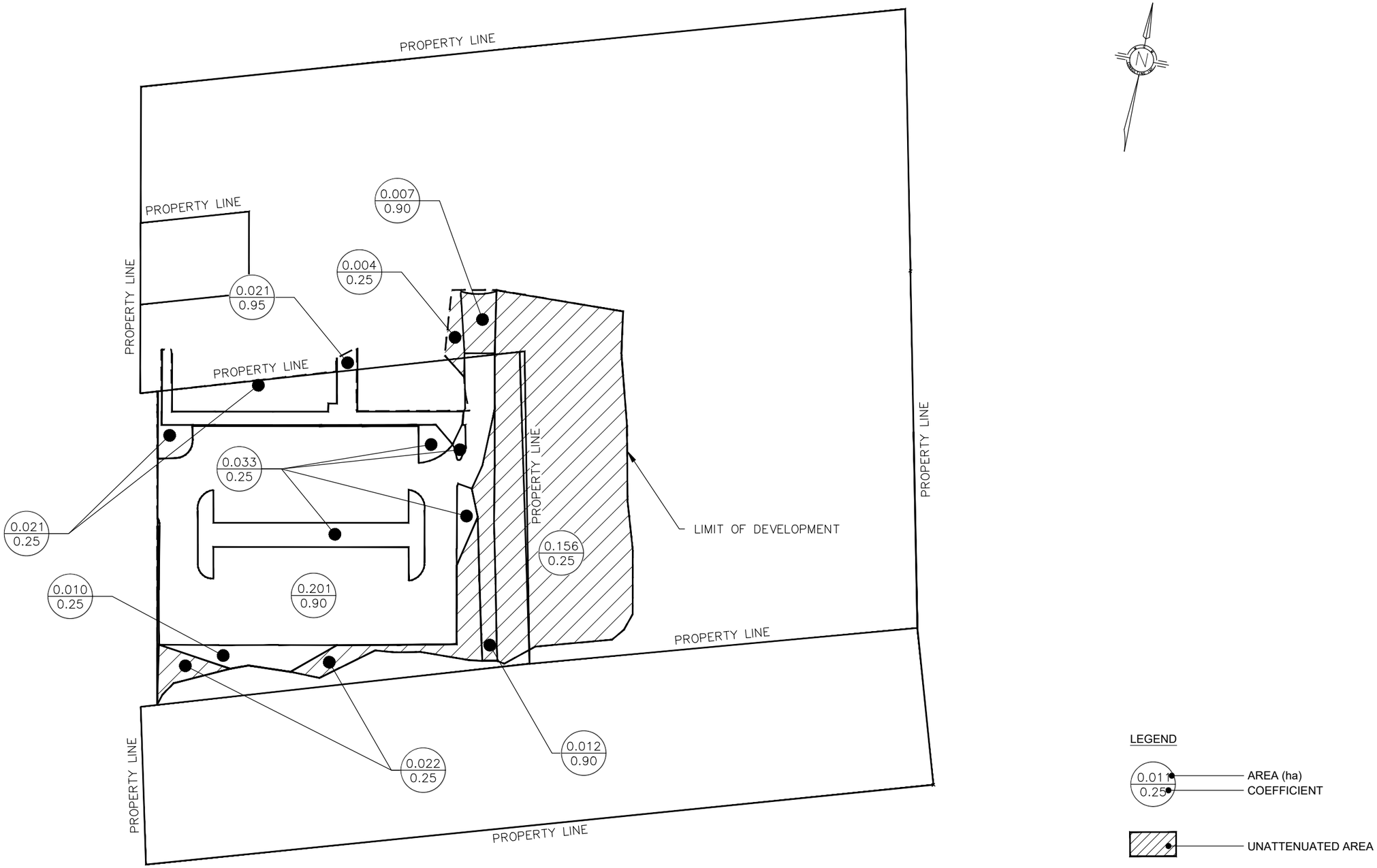

VAUGHAN FIRE STATION 7-12
9541 WESTON ROAD, WOODBRIDGE, ONTARIO

PRE-DEVELOPMENT DRAINAGE AREAS

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FIGURE #3

DATE: APR. 2022
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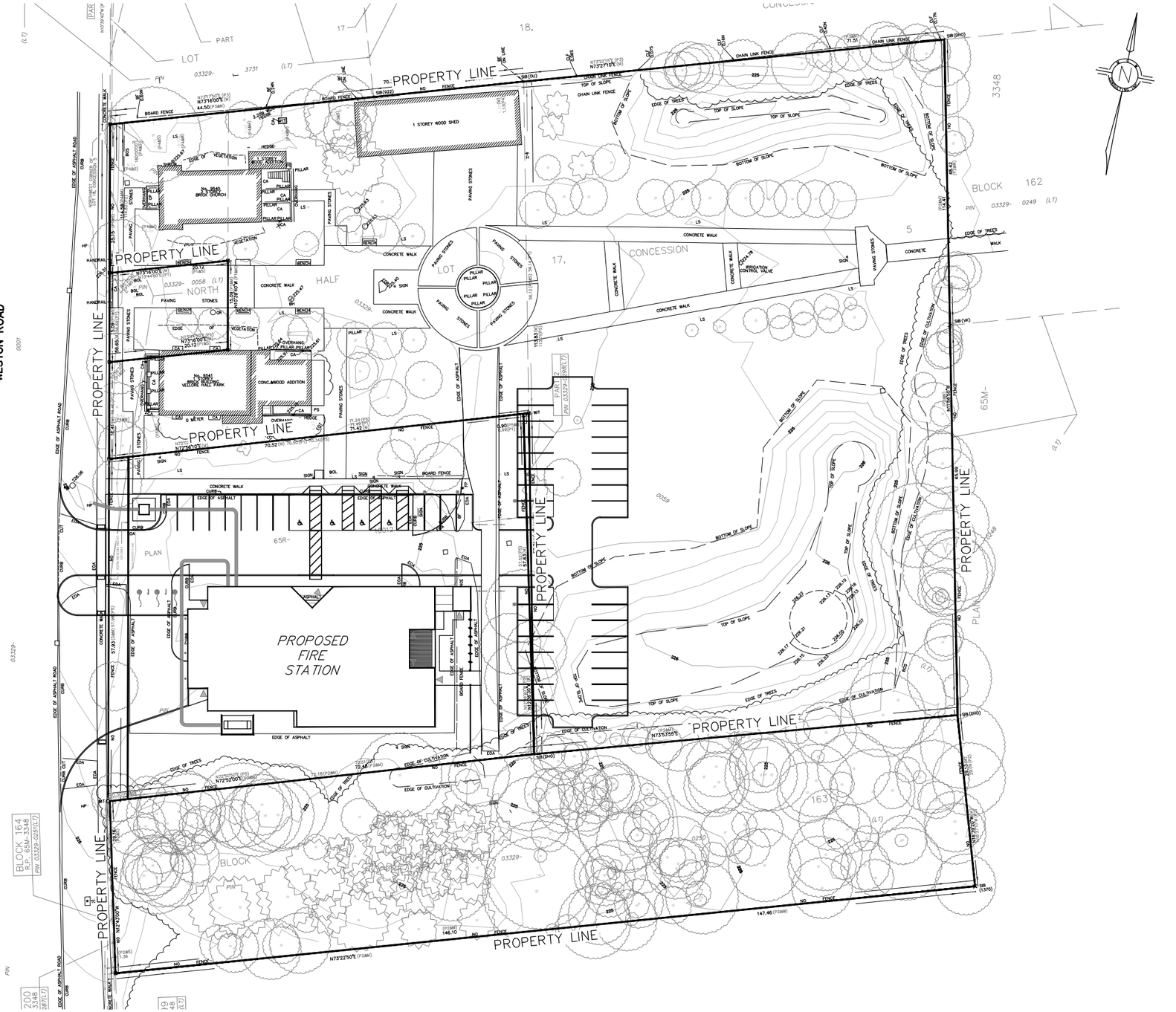

VAUGHAN FIRE STATION 7-12
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PROPOSED DEVELOPMENT

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FIGURE #4

DATE: APR. 2022
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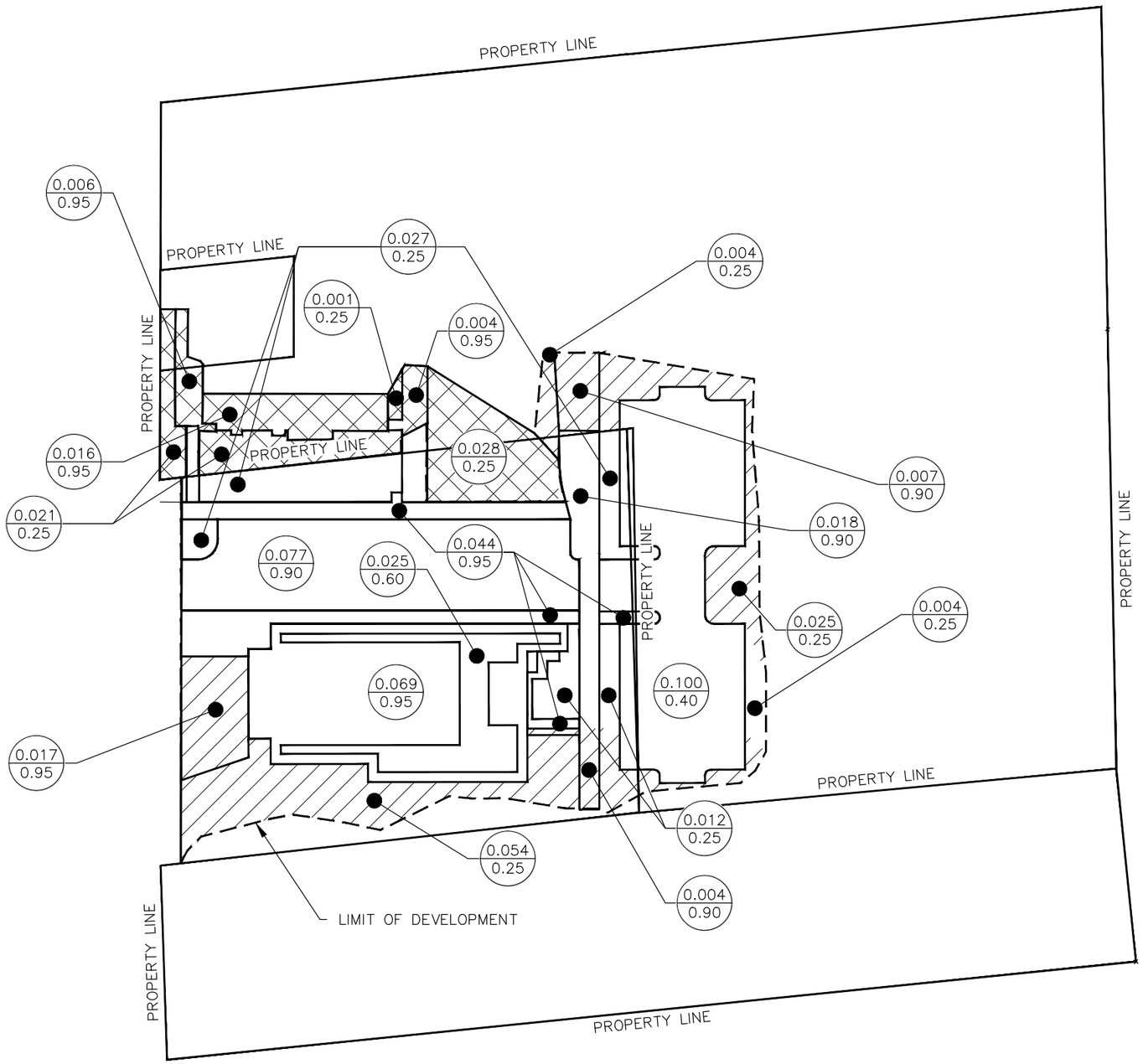

LEGEND

0.011 / 0.25 — AREA (ha) / COEFFICIENT

UNATTENUATED AREA

EXTERNAL DRAINAGE AREA

VAUGHAN FIRE STATION 7-12
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POST-DEVELOPMENT DRAINAGE AREAS

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FIGURE #5

DATE: APR. 2022
SCALE: 1:750
DWG#: 2022-007 C1

APPENDIX A

STORMWATER MANAGEMENT CALCULATIONS

STORMWATER MANAGEMENT CALCULATIONS

9541 Weston Road, Woodbridge, Ontario

1.0 Drainage Characteristics

1.1 Existing Drainage Areas: (see Figure No.3)

		"c"	Area (ha)
Attenuated Areas:			
	Asphalt Pavement	0.90	0.205
	Uncontrolled Roof	0.95	0.000
	Concrete	0.95	0.021
	Landscape Area	0.25	0.064
	Subtotal Area (Ha)		**0.290**
Unattenuated Areas:			
	Asphalt Pavement	0.90	0.019
	Uncontrolled Roof	0.95	0.000
	Concrete	0.95	0.000
	Landscape Area	0.25	0.182
	Subtotal Area (Ha)		**0.201**
	Total Area (Ha)		**0.491**
	Average Site Imperviousness		**0.58**

1.2 Proposed Drainage Areas (see Figure No.4)

		"c"	Area (ha)
Attenuated Areas:			
	Concrete	0.95	0.044
	Asphalt Pavement	0.90	0.095
	Controlled Green Roof	0.60	0.025
	Controlled Roofs	0.95	0.069
	Uncontrolled Roofs	0.95	0.000
	Landscape Area	0.25	0.043
	Permeable Paver	0.40	0.100
	Subtotal Area (Ha)		**0.376**

		"c"	Area (ha)
Unattenuated Areas:			
	Landscape Area	0.25	0.085
	Asphalt Area	0.90	0.013
	Roof	0.95	0.000
	Concrete	0.95	0.017
	Subtotal Area (Ha)		**0.115**

		"c"	Area (ha)
External Areas:			
	Landscape Area	0.25	0.050
	Roof	0.95	0.016
	Concrete	0.95	0.010
	Subtotal Area (Ha)		**0.076**
	Total Area (Ha)		**0.567**
	Average Site Imperviousness		**0.54**

2.0 Allowable Post Development Flows

The allowable flow from the site should be based on unitary flow rates according to Equation G - Sub Basin 46 of Humber River Stormwater Management Quantity Control Release Rates drawing of the TRCA SWM criteria (2012) document.

For example,
Based on Equation G,
For 2 year storm,

Q =	7.745-0.762*ln(A)		
Area of the site =		0.567	ha
Therefore Q allowable =		0.0046	**cms**

Similarly, table below calculates the allowable flows for storm events from 2 to 100 years.

Storm Event (years)	a	b	c	Equation G - Unitary Flow Rate	A (ha)	Q (allow.) (cms)
2	647.7	4	-0.784	7.745-0.762 * ln(A)	0.567	0.0046
5	929.6	4	-0.798	11.468-1.123 * ln(A)	0.567	0.0069
10	1021	3	-0.787	13.877-1.342 * ln(A)	0.567	0.0083
25	1100	2	-0.776	17.381-1.690 * ln(A)	0.567	0.0104
50	1488	3	-0.803	20.164-1.973 * ln(A)	0.567	0.0121
100	1770	4	-0.82	22.973-2.256 * ln(A)	0.567	0.0138

Since achiving allowable flows for storm events from 2 to 100 years is not practical because of the site characteristics, a 75 mm orifice tube is proposed that will provide the required storage volume based on the 100 year unitary flow rate per Equation G.

3.0 Rooftop & Orifice Controlled Flow Calculations

3.1 Roof Storage

Controlled Roof Area =	0.094	ha
Total No. of Hoppers=	4	each
1 weir per Hopper	1	each
Weir Rating=	0.015	l/sec/mm
Max ponding depth =	150	mm.
Max. Storage provided =	47.0	cu.m.
Peak Flow from roof =	0.0090	m^3/sec.

3.2 Orifice Control Calculations

An orifice tube is proposed at the outlet of MH2, discharging to OGS.

Orifice Equation:

Qa = CA*(2gh)^1/2, where

C = entrance loss coefficient =	0.62	
A = cross section area =	0.004418	m^2
g = gravitational acceleration =	9.81	m/sec^2
Selected Orifice Diameter =	75	mm.
Orifice tube invert=	223.95	
Orifice tube centerline elev.=	223.99	

3.2.1 One Hundred Year Controlled Flow Calculation

100-year ponding elevation	225.20	
100-year Head	1.21	m.
Therefore Q100	0.0134	cms.

4.0 Required Storage Calculations:

4.3 100 Year Storage Calculation

Rainfall Duration min.	s	100 Year Rainfall Intensity (I) mm/h	Controlled Roof Flow m^3/s	Uncontrolled Flow m^3/s	Unattenuated Flow m^3/s	Controlled Orifice Flow m^3/s	Aprox. Detention Volumes m^3
10	600	203.3	0.0090	0.1216	0.0277	0.0134	70.3
20	1200	130.7	0.0090	0.0781	0.0178	0.0134	88.5
30	1800	98.2	0.0090	0.0587	0.0134	0.0134	97.9
40	2400	79.5	0.0090	0.0475	0.0108	0.0134	103.6
50	3000	67.2	0.0090	0.0402	0.0092	0.0134	107.5
60	3600	58.5	0.0090	0.0350	0.0080	0.0134	110.2

70	4200	51.9	0.0090	0.0310	0.0071	0.0134	112.0
80	4800	46.8	0.0090	0.0280	0.0064	0.0134	113.3
90	5400	42.7	0.0090	0.0255	0.0058	0.0134	114.2
100	6000	39.3	0.0090	0.0235	0.0054	0.0134	114.7
110	**6600**	**36.4**	**0.0090**	0.0218	**0.0050**	**0.0134**	**114.9**
120	7200	34.0	0.0090	0.0203	0.0046	0.0134	114.9
130	7800	31.9	0.0090	0.0191	0.0044	0.0134	114.8
140	8400	30.1	0.0090	0.0180	0.0041	0.0134	114.4
150	9000	28.5	0.0090	0.0170	0.0039	0.0134	113.9
160	9600	27.0	0.0090	0.0162	0.0037	0.0134	113.3

The peak flow during a 100 year storm event (attenuated flow + unattenuated flow) = 0.0183 cms.

4.4 Summary of Storage Requirements

The total flow from the site during the 100 year storm event (attenuated + unattenuated flows) = 0.0183 cms

5.0 On-Site Storage Provided

On-site storage is proposed within the pipes.

5.1 Pipe Storage

	Length (m)	Width (m)	AREA (m2)	Volume (m3)
Infiltration Gallery-OGS	3.6	200	0.031	0.11
OGS-MH2	3.3	375	0.110	0.36
MH2-MH3	3.0	250	0.049	0.15
MH3-CBMH4	19.6	250	0.049	0.96
CBMH4-CB2	13.1	200	0.031	0.41
MH2-CBMH5	6.2	375	0.110	0.68
CBMH5-MH6	7.1	200	0.031	0.22
MH6-CB1	16.8	200	0.031	0.53
CBMH5-CBMH7	37.8	375	0.110	4.17
CBMH7-MH8	4.1	250	0.049	0.20
			SUBTOTAL =	**7.81**

5.2 StormBrixx Storage Sizing

Area of Stormbrixx (m^2)		120.96
Depth of Stormbrixx (m)		0.914
Void Ratio (%)		97
Available Volume for Detention (m^3)	**SUBTOTAL=**	**107.24**
	TOTAL=	**115.05**

6.0 Water Balance Calculations

Water balance requirements for the site is to retain the required infiltration based on pre to post infiltration difference through retention, evapotranspiration and/or infiltration.

6.1 Storage & Infiltration Time of Infiltration Pit

6.1.1 Existing Infiltration Pit

The volume available within the Infiltration Pit is as follows:

Length of pit=	12	
Width of pit=	7	
Area of pit=	84	
Depth of pit=	0.5	
Volume of pit=	42	cu.m.
Porosity of Stone	40	%
Available Storage in pit=	16.80	cu.m.

6.1.2 Proposed Infiltration Pit

The volume available within the Infiltration Pit is as follows:

Length of pit=	36.5	
Width of pit=	5	
Area of pit=	182.5	
Depth of pit=	0.5	
Volume of pit=	91	cu.m.
Porosity of Stone	40	%
Available Storage in pit=	36.50	cu.m.

From Geotech Report, percolation rate of 41.60mm/hr for infitration trenches, and a factor of safety of 2.5 is used.

Assumed Infiltration rate =	41.60	mm/hr
Factor of safety =	2.50	
Design infiltration rate =	16.64	mm/hr

The estimated time to infiltrate water contained within the sand filter is as follows:

Depth of pit =	500.00	mm
Time to infiltrate =	30.05	hrs

6.3 Water Balance Provided

6.3.1 Water Balance Provided over the Construction Limit Area

Water Re-use Provided:

Description	Area (sq.m.)	Initial Abstraction (mm)	Volume (cu.m.)
Soft Landscaping	430	5	2.15
Asphalt/Gravel	950	1	0.95
Permeable Pavers	1000	5	5.00
Roof	690	1	0.7
Green Roofs	250	7	1.8
Concrete	440	1	0.4
Infiltration Pit			53.3
Total	3760		64.3

Pro-rated rainfall depth over the construction limit area = 17.1 mm.

7.0 Summary

Item	Storage Required (cu.m)	Max Storage Provided (cu.m)	Actual Discharge (cms)
Water balance	-	53.3	-
100 Year Flood	114.9	115.1	0.0183

APPENDIX B

STORM SEWER DESIGN SHEET

MGM CONSULTING INC.
5-YEAR STORM SEWER DESIGN

Project No.: 2022-007
Subdivision:
Date: 12-Jun-23 Revised: -
Des. By: AB Chk. By: KL

Location				Areas		A * C			Rainfall			Sewer Design						Surcharge					
Manhole from	Invert	Manhole to	Invert	Area	Cumulative Area	Coefficent	Incremental A * C	Cumulative A * C	Time	Intensity 5-year	Q Total	Equiv. Circ. Pipe Diameter	Slope	Max. Flow Q cap	Max Velocity V max	Length	Time in Section	Actual Flow to Max. Allowable Flow Ratio					
	m.		m.	ha	ha	C			min	mm/hr.	cms	mm.	%	cms	m./sec.	m.	min.	%	0.25	0.40	0.60	0.90	0.95
CB1		MH6		0.042	0.042	0.40	0.017	0.017	7.0	137.2	0.006	200	0.70	0.027	0.88	16.8	0.32	23%		0.042			
MH6		CBMH5		0.000	0.042	0.00	0.000	0.017	7.3	134.1	0.006	200	0.50	0.023	0.74	7.1	0.16	27%					
CB2		CBMH4		0.066	0.066	0.42	0.028	0.061	7.0	137.2	0.023	200	0.70	0.027	0.88	13.1	0.25	85%	0.016	0.043		0.005	0.002
CBMH4		MH3		0.017	0.083	0.42	0.007	0.068	7.2	134.7	0.026	250	0.50	0.042	0.86	19.6	0.38	61%	0.001	0.015			0.001
MH3		MH2		0.000	0.083	0.00	0.000	0.068	7.6	131.2	0.025	250	0.50	0.042	0.86	3.0	0.06	59%					
BLDG		MH8		0.094	0.094	0.86	0.081	0.081	7.0	137.2	0.031	250	1.00	0.060	1.21	6.1	0.08	52%			0.025		0.069
MH8		CBMH7		0.000	0.094	0.00	0.000	0.081	7.1	136.3	0.031	250	0.50	0.042	0.86	4.1	0.08	72%					
CBMH7		CBMH5		0.122	0.216	0.74	0.090	0.171	7.2	135.6	0.064	375	0.50	0.124	1.12	37.8	0.56	52%	0.034			0.038	0.050
CBMH5		MH2		0.102	0.360	0.69	0.071	0.258	7.5	132.6	0.095	375	0.50	0.124	1.12	6.2	0.09	77%	0.034			0.052	0.016
MH2		OGS		0.000	0.443	0.00	0.000	0.326	7.7	130.7	0.119	375	0.50	0.124	1.12	3.3	0.05	95%					
OGS		Ex. Infiltration Gallery		0.000	0.443	0.00	0.000	0.326	7.7	130.3	0.118	375	1.00	0.176	1.59	2.9	0.03	67%					

n = 0.013

$$5\,Year\ I = \frac{929.6}{(T + 4.0)^{0.798}}\ \text{mm/hr}$$

5 Year Storm
Intial T_c = 7 minutes

APPENDIX C

FIRE DEMAND CALCULATIONS

FIRE FLOW REQUIREMENT

Proposed 1-Storey Fire Station,
9541 Weston Road, Woodbridge

Project # 2022-007
DATE: May 2022

Design Note

For non-combustible construction, the area shall be a total of all floors (excluding basements at least 50 percent below grade) in the building being considered.

Gross floor area = 940.93 m^2

Base Flow Calculation

A=	Effective area	941	m^2
C=	Wood frame	1.5	
F=	Required fire flow	10,123	L/min.
	Flow Rounded up to nearest 1,000	10,000	L/min

Flow 'F' Adjustments				**Credits (L/min)**	**Charges (L/min)**	**Flow Adjusted (L/min)**
Occupancy Adjustment	%					
Combustible	0%			0		10,000
Sprinkler Adjusments						
Sprinklered as per NFPA 13	Yes			-3,000		
Standard Water Supply	Yes			-1,000		
Fully supervised watersupply	No					6,000
Exposure Adjustments						
Exposure	Sep. (m)	Charge				
North	27.6	10%	Building			
East	>45	0%	Road			
South	>45	0%	Woodlot			
West	>45	0%	Park			
Total Exposure Charge		10%			1000	7,000

Total Required Flow (Flow Rounded up to nearest 1,000) L/min = 7000 L/min

or

117 L/s

APPENDIX D

WATER DEMAND CALCULATIONS

Water Demand Calculations
Proposed 1-Storey Fire Development,
9541 Weston Road, Woodbridge

Project # 2022-007
DATE: May 2022

Water Connection

Connection Point	Existing 150mm dia. Watermain located North of proposed development

Population to be Serviced		
Industrial (70 persons/ha)	9.0	
Commercial	0.0	
Residential	0.0	
	Total Population	9.00

Hydrant flow test

Hydrant flow test location	N/A

	Presure (kPa)	Flow (in L/s)	Time
Minimum water pressure	***Refer to Fire Hydrant Flow Test Results***		
Maximum water pressure			

Water Demands

No.		Commercial	Industrial (L/s)	Residential (L/s)	Total (L/s)
1	Average day flow[1]	0.0000	0.0300	0.0000	0.0300
2	Maximum day flow[2]	0.0000	0.0700	0.0000	0.0700
3	Peak hour flow[3]	0.0000	0.0700	0.0000	0.0700
4	Fire flow[4]				117

Analysis

5	Maximum day plus fire flow		**117.07 L/s**
		or	**7024.20 L/min**
		or	**1855.60 usgpm**

APPENDIX E

SANITARY FLOW CALCULATIONS

Proposed 1-Storey Fire Station,
9541 Weston Road, Woodbridge

SANITARY DESIGN FLOWS:

Total Site Area:		0.49	ha
Institutional Usage (Fire Station)			
Land Use	Total Population		
Institutional	25.0		
Gross floor area		0.094	ha
Population (P):		25	persons
Institutional Sewage Generation Rate:		450	l/cap/day
Institutional Residential Sewage Flow:		0.53	l/s
Peak Factor:	1+(14/(4+sqrt(P/1000))	4.367	
Peak Institutional Flow:		2.31	l/s
Infiltration Allowance	0.26 l/s/ha	0.02	l/s
Peak Design Flow:		2.34	l/s

Total Sanitary Usage **2.34 l/s**

Notes:
1. Institutional flows are calculated based on an average flow of 450 l/cap/day
2. Infiltration is calculated based on the area contributing to the subject sewer segment at a rate of 0.26 L/s/ha
3. Institutional peaking factor is calculated based on the modified Harmon formula

APPENDIX F

OGS SIZING REPORT –

HYDROSTORM UNIT

Hydroworks Sizing Summary

9541 Weston Rd
Woodbridge, Ontario

05-11-2022

Recommended Size: HydroStorm HS 4

A HydroStorm HS 4 is recommended to provide 80 % annual TSS removal based on a drainage area of .444 (ha) with an imperviousness of 35 % and Toronto Central, Ontario rainfall for the City of Toronto particle size distribution.

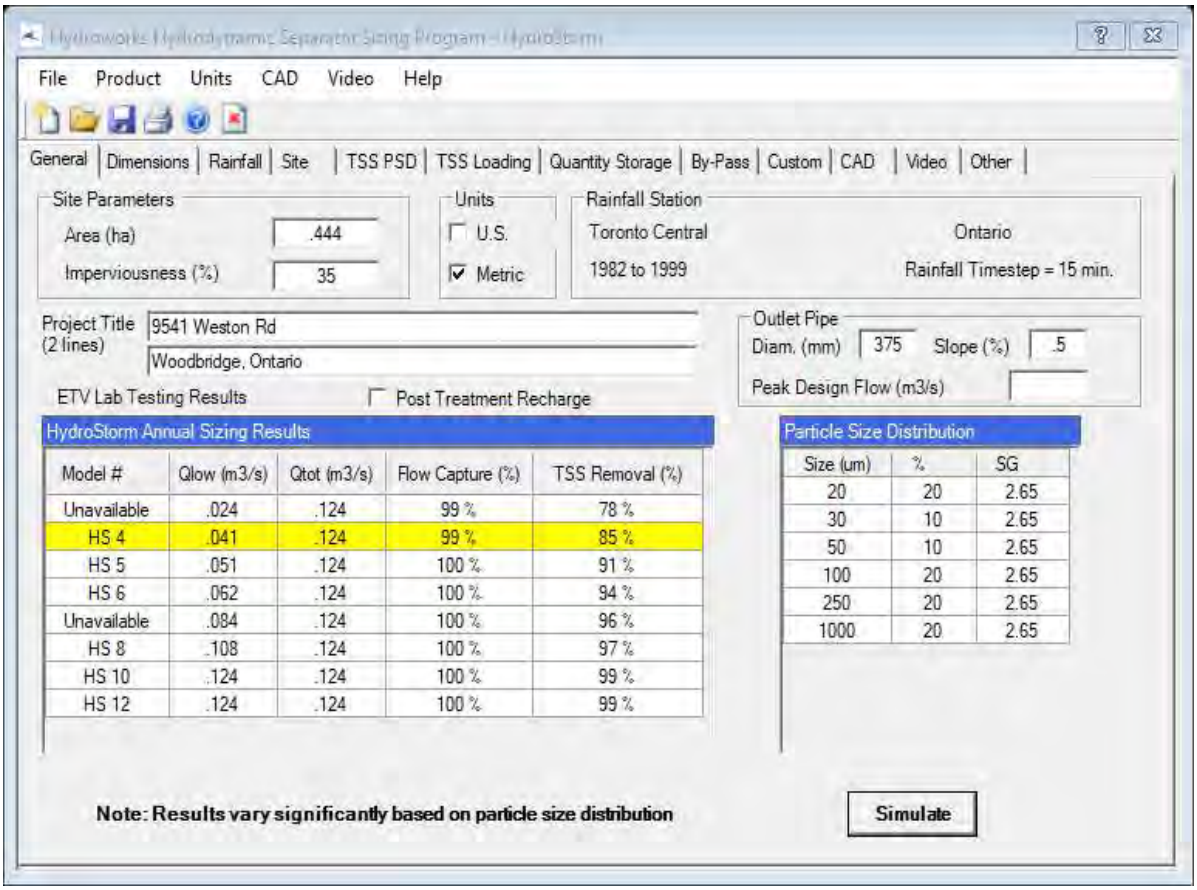
The recommended HydroStorm HS 4 treats 99 % of the annual runoff and provides 85 % annual TSS removal for the Toronto Central rainfall records and City of Toronto particle size distribution.

The HydroStorm has a headloss coefficient (K) of 1.04. Since a peak flow was not specified, headloss was calculated using the full pipe flow of .12 (m3/s) for the given 375 (mm) pipe diameter at .5% slope. The headloss was calculated to be 67 (mm) based on a flow depth of 375 (mm) (full pipe flow).

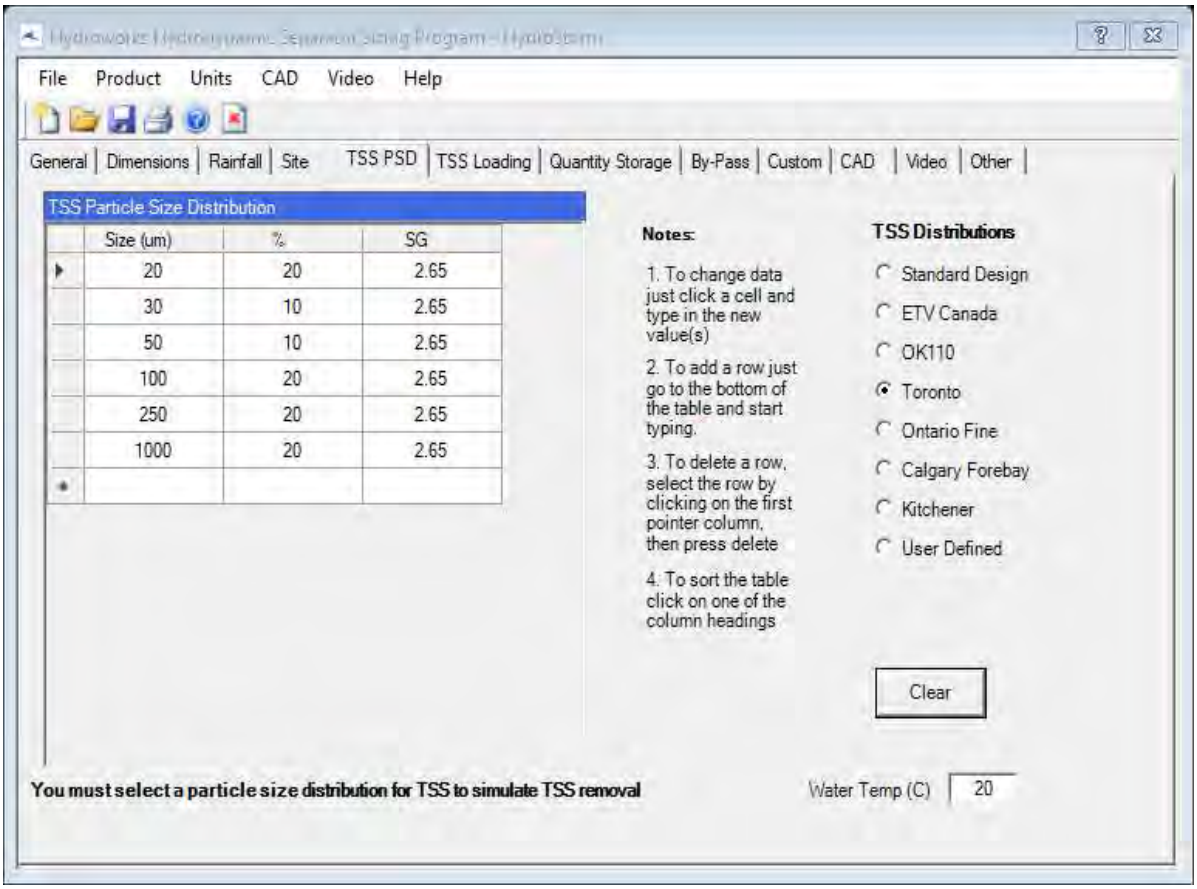
This summary report provides the main parameters that were used for sizing. These parameters are shown on the summary tables and graphs provided in this report.

If you have any questions regarding this sizing summary please do not hesitate to contact Hydroworks at 888-290-7900 or email us at support@hydroworks.com.

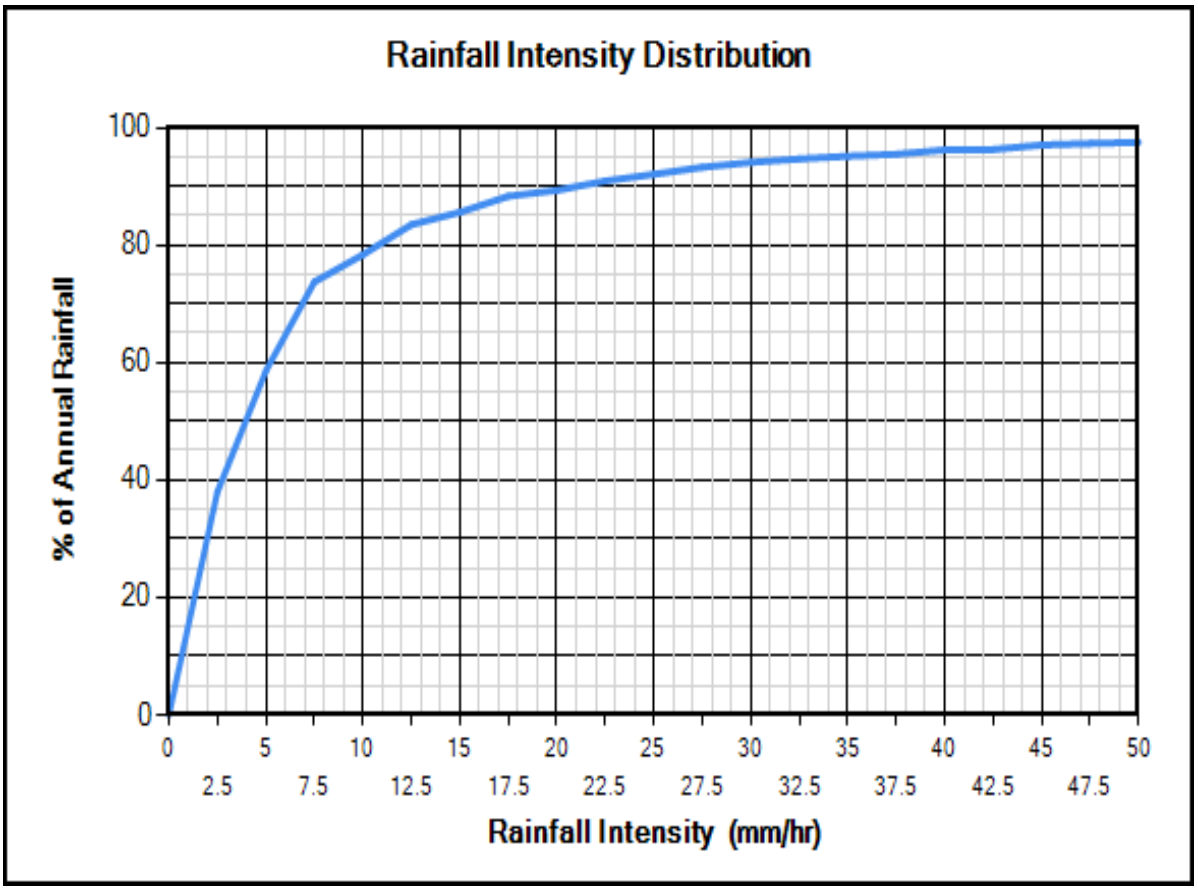
TSS Removal Sizing Summary



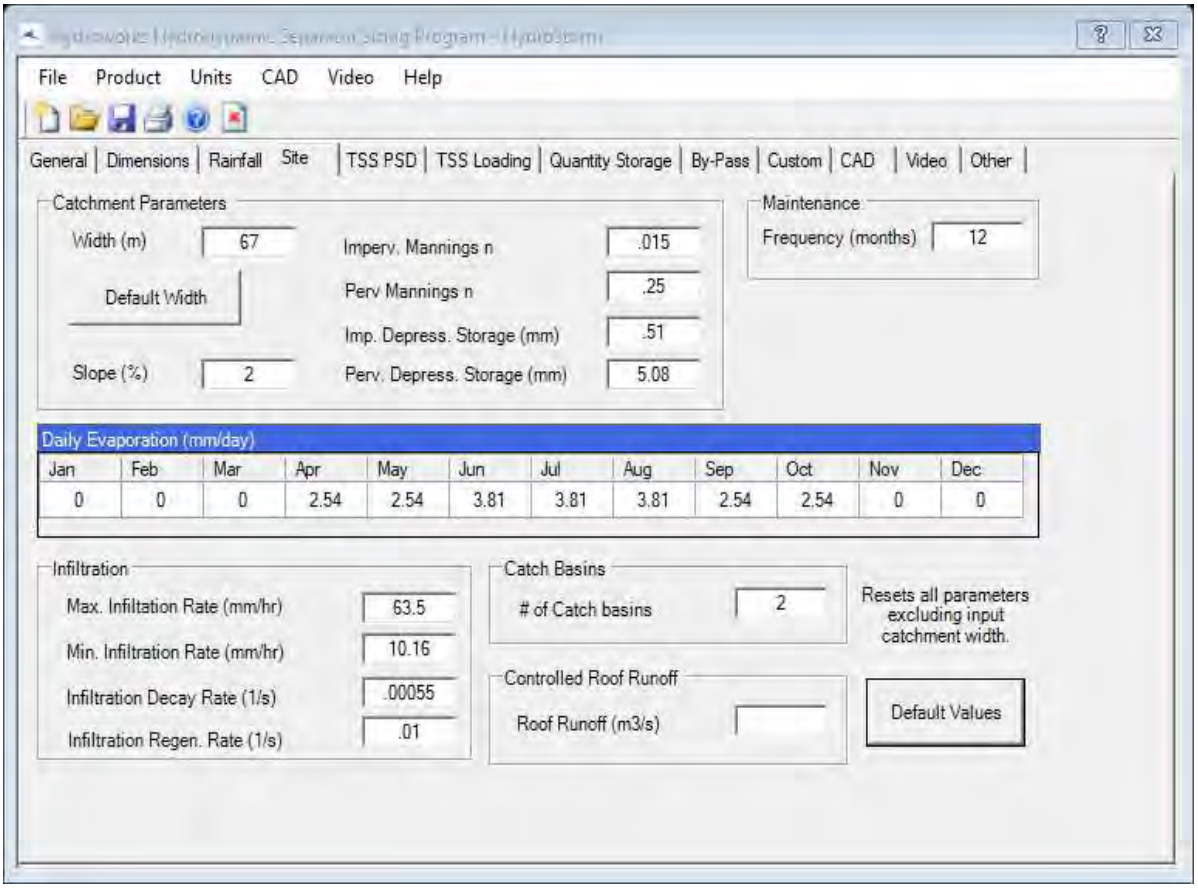
TSS Particle Size Distribution



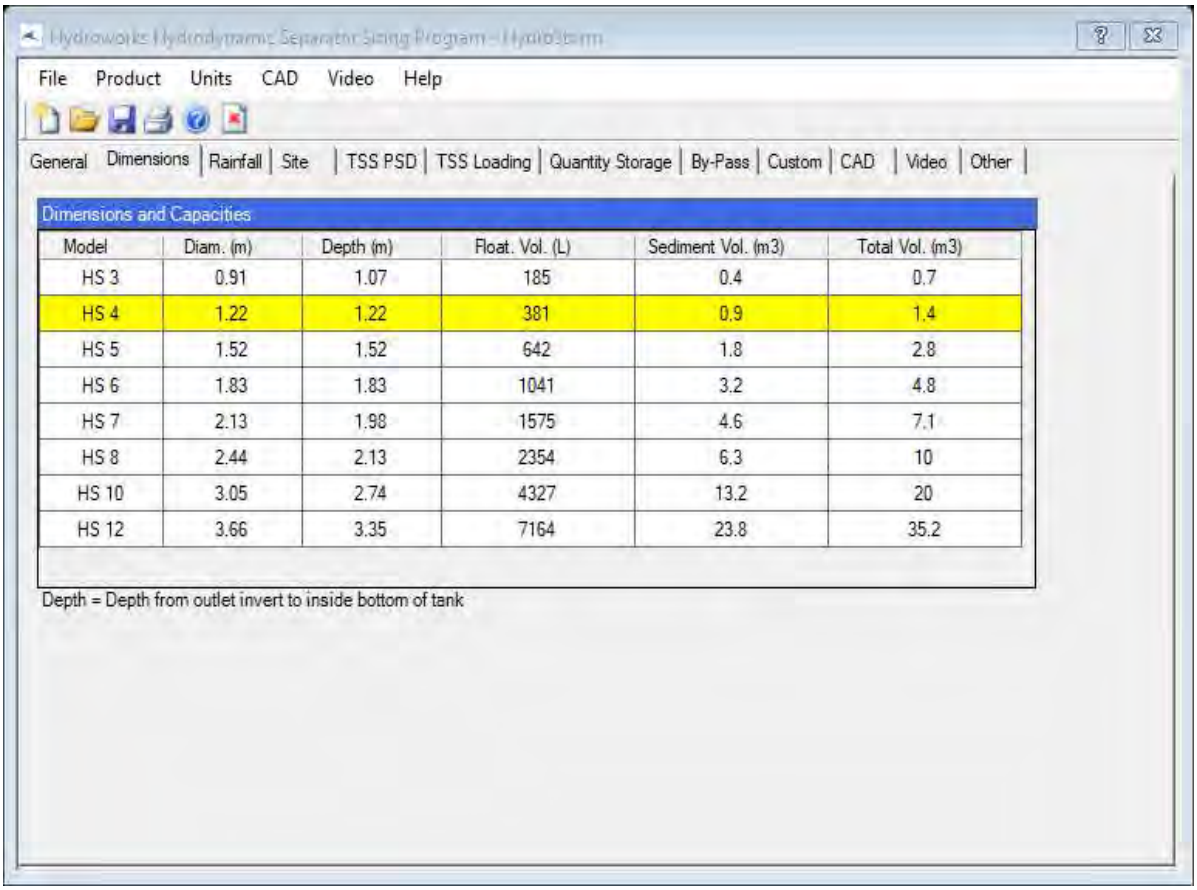
Rainfall Station - Toronto Central, Ontario(1982 to 1999)



Site Physical Characteristics



Dimensions And Capacities

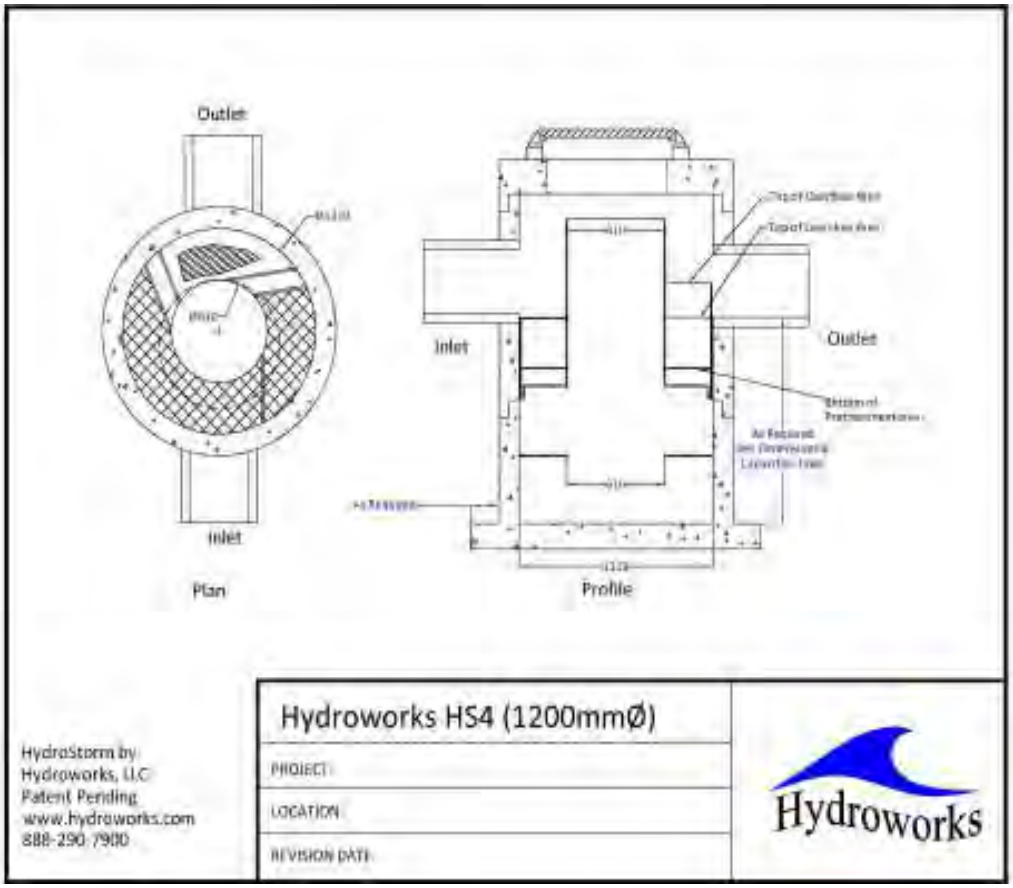


Dimensions and Capacities

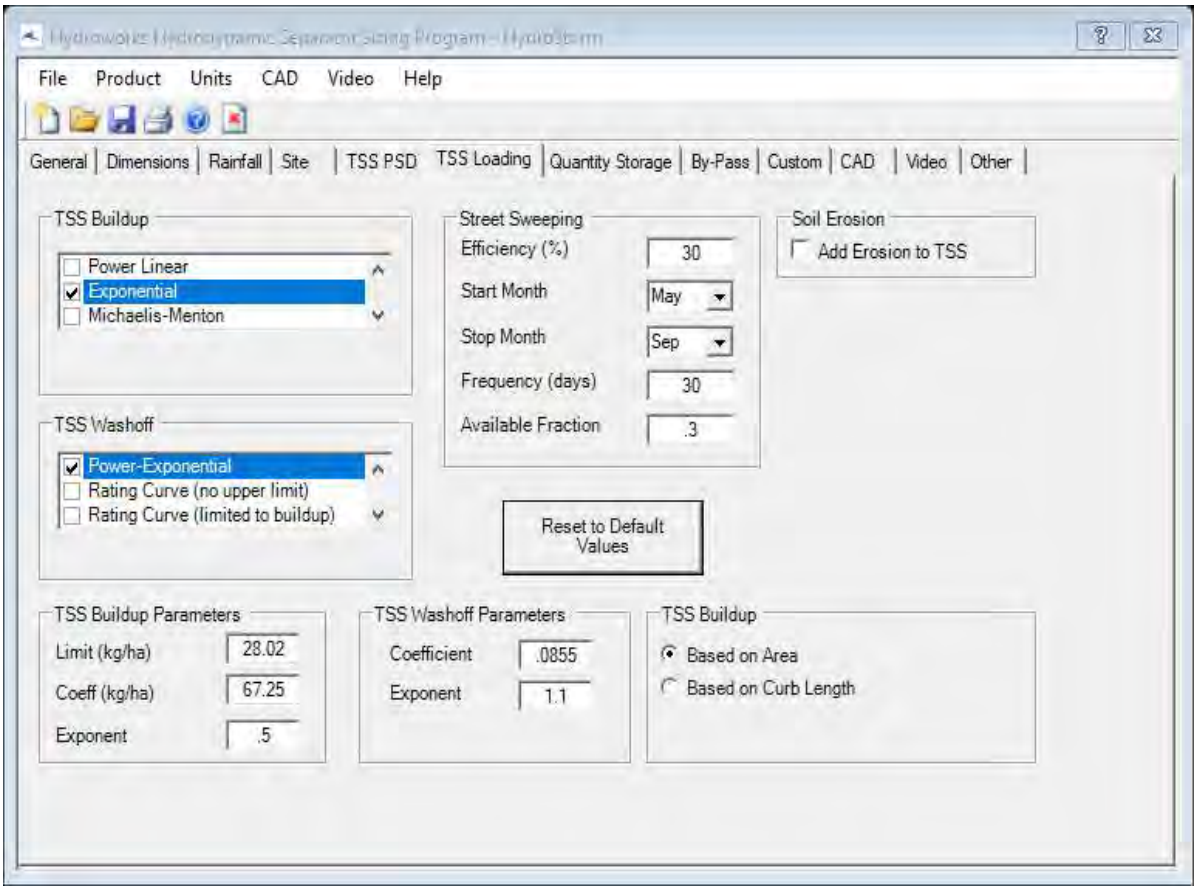
Model	Diam. (m)	Depth (m)	Float. Vol. (L)	Sediment Vol. (m3)	Total Vol. (m3)
HS 3	0.91	1.07	185	0.4	0.7
HS 4	1.22	1.22	381	0.9	1.4
HS 5	1.52	1.52	642	1.8	2.8
HS 6	1.83	1.83	1041	3.2	4.8
HS 7	2.13	1.98	1575	4.6	7.1
HS 8	2.44	2.13	2354	6.3	10
HS 10	3.05	2.74	4327	13.2	20
HS 12	3.66	3.35	7164	23.8	35.2

Depth = Depth from outlet invert to inside bottom of tank

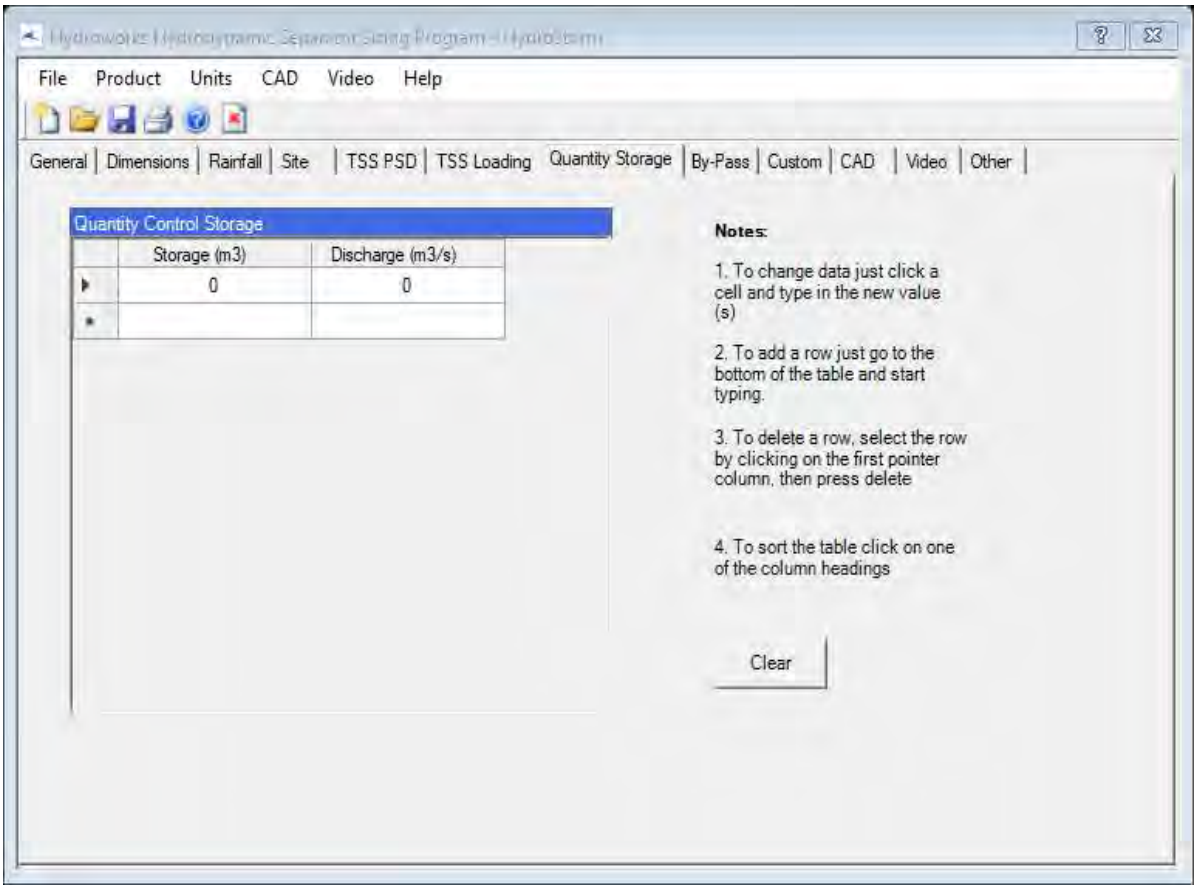
Generic HS 4 CAD Drawing



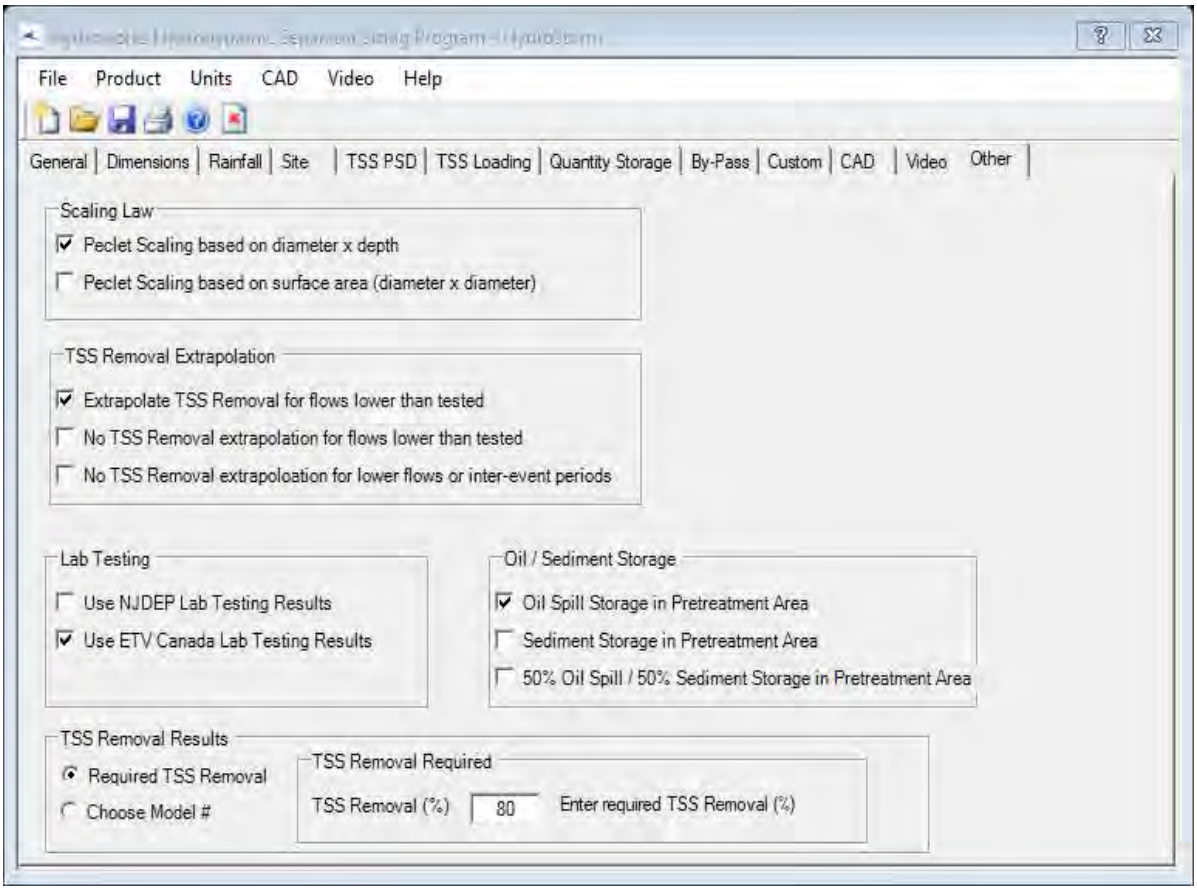
TSS Buildup And Washoff



Upstream Quantity Storage



Other Parameters



Hydroworks Sizing Program - Version 5.5
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APPENDIX G

COMBINED TSS REMOVAL

CALCULATIONS

Combined TSS Removal Calculations for Treatment Options in Series (Point Source Inlet Flows)

Catchbasin Sediment Traps / OGS Treatment Train

R= A + B - [(AxB)/100]

A= Catch Basin Shield
B= OGS Stormceptor

R= Total TSS Removal Rate
A= First Removal (upstream)
B= Second Removal (downstream)

A =	64	%
B =	50	%
Total TSS Removal Rate =	82	%

APPENDIX H

HYDRANT TEST RESULTS

HYDRANT INSPECTION & FLOW REPORT

SUGGESTED NFPA RATING	
BLUE	CLASS AA
5182 gpm @ 20 psi (138 kPa)	

ONTARIO CLEAN WATER AGENCY
AGENCE ONTARIENNE DES EAUX

Prepared By: The Ontario Clean Water Agency
Prepared For: SCS Consulting
Residual Hyd Andrew Cruickshank
Flow Hyd(s) Seth Wiggins, Daniel Johnston

Date: 24-Mar-22 Time: 12:14 PM

HYDRANT DESCRIPTION

Hydrant ID:	3203_3525	Side of Street:	North	Make:	Canada Valve	Open Dir:	Left
Address:	9541 Weston Road			Model:	Century	Latitude:	
Location:	Vaughan ON			Year:	2007	Longitude:	

GENERAL INSPECTION

OK - Good Condition *FR - Future Repair Required* *N/A - Not Applicable* *CF - Component Failure*

Upper Section	OK	FR	N/A	CF
Bonnet	☐	☐	☑	☐
Operating Nut	☐	☐	☑	☐
Gaskets / Bolts	☐	☐	☑	☐
O-Ring(s)	☐	☐	☑	☐

Mid Section	OK	FR	N/A	CF
Port Height	☐	☐	☑	☐
Caps / Nozzles	☐	☐	☑	☐
Chains	☐	☐	☑	☐
Traffic Flange	☐	☐	☑	☐

General	OK	FR	N/A	CF
Accessibility	☐	☐	☑	☐
Position / Height	☐	☐	☑	☐
Paint Cond	☐	☐	☑	☐
Drain Ports	☐	☐	☑	☐

Hydrostatic Leak Testing		
Hydrant Closed	Above Grade Leak	N/A
	Subsurface Leak	N/A
Hydrant Open	Above Grade Leak	N/A
	Subsurface Leak	N/A

Maintenance	
Lubricate Operating Nut	N/A
Lubricate & Clean Nozzle Threads	N/A
Lubricate & Clean Cap Threads	N/A
Water Removed (if non-draining)	N/A

Auxiliary / Secondary Valve	
Located / Accessible	N/A
Operated/Exercised	N/A
Number of Turns	N/A
Open Direction	

Comments:

Auxiliary Valve Location:

FLUSHING

*If hydrants are being flow tested, inspections and flushing are completed prior to testing

Hydrant Operated	Clear Flow Obtained	Cl2 Residual	Time Flushed	Flow	Total Flow	Dechlorinated
Yes - Easily Operated	Yes	N/A	5 minutes	1294 gal	6472 gal	Yes

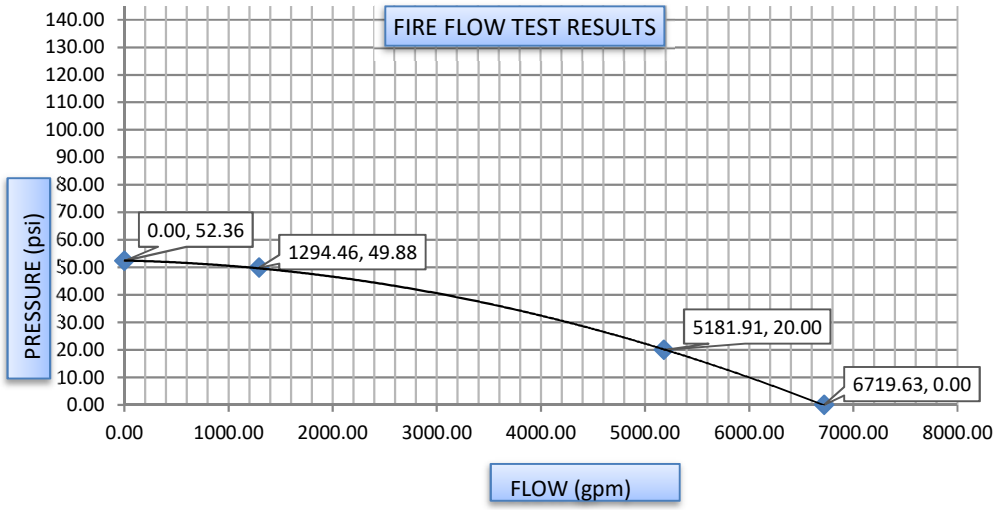
Comments: **STATIC AFTER FLOW TEST WAS PERFORMED 51.12 PSI**

FLOW TESTING

*Flow testing results may be from previous year(s). Note date & time

Date: 24-Mar-22 Time: 12:14 PM

Flow Hydrant								Test Hydrant		
ID	Flow Device Used	Size	Coefficient	Time Flushed	Flow	Total Flow	Pitot	ID	Static	Residual
69	Pollard Diffuser	2.5"	0.832	5.0 minutes	694 gal	3468 gal	20 psi	3203_3525	52.36	49.88
69	Pollard Diffuser	2.5"	0.832	5.0 minutes	601 gal	3004 gal	15 psi			



Calculated Results	
Calculated Flow @ 20 psi	5182 gpm
Calculated Flow @ 0 psi	6720 gpm
Pressure Drop	4.74%

Comments: