FUNCTIONAL SERVICING REPORT

PROPOSED RESIDENTIAL DEVELOPMENT AT 7085 GOREWAY DRIVE

PRESTON HOMES
REDWOOD PROPERTIES INC.

CITY OF MISSISSAUGA
Project: 2020-4866 **July 2023**

Revision	Description	Prepared		Checked	
		By:	Date:	By:	Date:
3.	Updated as per City Comments	N. Akbarzadeh	July 2023	H. Milukow	July 2023
2.	Updated as per City Comments	I. Chandan H. Milukow	March 2023	K. Shahbikian	March 2023
1.	Revised as per updated site plan and City Comment	G. Volpe	May 2022	K. Shahbikian	May 2022
0.	Original Report	D. Tabuas	May 2020	H. Sarkissian	May 2020

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CONTENTS

Figures

Tables

Appendices

1.0 INTRODUCTION

1.1 Objective

This Functional Servicing Report is provided in support of the proposed residential development located at 7085 Goreway Drive in the City of Mississauga and prepared at the request of Preston Homes, in association with Redwood Properties Inc. The property is legally defined as Part of Lot 11, Concession 8, east of Hurontario Street, City of Mississauga, Regional Municipality of Peel.

The property is 0.99ha and is bound on the north by an existing fire station and residential properties, on the east at south by Mimico Creek, and on the west by Goreway Drive, as shown in **Figure 1**.

This report evaluates the existing and proposed water supply, sanitary, and stormwater management services within and surrounding the subject property, thereby demonstrating the viability of the proposed development, and guiding its detailed design.

1.2 Existing Conditions/Site Constraints

Presently there is an existing commercial property adjacent to Goreway Drive with an associated parking area in the rear. Available topography indicates a variance in elevation of about 2m. The peak elevation is just above 166.0m at the northwest portion of the site, in the parking lot. The lowest point has an elevation of just under 164.0m at the southeast portion of the site adjacent to the Creek. This suggests that the site drains to the southeast.

The surrounding properties are well developed with existing commercial along the eastern side of Goreway Drive, and existing single detached homes in the adjacent subdivision north of the site. Utility services exist off of Goreway Drive.

A Flood Hazard Assessment was conducted by Greck and Associates Limited (Greck), dated August 2019, which determined that there will be no negative impact to the flood hazard due to the proposed development. Since their original assessment, their findings have been updated as of May 2022. The results of their updated assessment have been considered in this report.

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REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

LEGEND

SUBJECT AREA

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FIGURE 1-1
LOCATION PLAN

2019-4866	JULY 2023	SCALE: N.T.S.

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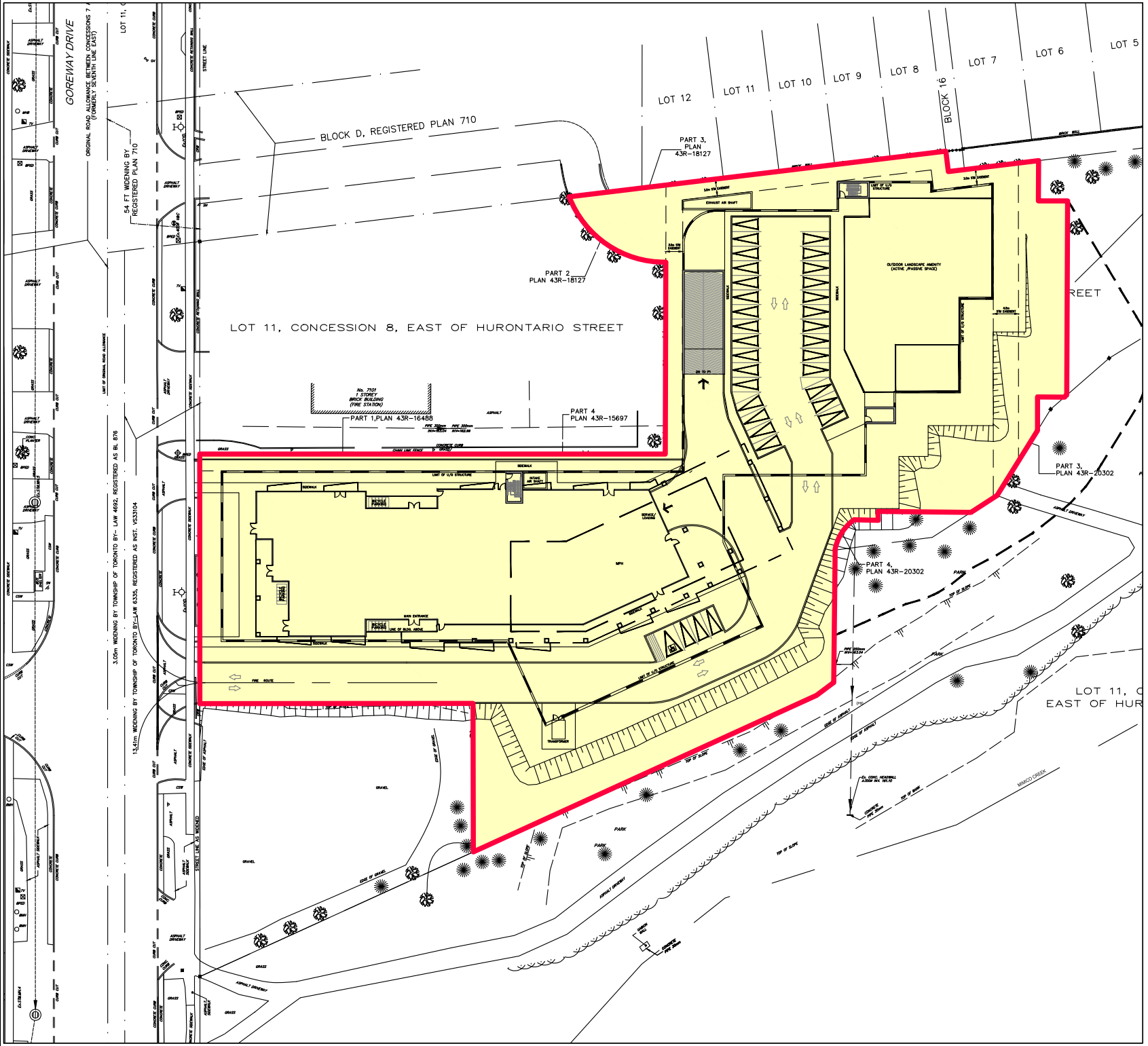

REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

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FIGURE 1-2
DEVELOPMENT PLAN

2019-4866	JULY 2023	SCALE: N.T.S.

1.3 Proposed Development Plan and Population

The subject site has an area of 0.99ha and is proposed to consist of a high-rise condominium; a 14-story tower, sharing 3 levels of underground parking, space for long-term and short-term bicycle parking, and Indoor and outdoor amenities. The site plan and associated site stats, prepared by IBI Group, have been included in **Appendix A** for reference.

The Region of Peel guidelines for sanitary sewer and water supply design recommends a population density of greater than 475 persons/hectare for high-density residential land use or 2.7 people per residential unit. Based on this criteria, the subject site's design population is **618 persons** (based on the more conservative 2.7ppu) as shown in **Table 1.1.**

Table 1.1: Estimated Population Summary

Land Use	Criteria	Qty	Population
1-bedroom	2.7 p.p.u.	100 units	270
2-bedroom	2.7 p.p.u.	81 units	219
3-bedroom	2.7 p.p.u.	47 units	127
Total Highrise based on Units	-	-	**616**
Residential Highrise based on Area	475 person/ha	0.953 ha	453
Commercial based on Area	50 person/ha	0.037 ha	**2**
Design Total	-	-	**618**

Based on the proposed site plan, the high-rise portion of the development encompasses approximately 0.99 ha which includes the existing commercial building area as well as the proposed underground ramp.

2.0 WATER SUPPLY

2.1 Existing Water Supply Services

The subject property is located within the South Peel Water Supply System Pressure Zone 4. Zone 4 is serviced by the Hanlan Reservoir and Pumping Station. Based on information received from the Region of Peel, the following watermains exist in the vicinity of the site:

- a 400mm diameter concrete watermain along the east side of Goreway Drive;
- a 150mm diameter PVC watermain along the west side of Goreway Drive;

There is an existing hydrant adjacent to the subject site. The existing water supply infrastructure can be seen schematically in **Figure 2**.

2.2 Design Criteria

The proposed water supply scheme will be designed in accordance with the Region of Peel design criteria for water systems. The following summarizes typical residential-use design criteria.

- The system shall be designed to provide sufficient flow and pressure to meet the greater of the Maximum Daily Demand Plus Fire Flow or the Maximum Hourly Demand;
- Average Daily Demand of 0.280 m^3/capita/day for residential areas;
- Maximum Daily Demand and Peak Hourly Demand factors shall be 2.0 and 3.0, respectively;
- Minimum watermain size of 300mm for residential areas;
- Operating pressure requirements are noted as follows:

Description	Pressure
Minimum Pressure	275 kPa (40 psi)
Maximum Pressure	690 kPa (100 psi)

- The dead ends shall be minimized by looping all watermains.
- Fire Flows in accordance with Water Supply for Public Fire Protection Survey;

2.3 Proposed Water Supply

One 200mm fire connection and one 150mm domestic water service connection are proposed to service the subject site. The proposed tower height of 47.5m (refer to architectural building statistics) is below the OBC threshold of 84m for tall buildings. As such, the proposed single fire connection is sufficient for fire protection requirements of the proposed building. It is proposed that the subject site be serviced via connection to the existing 400mm watermain along the east side of Goreway Drive. A preliminary servicing scheme is illustrated in **Figure 2-1**.

Based on the Region of Peel's design criteria for water supply, the population of the site is 616 persons (as shown in **Table 1.1**: Estimated Population Summary). **Table 2-1** summarizes the estimated potable water demand.

Table 2.1: Summary of Estimated Potable Water Demand

Land Use	Population	Average Daily Demand (L/s)[1]	Maximum Daily Demand (L/s)[2]	Peak Hour Demand (L/s)[3]
Residential (High-Rise)	616	2	3.99	5.99
Commercial	2	0.01	0.01	0.02

1. Based on 0.280 m^3/capita/day for residential area and 0.3 m^3/capita/day for commercial area
2. Based on a Max Day Factor of 2.0
3. Based on a Peak Hour Factor of 3.0

The fire flow demand for the high-rise building was calculated assuming the building will be fire-resistive construction, and that the vertical openings and exterior vertical communications are properly protected (one-hour rating). A maximum fire flow of 5,000L/min, or 83L/s, has been calculated using FUS. Supporting calculations can be found in **Appendix B** for the high-rise.

2.3.1 HYDRANT TESTING

A hydrant test was performed adjacent to the subject site along 7085Goreway Drive dated June 2nd, 2022. The results of this test found that the existing 400mmØ watermain along 7085Goreway Drive has a static pressure of 92psi. Based on the results of the hydrant test analysis it is anticipated that at the minimum operating pressure of 40 psi (276kPa) a maximum flow of 432L/s is available. Since this flow is greater than the required flow listed in **Table 4-1**, it is anticipated that no servicing constraints will be present in the existing water supply system.

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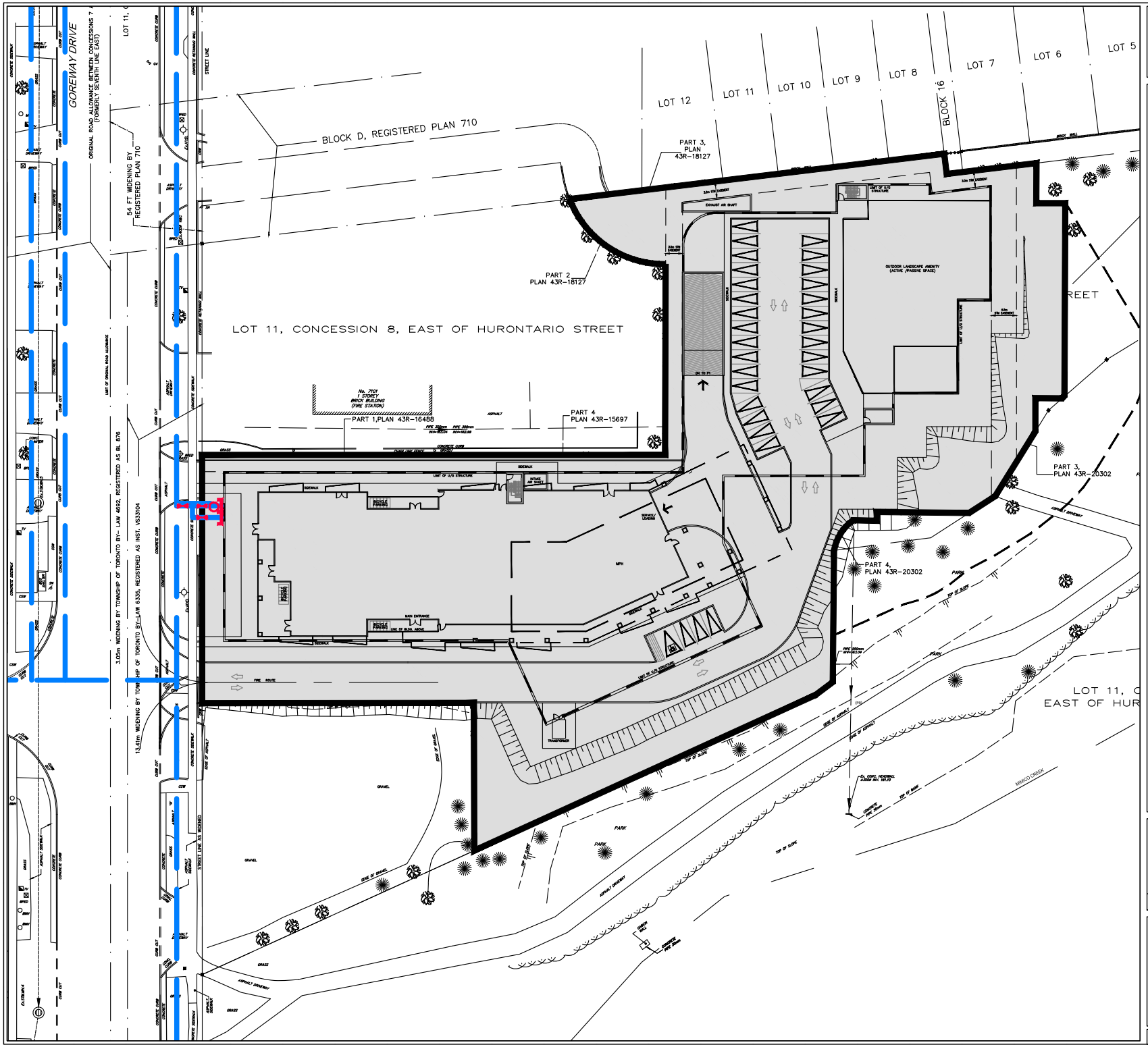

REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

LEGEND

- SUBJECT AREA
- PROPOSED WATERMAIN
- EXISTING WATERMAIN

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FIGURE 2-1
WATER SUPPLY SERVICING PLAN

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3.0 SANITARY SERVICING

3.1 Existing Sanitary Infrastructure

Based on information received from the Region of Peel, there is an existing 250mmØ sanitary sewer on the west side of Goreway Drive, which drains west on Dorcas Street and then south on Minotola Avenue. As the subject site is 0.99 ha, based on the Region's 50 persons per hectare population equivalency, we can expect that the site's existing design population to be approximately 50 persons.

3.2 Design Criteria

The proposed sanitary servicing of the subject site will be designed in accordance with the Region of Peel's "Public Works Design, Specifications and Procedures Manual". These criteria, where applicable to the proposed development, are summarized below.

- The design flow is equal to the Average Dry Weather Flow multiplied by the Average Peak Sanitary Flow Factor, plus the Infiltration Allowance;
- The Average Dry Weather Flow is based on 302.8 L/capita/day;
- If the population is less than 1000 persons, the domestic sewage flow shall be 13L/s plus the infiltration allowance;
- For residential areas, the peak sanitary flow factor is based on the Harmon formula ($M = 1 + 14/(4 + P^{0.5})$, where P is population in thousands;
- Except under unusual circumstances, infiltration allowance shall be determined at 0.2×10^{-3} $m^3/s/ha$ for all types of land use;
- Determination of pipe sizes and capacities to be based on Region of Peel standard drawing SD-2-9-3 or use Manning's Formula;

3.3 Proposed Sanitary Servicing

The subject development is proposed to be serviced via connection to the existing 250mm sewer along Goreway Drive (**Figure 2**).

A preliminary assessment of the anticipated design flow rates has been conducted in accordance with Region of Peel design criteria. With an estimated population of 618 persons, the expected design flow is **13.20 L/s** as according to the Region of Peel standard drawing 2-9-2, the domestic

sewage flow for populations less than 1000 persons, shall be 0.013m^3/s plus the infiltration allowance. **Table 3.1** summarizes the estimated sanitary flow demands, and supporting calculations can be found in **Appendix C**.

Table 3.1: Summary of Estimated Sanitary Flows

Land Use	Area (ha)	Expected Population (1)	Average Sewage Flow (2) (L/s)	Infiltration Inflow (3) (L/s)	Estimated Total Flow (L/s)
Residential (High-Rise)	0.953	616	2.2	0.19	8.89
Commercial	0.037	2	0.01	0.01	0.04
Total	**0.99**	**618**	**2.21**	**0.20**	**13.20**

(1) From Table 1.1
(2) According to the Region of Peel STD.DWG.2-9-2
(3) Infiltration rate of 0.2 L/s/ha (Region of Peel Design Criteria)

Based on the information received from the Region, the existing sanitary sewer estimated a population of 156 for 2.8 ha (6.9 acres) for an area which appears to only include the west side of Goreway Drive. As such, the downstream sanitary calculations which are included in **Appendix C** have added the east side of Goreway Drive, and analyzed the downstream sewers down to Minotola Avenue. The design sheets indicate sufficient capacity to convey the increased flows for the redevelopment. It should be noted that the sanitary design sheet provided by the Region of Peel (for Pastoria Holdings) included in **Appendix C** denotes mention of a 30" dia. trunk sewer, the location of which has not been provided based on the records obtained from the Region.

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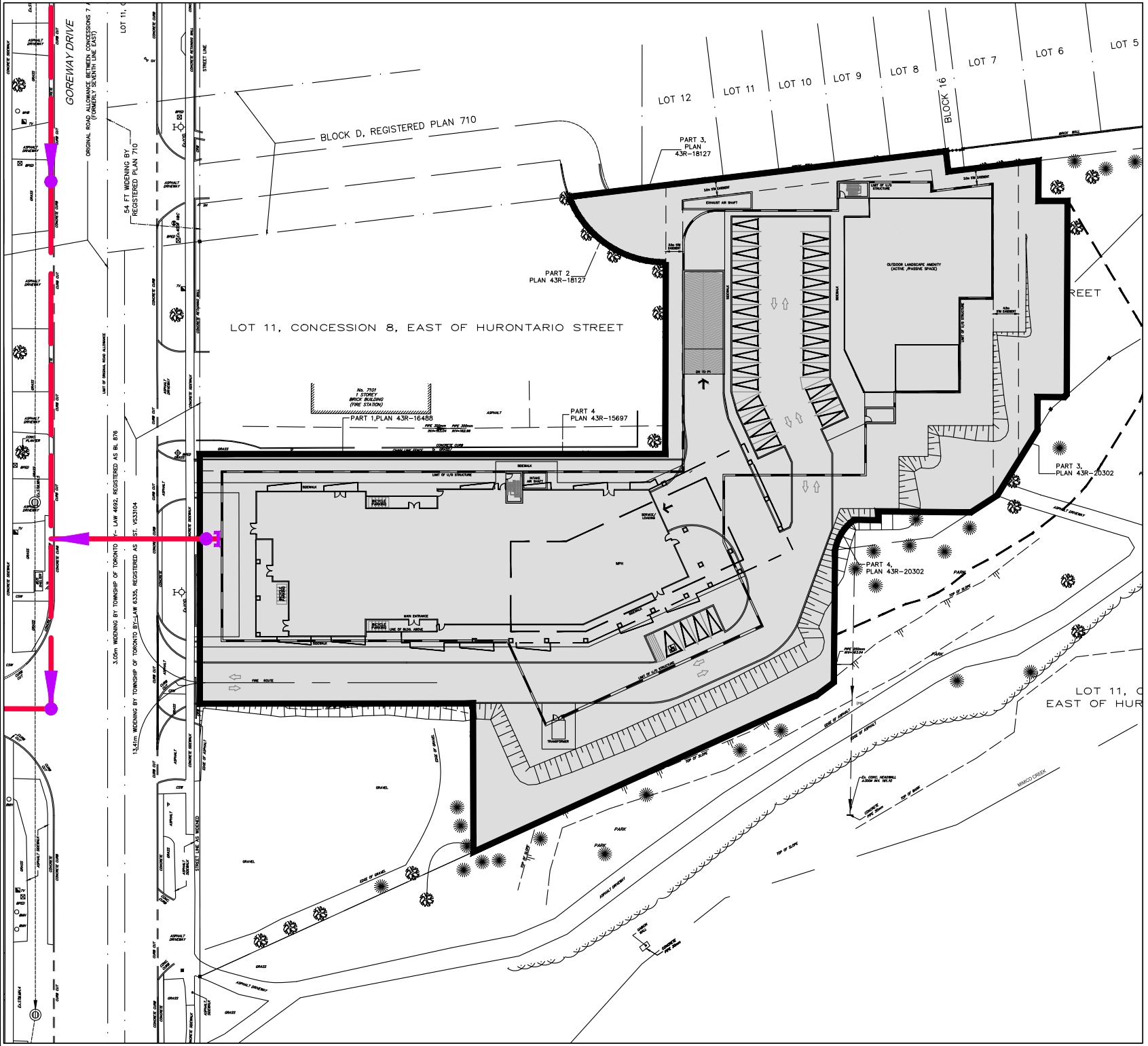

REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

LEGEND

- SUBJECT AREA
- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER

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FIGURE 3-1
SANITARY SERVICING PLAN

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4.0 STORM DRAINAGE

4.1 Existing Site Conditions and Servicing

As previously noted, the subject site currently consists of a commercial building and a large parking lot area, that appears to have been constructed during the years of 1980/1985 based on aerial photography per the City of Mississauga's Online Mapping Service. According to information provided by the City and Region, there is an **existing 450/525mmØ storm sewer** located on Goreway Drive, to the west of the subject site. Site investigations and the topographic survey indicate that the exiting site's flows are captured via various on-site catchbasins and discharge to the East Branch of Mimico Creek, located immediately south of the subject site. This drainage scheme will be maintained in the post-development condition, as the subject site is proposed to drain into Mimico Creek, discharging via a proposed headwall.

The subject site has an area of **0.987 ha**. In order to establish the site's allowable release rate to the East Branch of Mimico Creek, only the area draining directly east to the creek was considered. Removing the small (**0.017ha**) piece at the existing site's northwest corner which is currently draining west to the adjacent 7125 Goreway retail parcel gives a net area of **0.970 ha** drains east in existing conditions.

Furthermore, the existing site is partially located within the regulatory flood line per the information provided by Greck & Associates Ltd., and as part of the site's development it is proposed to re-grade within the public lands to the east of the site, and predominantly within the property in order to allow for the redevelopment to proceed. The proposed regulatory floodplain has been provided by Greck & Associates Ltd. (refer to **Appendix D**) and shown on the preliminary engineering plans.

As part of the aforementioned re-grading, approximately **0.205ha** of the subject lands will drain overland directly towards Mimico Creek. As this area will drain uncontrolled, and will be covered in pervious landscape (i.e., C = 0.25) this proposed land cover is considered to be an improvement over the current site land use conditions, which are predominantly paved parking and roof area. By providing entirely pervious cover in the proposed condition for the uncontrolled area, this ensures that post-development flows remain less than the existing condition for all storm events over this area.

Considering the above areas, it is proposed to establish the target release rates of the site based on the remaining **0.765 ha** (i.e., 0.987ha – 0.017ha – 0.205ha = 0.765ha).

It should be noted that site investigations have revealed that the stormwater flows from 7101 Goreway Drive (the adjacent Fire Station) are also discharged to Mimico Creek by a sewer which currently cuts through the middle of the subject site via an easement. As part of this redevelopment proposal, and as indicated on the site servicing plan SS-1, it is proposed to redirect the sewer around the proposed development via a new easement to Mimico Creek. The re-routed storm sewers leaving the subject site are proposed at 161.64m which is below the regulatory floodplain elevation. As the existing storm sewer from 7101 Goreway is located under the floodplain elevation, it is not feasible to raise the sewer above the regulatory (or 100-year) flood plain level and will therefore mimics the existing conditions.

As indicated on the Site Servicing drawing (SS-1) included in **Appendix E**, the subject site's flows will be discharging via a proposed storm sewer towards Mimico Creek. The proposed regulatory and 100-yr flood lines provided by Greck & Associates Ltd. are at elevations of **165.06m** and **163.23m**, respectively. The storm system leaving the subject site is proposed at 163.55m, which is 0.32m higher than the 100-year flood line, to mitigate flooding risks. In order to further mitigate the effects of the regulatory flood line, backwater preventers will be considered during the detailed design stage.

4.2 Design Criteria

The stormwater flow calculations are based on the following the City of Mississauga design criteria:

- As the storm flows will discharge to the Mimico Creek, Post to Pre-development controls for all storm events are to be provided;
- The first 5mm of runoff shall be retained on-site and managed by way of infiltration, evapotranspiration or re-use;
- Storm sewers shall be designed using Rational Formula; Q = 0.0028 CIA, where Q is the flow rate in m^3/s, C is the runoff coefficient (dimensionless), I is rainfall intensity in mm/hr and A is area in ha;
- Storm sewer design should be based on City of Mississauga Rainfall Intensity Curves and a minimum time of concentration of 15 min. $I = A/ (T + B)^C$, where I is rainfall intensity in

mm/hr, T is time of Concentration in hours, A = 610, B = 4.6, C = 0.78 for the 2-year storm event;

- Runoff Coefficient:
  - Impervious surfaces 0.90
  - Sodded/Pervious/surfaces 0.25
  - Runoff Coefficients are to be adjusted per the City's Design Criteria, to account for increase in runoff due to saturation of the catchment surface that would occur for larger, less frequent storms.

4.3 Stormwater Management Plan

As noted above, it is proposed to have the subject site's stormwater discharge mimic the pre-development scenario of discharging to Mimico Creek. To ensure post-development discharge does not exceed the pre-development site discharge for each storm return period, on-site attenuation is proposed. On-site detention, required to meet the target release rates, shown in **Table 4.1**, will be provided via an underground detention storage tank provided in the underground parking levels. An orifice control structure will be provided at the downstream end of the tank, and will be appropriately sized to restrict the site's release to the aforementioned peak flow rates.

The proposed storm drainage area and stormwater management features are shown in **Figure 4-3**, and supporting calculations are included in **Appendix D**. The new on-site storm sewers, which will be located within the parking garage, will be designed by the site mechanical engineer to meet the standards of the Ontario Building Code.

4.4 Allowable Release Rate

All storm flows will be directed to the proposed stormwater management tank and controlled to an allowable release rate which will conform to the requirements noted above. The pre-development and post-development hydrologic conditions for the site were established using the City's IDF data, a recommended entry time of 15 minutes, and weighted runoff coefficients.

As mentioned in Section 4.1, the site's-controlled release rates will be established based on the existing site area draining to Mimico Creek. In the estimation of the allowable release rates, a weighted pre-development runoff coefficient was calculated reflecting the imperviousness of the

existing site area, which consists of predominantly building roof and paved parking area (i.e., C = 0.90). As per the City's design criteria, the pre-development runoff coefficient shall be limited to a maximum 0.50, and therefore governs in this case. Using the rational method, the peak release rate was calculated for the subject site. The calculations have been included in **Appendix D**, and results summarized below.

Table 4.1: Pre-Development Conditions

Return Period (years)	Runoff Coefficient (1)	Intensity (mm/hr)	Peak Flow (L/s)
2	0.50	59.89	63.7
5	0.50	80.51	85.6
10	0.50	99.17	105.5
25	0.55	113.89	133.2
50	0.60	127.13	162.2
100	0.63	140.69	187.0

(1) RC adjusted per City of Mississauga design criteria.

4.5 Water Quantity Control

Stormwater management for the proposed development will consist of on-site detention to attenuate the site's post-development flows to levels that are less than or equal to the maximum allowable release rates by utilizing detention storage tanks equipped with orifice control structures upstream of the quality control devices to control flows.

The Modified Rational Method was used to calculate the required storage volume for each storm event based upon the allowable release rate during the 2-year through 100-year storm events. The Maximum Allowable Release Rate from the site is noted in the table above. The determination of the site's required storage was calculated using an assumed runoff coefficient of **0.85** for post-development conditions, considering that the proposed site will be made up of predominately impervious roof, drive isles, and landscape, with local pervious landscape and planters. The appropriate design runoff coefficient will be confirmed at the detailed design stage. Calculation of the site storage requirements are included in **Appendix D**, and are summarized in **Table 4.2**.

In order to simplify the proposed stormwater management tank design, it is proposed to control all storm events to the existing 2-year peak flow estimated and provided in **Table 4.2**.

Furthermore, it is expected that approximately **0.023ha** of area fronting Goreway Drive will drain uncontrolled from the site due to grading constraints. Based on the site runoff coefficient of 0.85, a 100-year peak flow of 3.3L/s is expected to be generated by this uncontrolled area. As a result, the effective allowable release rate from the tank is expected to be 60.40 L/s (i.e., 63.70 L/s – 3.30 L/s = 60.40 L/s).

Based on the uncontrolled area to Goreway Drive, the remaining **0.742ha** (i.e., 0.765ha – 0.023ha = 0.742ha) will be controlled via an underground detention storage tank. The expected storage for the tank design is summarized in the table below. Based on the results below, the site will be provided a minimum **239m^3** of detention storage.

Table 4.2: Site Storage Requirements

Control Area (ha)	Design Runoff Coefficient (1)	Time of Concentration (min.)	Orifice Control Structure	Uncontrolled Site Release Rate (L/s)	Tank Release Rate (L/s)	Required Storage (m^3)	Total Release from Site (L/s)
0.742	1.00	10	172mmØ Plate	3.30	60.40 L/s	228	63.70

(1) RC adjusted per City requirements, with max RC of 1.0

Furthermore, it should be noted that as a result of the downstream storm sewer's elevations, a pumped solution is required in order to facilitate drainage from the site. In the proposed SWM scheme, storm water will be pumped from the proposed detention storage tank up to a stabilization chamber at a rate equal to the prescribed tank release rate in **Table 4.2**. Flows which enter the stabilization chamber via a pump will be discharged out of the site via gravity through the proposed orifice structure, listed in **Table 4.2.**

In order to ensure the release rate from the tank orifice does not exceed the allowable release rate, an overflow weir will be provided within the stabilization chamber to regulate the water elevation over the proposed orifice structure such that it does not exceed the allowable release rate. In this way, any excess flows, which may occur in the case which the pump releases at a rate greater than the allowable controlled release rate, can be safely discharged over the overflow weir and back into the detention storage tank. Details of the proposed tank and requirements for the proposed pump design will be finalized at the detailed design stage. It is lastly noted that any proposed pumping system shall be designed by the site mechanical engineer.

4.6 Water Balance & Quality Control

The City's T&W Development Requirements a 5mm runoff reduction is required for on-site waterbalance and retention. As such, **39 m^3** (0.765 ha x 5mm x 10 = 39 m^3) based on the site's impervious area is required to be retained on-site and managed by way of infiltration, evapotranspiration or re-use. Clean water will be re-used on-site by non-potable means such irrigation and/or in the mechanical cooling system for the development. Specific re-uses, as well as the detailed re-use volume, will be confirmed at the detailed design stage.

On-site quality controls to provide 'Enhanced' (Level 1) protection is proposed for the subject site to meet site quality requirements. In order to achieve this, a treatment unit will be sized to provide the long-term average removal of 80% of Total Suspended Solids (TSS) on an annual loading basis, for 90% of the average annual site runoff. A preliminary Jellyfish Unit Sizing by Imbrium has been sized to provide an enhanced level of treatment. The sizing has been provided in **Appendix D** for reference. The final unit to be used on-site will be confirmed at the detailed design stage.

4.7 Groundwater and Foundation Drainage

Hydrogeological calculations and analysis for the subject site was undertaken by Grounded Engineering and summarized in their Geohydrology Assessment, to assess the potential effects of groundwater on the proposed development. They have noted that preliminary estimates for the long-term dewatering total 130,000 l/day (1.5 l/sec). As such, the foundation drainage / groundwater may discharge to the storm outlet for the site, which is directed to Mimico Creek.

As per the hydro-geotechnical reports the unfiltered groundwater sample exceeds the limits for storm sewer discharge, however, the groundwater sample meets the limits for sanitary and combined sewer discharge. It is understood that if the groundwater will be discharged to the City's storm sewer, it must meet the City's satisfaction for both quantity (combined stormwater and groundwater releases not to exceed the allowable release rate) and quality requirements (per the City Storm Sewer By-Law). The groundwater flows will be reviewed in greater detail during the detailed design of the development, but given the approximate flows noted by the hydrogeologist, quantity-related issues are not expected on this site. Refer to the hydro-geotechnical reports provided in **Appendix A** for details. It is currently proposed to discharge the groundwater flows to Mimico creek, where quality control is provided by the proposed Jellyfish unit (proven or equivalent).

4.8 External Drainage and Pipe Considerations

As discussed, 7101 Goreway's existing fire station site currently drains through an existing easement to Mimico Creek through the subject site. In order to continue facilitating drainage from the site in post development conditions, design sheets have been provided in **Appendix D** in support of a proposed by-pass sewer to convey the flows through the site separately. It should be noted that in general site drainage from the eastmost property line of the 7101 Goreway Fire Station, directly adjacent to the site, drains east to west. Along the westmost boundary of the site, catchbasins are present at low points which capture and convey flows through the existing storm sewer which passes through the subject site. As a result of this it is expected that no overland drainage is or will be tributary to the subject site in post development conditions. Furthermore the pipe sizing of the proposed by-pass sewer has been designed such that it considers the full capture of all storm events up to and including the 100-year storm event from the 7101 Goreway site will be tributary to the proposed by-pass sewer to Mimico Creek. Therefore, it is expected that the proposed by-pass sewer will have sufficient capacity to convey flows from the Fire Station in post-development conditions.

In addition to the Fire Station, consideration has been made for external drainage from the existing single-detached homes north of the proposed development. As per the existing lot grading plan provided in **Appendix A**, for the site to the north, it was determined that these lots drain from the backyards south easterly towards Mimico Creek. In order to maintain this existing drainage condition, it is proposed to provide a swale running west to east along the site property boundary to convey flows from the external lands to Mimico, as it does in existing conditions. It is noted that these swales are also considered to convey some flows from grassed areas within the subject site to the creek. These areas have been identified in **Figure 4-2**. The design of the proposed swale will be confirmed at the detailed design stage.

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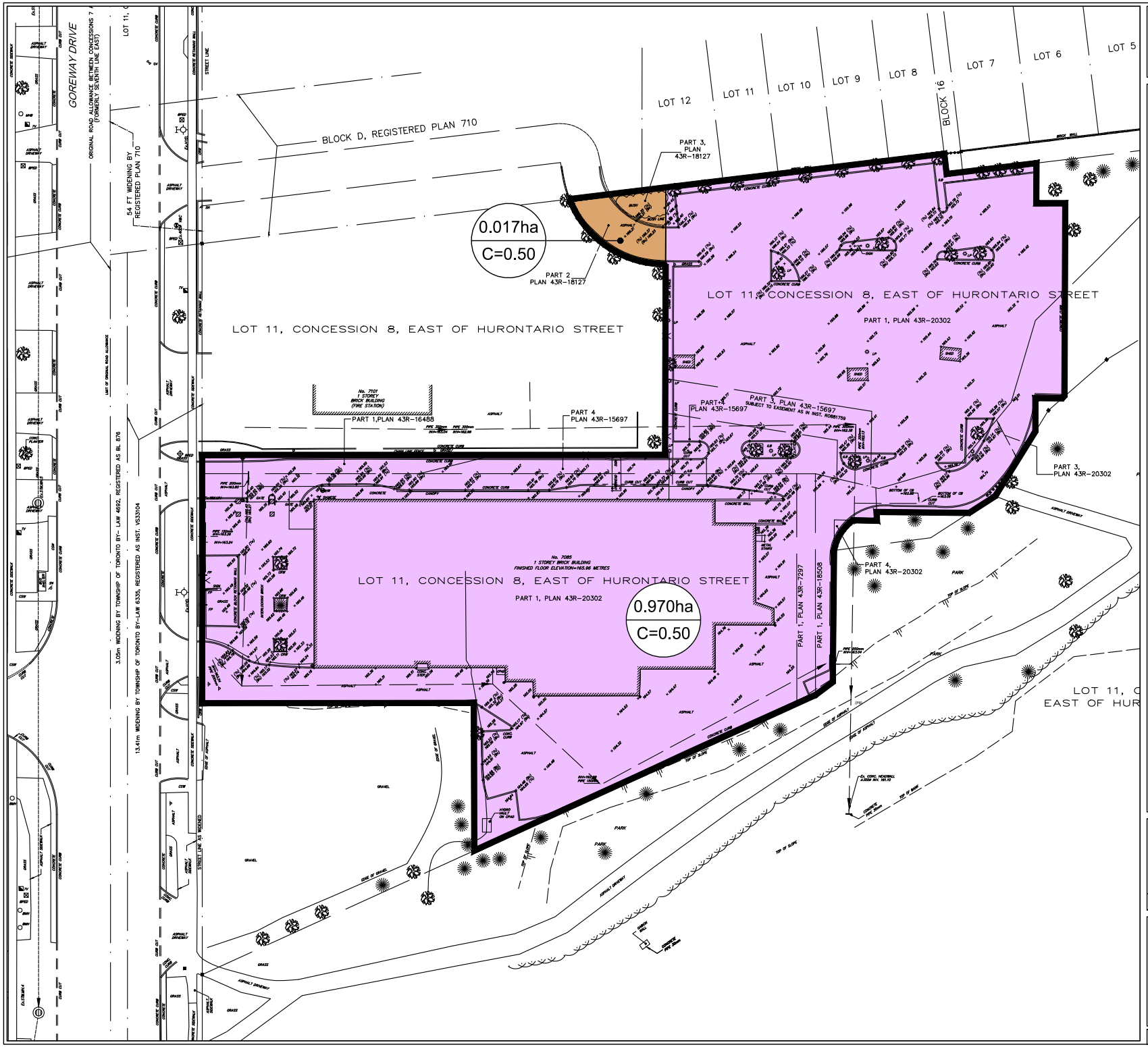

REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

LEGEND

SUBJECT AREA

EXISTING DRAINAGE AREA
(EXISTING DESIGN MAX COEFFICIENT)

WEST DRAINING - UNDEVELOPED

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FIGURE 4-1
EXISTING DRAINAGE PLAN

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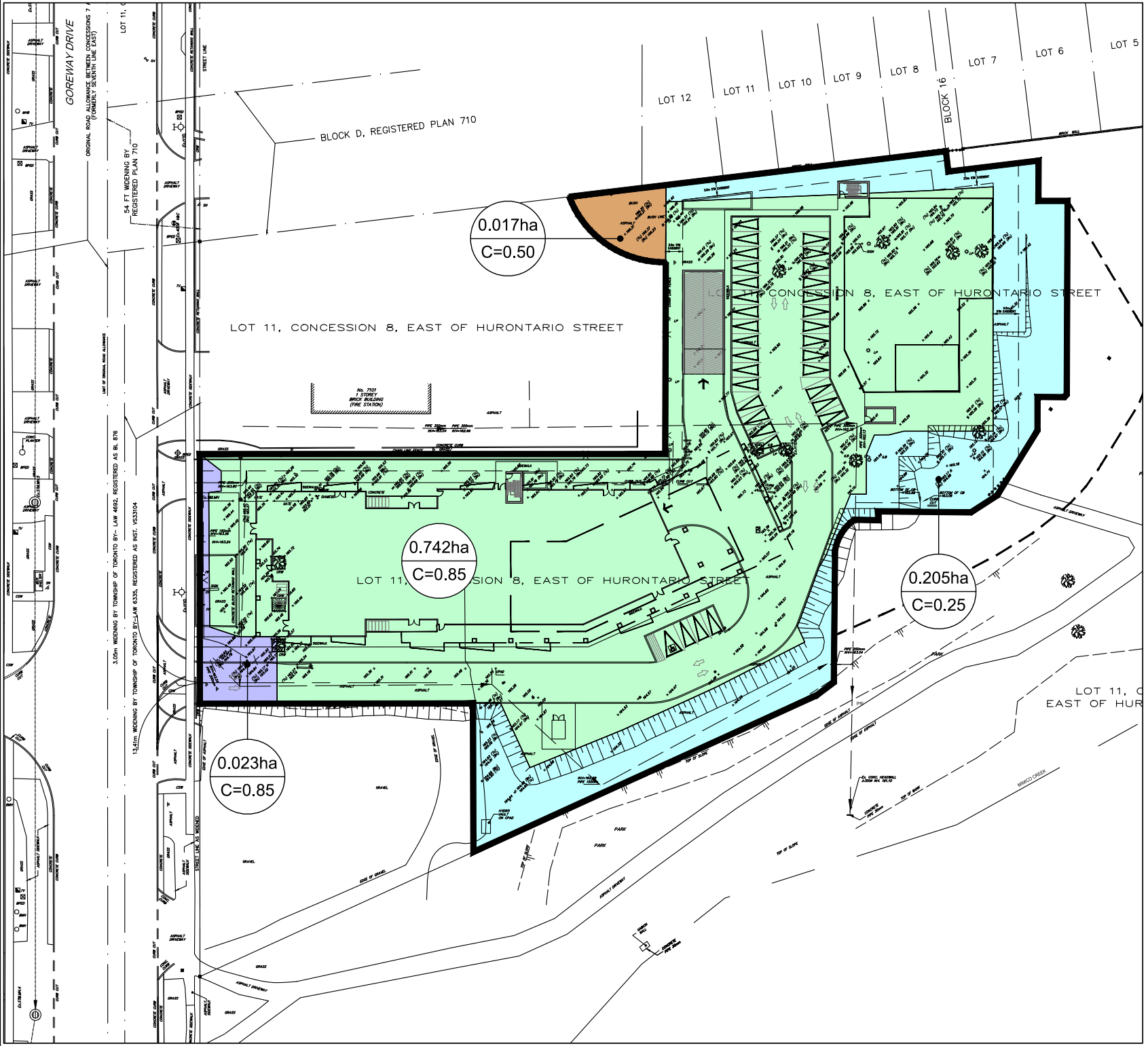


REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

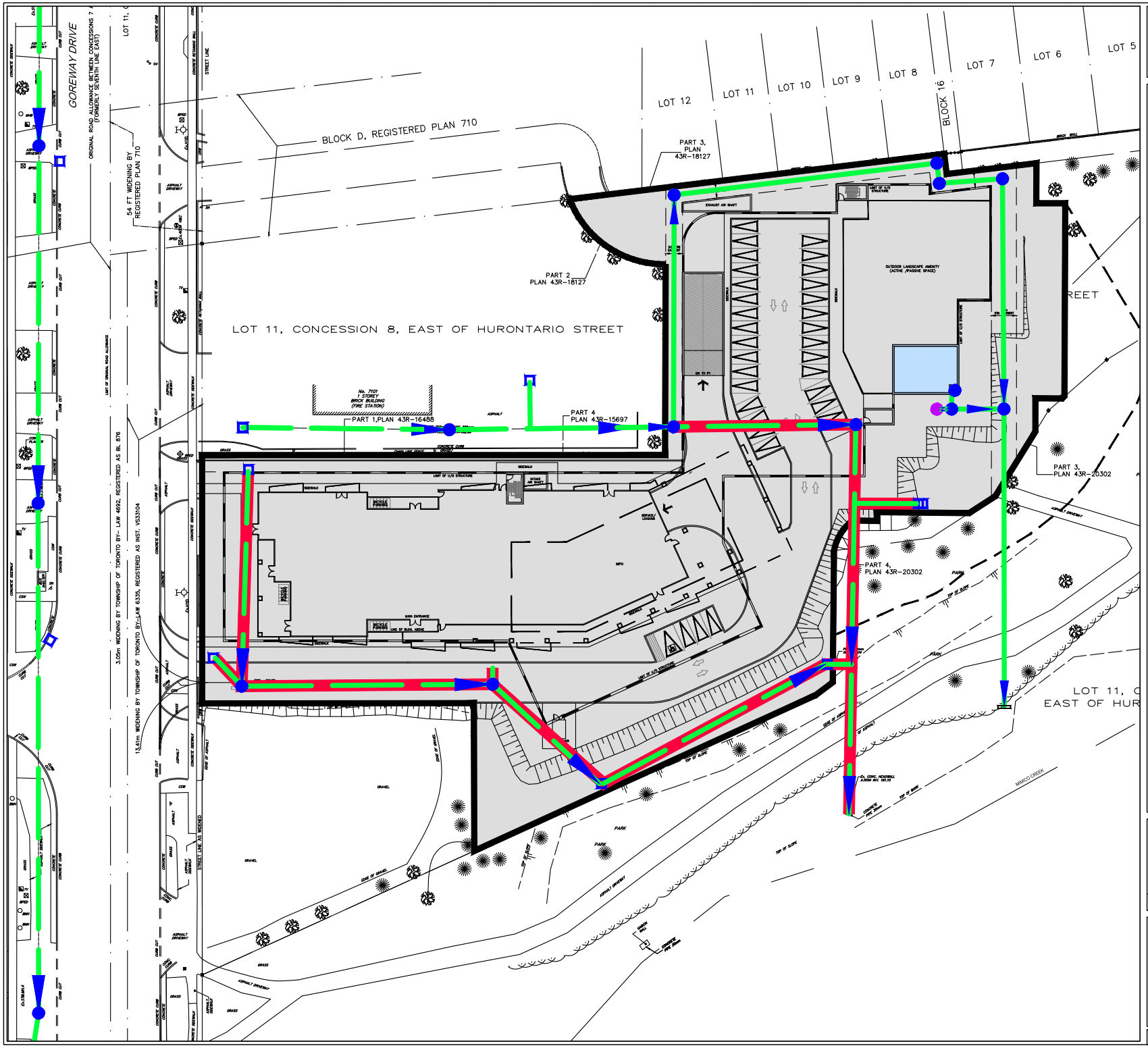
LEGEND

- SUBJECT AREA
- CONTROLLED DRAINAGE AREA
- UNCONTROLLED TO GOREWAY
- UNCONTROLLED TO CREEK
- WEST DRAINING - UNDEVELOPED

SCHAEFFERS
CONSULTING ENGINEERS
6 Ronrose Drive, Concord, Ontario L4K 4R3
Tel: (905) 738-6100 Email: general@schaeffers.com
www.schaeffers.com

FIGURE 4-2
PROPOSED DRAINAGE PLAN

2019-4866	JULY 2023	SCALE: N.T.S.



REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

LEGEND

- SUBJECT AREA
- PROPOSED STORM SEWER
- EXISTING STORM SEWER
- EXISTING STORM SEWER TO BE REMOVED
- PROPOSED SWM TANK
- FILTER UNIT

SCHAEFFERS
CONSULTING ENGINEERS
6 Ronrose Drive, Concord, Ontario L4K 4R3
Tel: (905) 738-6100 Email: general@schaeffers.com
www.schaeffers.com

FIGURE 4-3
STORM SERVICING PLAN

2019-4866	JULY 2023	SCALE: N.T.S.

5.0 SUMMARY

This Functional Servicing Report provides an overview of the proposed servicing plan for the residential development located at 7085 Goreway Drive, within the City of Mississauga. This report demonstrates that adequate stormwater, sanitary, and water supply servicing will be available for the proposed development. In summary, the functional servicing analysis established the following:

Water Supply

- Water supply servicing will be provided from an existing 400 mm diameter watermain located along Goreway Drive.
- A hydrant test was conducted and no servicing constraints are expected.

Sanitary Servicing

- The proposed developments will be serviced by the existing 250mm diameter sanitary sewer located along Goreway Drive.
- No constraints are expected on the downstream sanitary sewers as per the conducted sanitary analysis.

Stormwater Servicing

- Peak flows from the subject property will be controlled via on-site measures which include a storage tank within the underground parking, prior to discharging to Mimico Creek.
- 5mm retention will be provided via re-use, and on-site irrigation.
- Water quality control will be provided using an on-site filtration unit upstream of the site's control outlet.

We trust the above information is suitable for your needs at this time. Should you have any questions or comments, please do not hesitate to contact the undersigned.

Sincerely,

SCHAEFFER & ASSOCIATES LTD.

Nayereh Akbarzadeh, M.Eng, EIT
Water Resources Analyst

Heather Milukow, M.Eng, P.Eng.
MESP/Site Plan Group Manager

Appendix A

Background Information

Project Statistics - 7085 Goreway Drive, Mississauga

2023-07-07

1.0 Site Area

	m2	ft2
Gross Lot Area	9,705	104,464

2.0 Density

F.S.I (Gross Floor Area / Gross Lot Area)	2.3

3.0 Building Area

* Excludes parking and loading areas within podium.

	Tower		Total	
	m2	ft2	m2	ft2
Residential GFA	22,267	239,680	22,267	239,680

	m2	ft2
Commercial GFA	371	3,993

	m2	**ft2**
Total GFA	**22,638**	**243,673**

4.0 Building Height

* maximum height, excluding 6.3m mechanical penthouse.

	Tower	
	m	storeys
Residential	47.50	14

5.0 Unit Mix Summary

Unit Type	Tower		Total	
	No.	%	No.	%
1 Bedroom	100	43.9%	100	44%
2 Bedroom	81	35.5%	81	36%
3 Bedroom	47	20.6%	47	21%
Subtotal	228	100%	228	100%

6.0 Parking - Per New By law

	Parking Rate	Proposed Units/GFA	Total
Res. Condominimum Apartment Tower	1.0 spaces per dwelling units	228	228
Commercial	4.0 paces per 100m2	336m2	16
Visitor Condominium	0.2 spaced per dwelling units	228	46
		Total Parking Spaces Required	290
		Pkg. Provided (3 Levels of UG Parking)	299
		Surplus (Deficit)	9

* Electric Vehicle ready parking spaces : 20% of the total requried parking (290 x 0.2 = 58 stalls)

7.0 Bicycle Parking

Long Term	0.6 spaces per unit	Required and Provided	137 Spaces
Short Term	The greater of 0.05 spaces per unit or 6.0 spaces	Required and Provided	11 Spaces

8.0 Amenity

			m2	ft2
Indoor and Outdoor Amentiy Combined Required		The greater of 5.6m2 per dwelling unit OR 10% of the site area.		
Unit Total : 228	5.6m2		1,277	13,743
Site Area (m2):9870	10%		987	10,624
Indoor Amenity Provided			503	5,414
Progreammed Outdoor Amenity Provided			787	8,471
Total Amenity Provided			1,290	13,885

5

IBI GROUP
7th Floor-55 St. Clair Avenue West
Toronto ON M4V 2Y7 Canada
tel 416 596 1930 fax 416 596 0644
ibigroup.com

KEY PLAN

SUBMISSION

#	Date	By	Comment
5	2023.07.07	DH	RE-ISSUED FOR OPA
4	2022.06.13	DH	RE-ISSUED FOR OPA
3	2020.08.12	DH	ISSUED FOR OPA
2	2020.06.12	DH	ISSUED FOR OPA
1	2019.09.06	DH	ISSUED FOR DARC

True North Project North

SEAL :

ONTARIO ASSOCIATION OF ARCHITECTS DAVID WRIGHT LICENCE 3041

PROJECT :

redwood PROPERTIES

REDWOOD ON GOREWAY

7085 Goreway Drive, Mississauga, Ontario

TITLE : Statistics

DATE : 2022-02-02

SCALE : N.T.S

DRAWN : SL/DV/EK

CHECKED : DH

PROJ. NO. 120212

A-001

Appendix B

Water Supply Calculations

Water Supply Calculation

Project No. 4866

Proposed Residential Development - 7085 Goreway Drive, City of Mississauga

Fire Flow:	**5000** l/min	**83.33** l/s
Water Supply Demand:	**280** l/capita/day	
Water Supply Demand for ICI:	**300** l/capita/day	

Land Use	Type	Units or Area	Pop. Density (persons/unit or ha) †	Population	Average Day Demand (l/s) ‡
Residential	High-Rise Units	228	2.7	616	**2.00**
Commercial		0.04	50	2	**0.01**
	Total			618	**2.00**

Land Use	Type	Average Day Demand (l/s) ‡	Peak Hour Demand Peaking Factor †	Peak Hour Demand (l/s)	Max Day Demand Peaking Factor †	Max Day Demand (l/s)	Max Day Demand + Fire (l/s)
Residential	High-Rise Units	**2.00**	3.0	**5.99**	2.0	**3.99**	**87.33**
Commmercial		**0.01**	3.0	**0.02**	1.4	**0.01**	

† As per Region of Peel Design Guidelines
‡ Based on 280 L/D for Residential and 300 L/D for Commercial per person based on Region of Peel Design Guidelines

Fire Flow - 7085 Goreway Drive: high-rise

A = Type of Construction

Type of Construction:	C	Description
Wood Frame	1.5	(essentially all combustible)
Ordinary	1	(brick/masonry walls, combustible interior)
Non-Combustible	0.8	(unprotected metal structure, masonry/metal walls)
Fire-Resistive	0.6	(fully protected frame, roof, floors)

Construction Coefficient:	**0.6**	

D = Fire Flow (000's)

For Detached 1-2 family dwellings <= 2 storeys (L/min)

Exposure Distance	**Wood Frame**	**Masonry/Brick**
< 3 m	See manual	6000
3 to 10 m	4000	4000
10.1 to 30 m	3000	3000
> 30 m	2000	2000

-OR-

GFA	2,848	square metres
Construction Type	0.6	
Fire Flow	7,044	L/min.

-> Fire Flow	**7,000**	**L/min.**

GFA includes the area of the largest floor (ground floor as delineated from CAD) plus 25% of the 2 above floors (assuming they are the same size as ground floor)

E = Occupancy Factor

Fire Hazard of Contents	**Charge**
Non-Combustible	-25%
Limited Combustible	-15%
Combustible	0%
Free Burning	15%
Rapid Burning	25%

Occupancy Factor	-15%	
Fire Flow	**5,950**	**L/min.**

F = Sprinkler Factor

Sprinkler System	**Charge**
n/a	0%
NFPA 13 System	-30%
Fully Supervised System	-50%

Sprinkler Factor:	-40%	incl 10% Standard Connection Size

G = Exposure Factor

Separation	**Charge**	
0 to 3 m	25%	
3.1 to 10 m	20%	
10.1 to 20 m	15%	north (18m to fire hall) and east (19m to TH)
20.1 to 30 m	10%	
30.1 to 45 m	5%	
Exposed Sides	2	

Exposure Factor	30%	(no more than 75%)

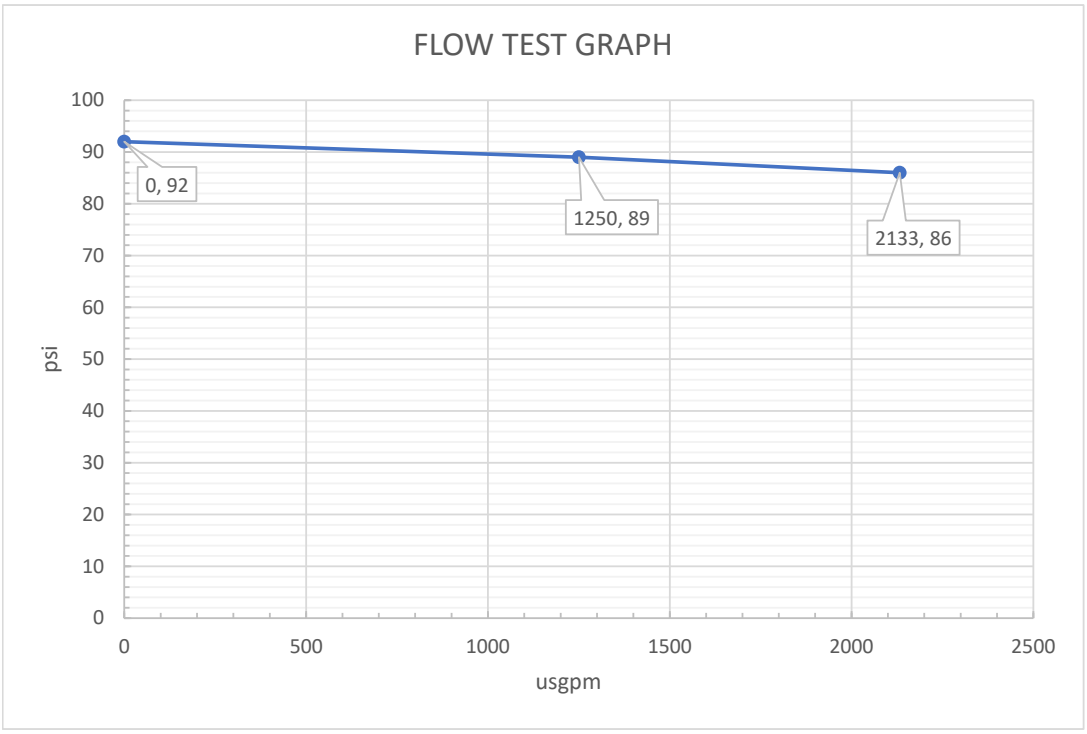
H - Net Fire Flow Required

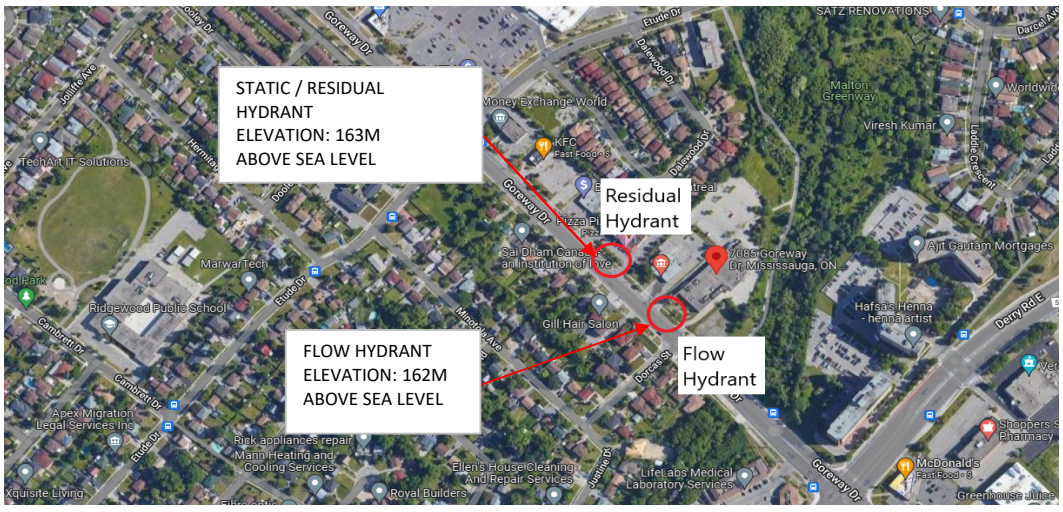
	Charge
F + G Factors	-10%

	5355	L/min.
Fire Flow:	5000	L/min.
	83	L/s

FLOW TEST REPORT

Name of risk:									
Address	7085 Goreway Dr., Mississauga								
Date:	6/2/2022			Time	12pm		Size of Main		16" conc
Static	Pitot. 1 (2.5")	Flow 1	Res. Pres. 1	Pitot 2a (2.5")	Flow 2a	Pitot 2b (2.5")	Flow 2b	Flow 2a+2b	Res. pres. 2
92	55	**1250**	**89**	40	1066	40	1066	**2133**	**86**





Note: Flow Test was performed as per NFPA 291.

Note: Hydrant's elevation is obtained from Google Earth.

7085 GOREWAY DRIVE, MISSISSAUGA

Project No. 4866

Test 1

Flow Test Results of June 02nd 2022

Location: Residual: Goreway Drive

Flow: On Gorway Driveway Northwest of Dorcas Sreet

Test Results			
Flow US. GPM	Residual Pressure psi	Flow L/s	Residual Presure kPa
0	92	0	635
1250	89	79	614
2133	86	135	593

For a total required flow demand of **87.33 L/s** the equivalent residual pressure is

616 **kPa**

89 **psi**

For a residual pressure of **40 psi** or **276 kPa** the equivalent flow is

432 **L/s**

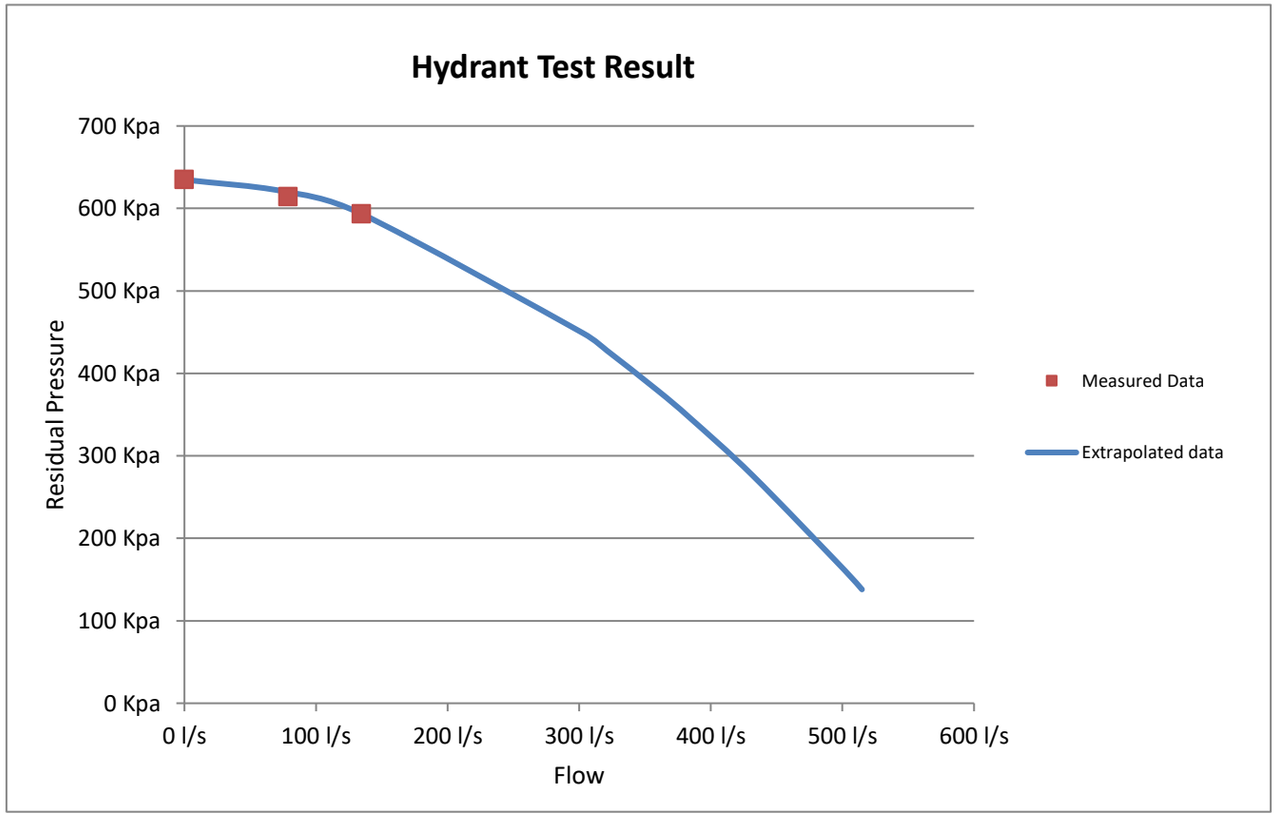
6846 **USGPM**

5700 **IGPM**

1 USG = 3.785 litres

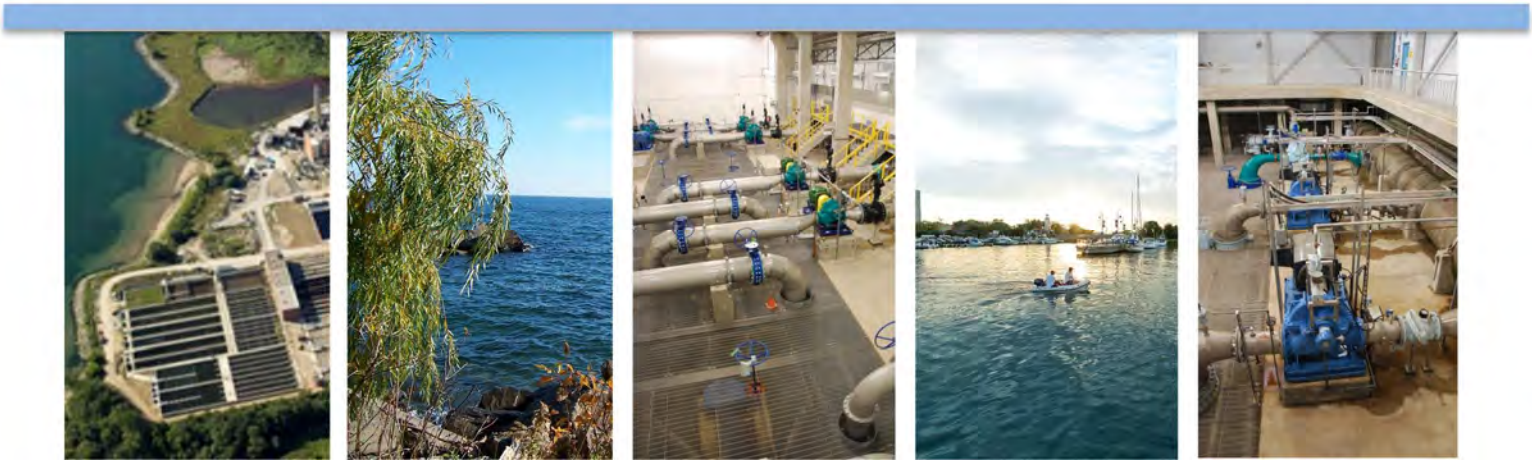
1 IG = 4.546 litres

1 psi = 6.9 kpa



Region of Peel
Working for you

2013 Water and Wastewater
Master Plan for the Lake-Based Systems



Volume III - Water Master Plan

Final Report
P001-0005

March 31, 2014

BluePlan
ENGINEERING CONSULTANTS LIMITED

AECOM

3 Existing Water Transmission System

3.1 Existing Infrastructure

The Region of Peel's lake-based water transmission system services the City of Mississauga, much of the City of Brampton, and parts of the Town of Caledon. The system consists of two Lake Ontario-based water treatment plants (the Lakeview WTP and the Lorne Park WTP), transmission mains, pumping stations, reservoirs and elevated tanks that deliver water to customers through seven pressure zones separated by approximately 30-metre intervals of elevation. The lake-based water transmission system consists of three main trunk systems: west, central and east. The existing transmission facilities are summarized in Table 3.2.

Separate from the water transmission system, the water distribution system conveys treated water from the water transmission facilities to the customers. With the exception of east-west sub-transmission, the water distribution system is not included in the 2013 Master Plan for the lake-based system.

The Region of Peel also maintains four municipal groundwater systems servicing rural communities in the Town of Caledon. These municipal groundwater systems are not included in the 2013 Master Plan for the lake-based system.

Table 3.2 Existing lake-based water transmission facilities

West Trunk System	Central Trunk System	East Trunk System
Snelgrove Elevated Tank (WS6)	Mayfield West Elevated Tank (CS7)	Bolton Elevated Tank (BS6)
West Brampton Reservoir (WS4) and Pumping Station (5LLP, 6HLP)	North Brampton Reservoir (CS5) and Pumping Station (6LLP, 7HLP)	Tullamore Reservoir (ES4) and Pumping Station (5LLP, 6HLP)
Meadowvale North Reservoir (WS3) and Pumping Station (4LLP, 5HLP)	East Brampton Reservoir (CS4) and Pumping Station (5LLP, 6HLP)	Airport Road Reservoir (ES3) and Pumping Station (4LLP, 5HLP, York)
Streetsville Reservoir (WS2) and Pumping Station (3LLP, 4HLP)	Beckett Sproule Reservoir (CS3) and Pumping Station (4LLP, 5HLP)	Beckett Sproule Transfer Pumping Station
Herridge Reservoir (WS1) and Pumping Station (2LLP, 3HLP)	Hanlan Reservoir (CS2) and Pumping Station (3LLP, 4HLP)	
Lorne Park Water Treatment Plant and Pumping Station (1HLP, 2HLP)	Silverthorn Reservoir (CS1) and Pumping Station (2LLP, 3HLP)	
	Lakeview Water Treatment Plant and Pumping Station (1HLP, 2HLP)	

Note: W – West; C – Central; E – East; S – Storage; LLP – Low Lift Pump; HLP – High Lift Pump
 # - pressure zone serviced by the facility

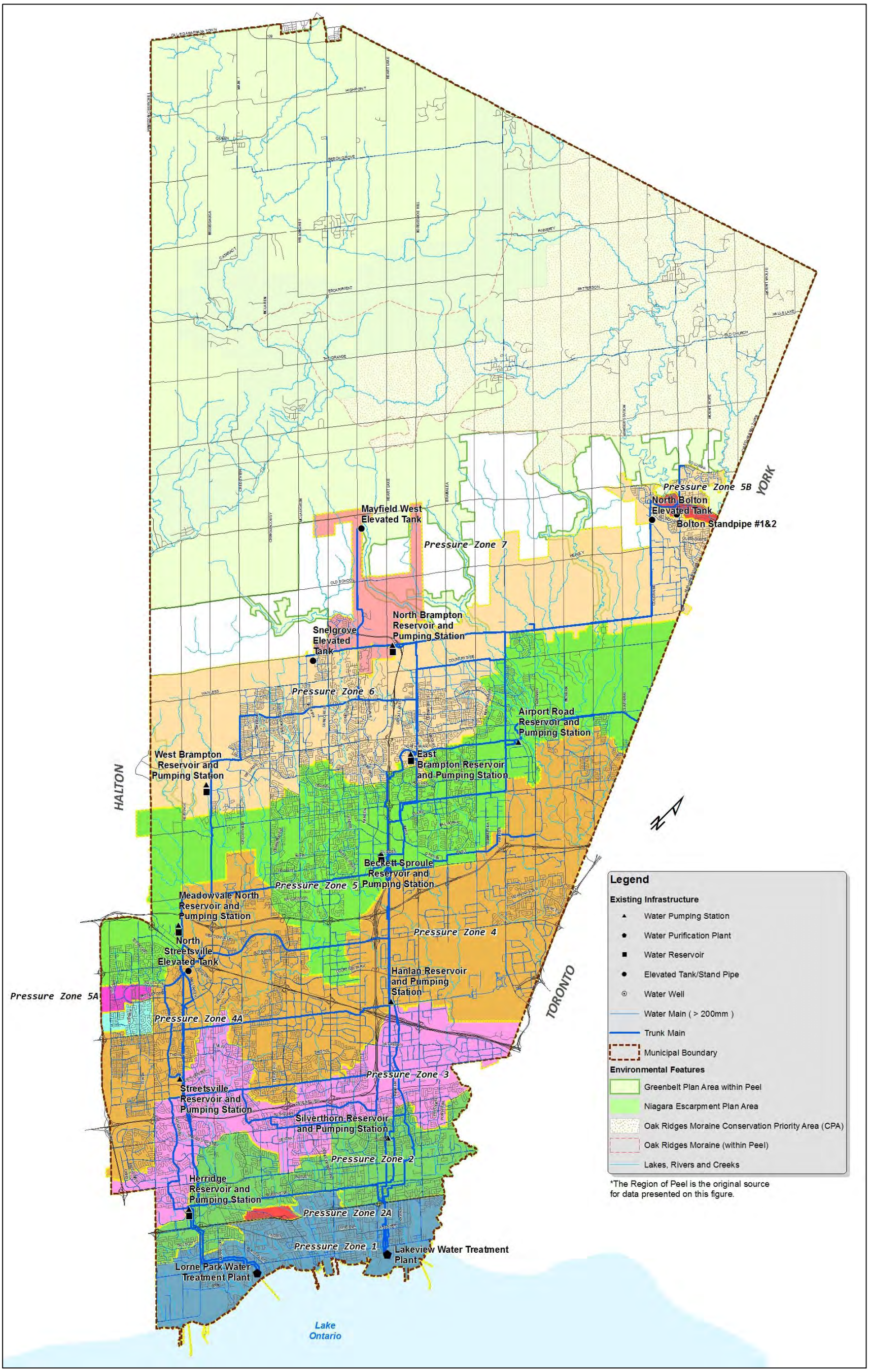


Figure 3.2 Existing Region of Peel Lake-Based Water Transmission System

Appendix C

Sanitary Servicing Calculations

Sanitary Flow Calculation

Project No. 4866

Proposed Residential Development - 7085 Goreway Drive, City of Mississauga

Site Area:	0.99	ha
Infiltration Rate:	0.2	l/ha/sec
Generation Rate:	302.8	l/person/day‡

Estimated Site Discharge

Land Use	Type	Area (ha.)	Units‡	Pop. Density (person/ha)†	Pop. Density (person/unit)†	Population	Average Flow (L/s)	Harmon's Peaking Factor	Peak Flow (L/s)	Infiltration (L/s)	Total Flow (L/s)
Residential	High-Rise Units	0.953	228	475	2.7	616	2.20	3.93	8.70	0.19	**8.89**
Commercial		0.037		50		2	0.01	4.46	0.03	0.01	**0.04**
		0.990									
					Total	**618**	2.21	3.93	~~**8.73 ***~~	0.20	**13.20**
									13.00 *		

† As per Region of Peel Design Criteria

‡ Based on site plan prepared by IBI Group

* Region of Peel Standard Drawing 2-9-2 states the domestic sewage flow for populations less than 1000 persons shall be 0.013 m^3/s

Population	Peak Flow (m^3/sec)	Population	Peak Flow (m^3/sec)	Population	Peak Flow (m^3/sec)
1000	0.0130	4750	0.0542	13000	0.1292
1050	0.0139	5000	0.0569	14000	0.1376
1100	0.0145	5250	0.0594	15000	0.1459
1150	0.0151	5500	0.0618	16000	0.1540
1200	0.0157	5750	0.0640	17000	0.1620
1300	0.0169	6000	0.0666	18000	0.1700
1400	0.0181	6250	0.0691	19000	0.1779
1500	0.0193	6500	0.0710	20000	0.1857
1600	0.0204	6750	0.0737	25000	0.2236
1700	0.0217	7000	0.0762	30000	0.2601
1800	0.0228	7250	0.0784	35000	0.2955
1900	0.0239	7500	0.0809	40000	0.3298
2000	0.0251	7750	0.0830	45000	0.3634
2200	0.0273	8000	0.0854	50000	0.3963
2400	0.0296	8250	0.0878	55000	0.4286
2600	0.0318	8500	0.0898	60000	0.4603
2800	0.0340	8750	0.0922	65000	0.4915
3000	0.0361	9000	0.0945	70000	0.5224
3250	0.0387	9250	0.0968	75000	0.5528
3500	0.0415	9500	0.0981	80000	0.5828
3750	0.0441	9750	0.1010	85000	0.6126
4000	0.0467	10000	0.1033	90000	0.6420
4250	0.0492	11000	0.1120	95000	0.6711
4500	0.0518	12000	0.1210	100000	0.7000

Notes:
1. Domestic sewage flows are based upon a unit sewage flow of 302.8 Lpcd.
2. The flows in the above table include the Harmon Peaking Factor.
3. Domestic sewage flow for less than 1000 persons shall be 0.013m^3/sec.
4. Domestic sewage flow for greater than 100,000 persons shall be 7.0 x 10^{-6} m^3/sec per capita.
5. Lpcd = Litres per capita per day 1 Litre = 0.001 $metre^3$

Region of Peel
Working for you

Date: June 2005 Rev: 1

Approved:

SEWAGE FLOWS
(EXCLUDING INFILTRATION)

STD. DWG. ~~2-5-2~~ 2-9-2

SUBDIVISION 7085 Goreway Drive
(CITY OF MISSISSAUGA)

CONSULTANT Schaeffer & Associates Ltd.

DRAINAGE AREA PLAN NO.

REGIONAL MUNICIPALITY OF PEEL

SANITARY SEWER DESIGN SHEET
PRE-DEVELOPMENT CONDITIONS

PROJECT No. 2019-4866
DESIGNED BY D.T. / H.S.
DATE 23-Apr-20

LOCATION	From MH	Up Stream Inv.	To MH	Down Stream Inv.	AREA (ha)	DENSITY persons per ha	POP	CUM. AREA (ha)	CUM. POP.	SEWAGE FLOW (L/sec)	INFILTRATION FLOW (L/sec)	FOUNDATION DRAINS (L/sec)	TOTAL FLOW (L/sec)	Length (m)	Pipe Dia NOM (mm)	Pipe Dia ACT (mm)	Grade (%)	Capacity (L/sec)	Full Velocity (m/s)
GOREWAY DRIVE - SOUTH			1A		0.51		33												
GOREWAY DRIVE - NORTH (West Side)			1A		3.40		123												
GOREWAY DRIVE - NORTH (East - excl.subject site)			1A		1.45	50	73												
GOREWAY DRIVE - NORTH (Subject site)			1A		0.99	50	50												
DORCAS STREET	**1A**		**2A**		**0.00**		**0**	**6.35**	**279**	**13.00**	**1.27**	**0.000**	**14.27**	**61.0**	**250**	**254.0**	**0.48**	**42.98**	**0.85**
								6.35	**279**										
MINOTOLA AVENUE - NORTH			2A		6.68		300												
MINOTOLA AVENUE	**2A**		**3A**		**1.82**		**271**	**14.85**	**850**	**13.00**	**2.97**	**0.000**	**15.97**	**67.0**	**250**	**254.0**	**0.50**	**43.87**	**0.87**
								14.85	**850**										

SUBDIVISION 7085 Goreway Drive
(CITY OF MISSISSAUGA)

CONSULTANT Schaeffer & Associates Ltd.

DRAINAGE AREA PLAN NO.

REGIONAL MUNICIPALITY OF PEEL

SANITARY SEWER DESIGN SHEET

POST-DEVELOPMENT CONDITIONS

PROJECT No. 2019-4866
DESIGNED BY G.V. / K.S.
DATE 10-Jul-23

LOCATION	From MH	Up Stream Inv.	To MH	Down Stream Inv.	AREA (ha)	DENSITY persons per ha	POP	CUM. AREA (ha)	CUM. POP.	SEWAGE FLOW (L/sec)	INFILTRATION FLOW (L/sec)	FOUNDATION DRAINS (L/sec)	TOTAL FLOW (L/sec)	Length (m)	Pipe Dia NOM (mm)	Pipe Dia ACT (mm)	Grade (%)	Capacity (L/sec)	Full Velocity (m/s)
GOREWAY DRIVE - SOUTH			1A		0.51		33												
GOREWAY DRIVE - NORTH (West Side)			1A		3.40		123												
GOREWAY DRIVE - NORTH (East - excl.subject site)			1A		1.45	50	73												
GOREWAY DRIVE - NORTH (Subject site)			1A		0.99		618												
DORCAS STREET	**1A**		**2A**		**0.00**		**0**	**6.35**	**847**	**13.00**	**1.27**	**0.000**	**14.27**	**61.0**	**250**	**254.0**	**0.48**	**42.98**	**0.85**
								6.35	**847**										
MINOTOLA AVENUE - NORTH			2A		6.68		300												
MINOTOLA AVENUE	**2A**		**3A**		**1.82**		**271**	**14.85**	**1418**	**18.37**	**2.97**	**0.000**	**21.34**	**67.0**	**250**	**254.0**	**0.50**	**43.87**	**0.87**
								14.85	**1418**										

SUBDIVISION PASTORIA HOLDINGS.
CONSULTANT.....
DRAINAGE AREA PLAN No. 48

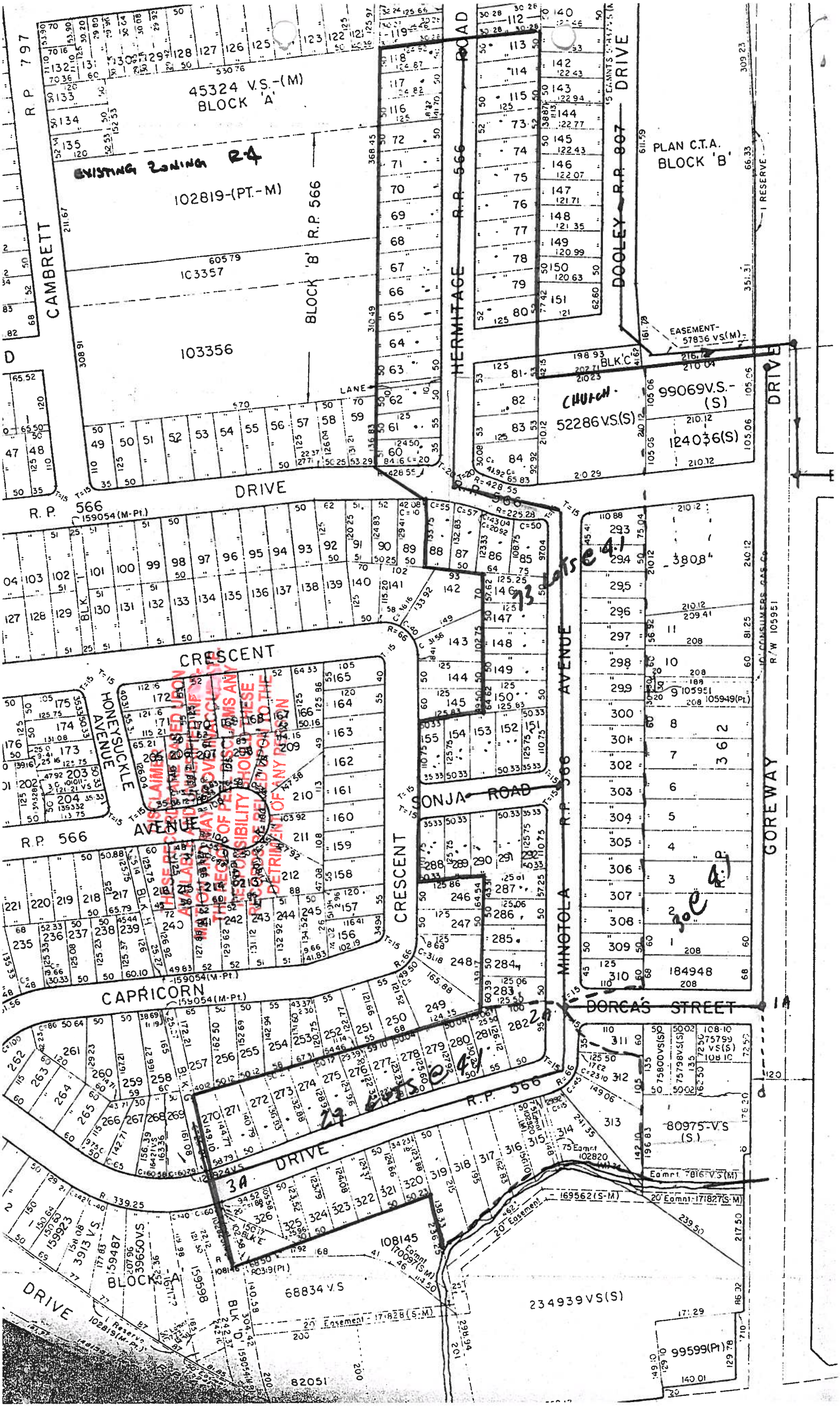
REGIONAL MUNICIPALITY OF PEEL
SANITARY SEWER DESIGN CHART

SHEET No..... OF.....
PROJECT No..... n= .....
DESIGNED..... DATE.....

Rd 248.2

LOCATION	FROM M.H.	TO M.H.	AREA (acres)	DENSITY persons per acre	POPULATION	CUM. AREA (acres)	CUM. POP.	SEWAGE FLOW ① (c.f.s.)	INFILTRATION FLOW ② (c.f.s.)	FOUNDATION DRAINS ③ (c.f.s.)	TOTAL FLOW ①+②+③ (c.f.s.)	LENGTH (ft.)	PIPE DIA. (Inches)	GRADIENT %	CAPACITY (c.f.s.)	VELOCITY (f.p.s.)	DROP IN LOWER M.H.
EXISTING SYSTEM.																	
	1A	2A															
GOREWAY NORTH TO 1A				30@4.1	123												
GOREWAY SOUTH TO 1A				8@4.1	33	6.90	156	.70	.50	—	1.27						
	2A	3A															
MINOTOLA NORTH			16.5	73@4.1	300	23.4	456	.70	.50	—	1.27		SYSTEM GRADE 10" @ .65%				
ADD PROPOSED PROPOSAL TO SYSTEM			4.5		271	27.9	727	.70	.50	—	1.27						
SUMMARY — EXISTING 10" SYSTEM WESTERLY TO MINOTOLA ADEQUATE IN TERMS OF CAPACITY																	
— NOT REC. DUE TO THE FOLLOWING:																	
1) CONNECTION MH. VERY SHALLOW (5')																	
2) HAVE TO CROSS NEWLY BUILT 4 LANE ARTERIAL ROAD																	
RECOMMEND TO CONNECT TO 30" TRUNK SEWER TO THE EAST																	

NOTE – SEWAGE FLOWS FOR LIGHT INDUSTRIAL AND LIGHT COMMERCIAL SHALL BE BASED ON 30 p.p.a. and 20 p.p.a. RESPECTIVELY.



CAMBRETT
R.P. 797
45324 V.S.-(M)
BLOCK 'A'
EXISTING ZONING R4
102819-(PT.-M)
BLOCK 'B' R.P. 566
103357
103356
LANE
HERMITAGE R.P. 566 ROAD
DOOLEY R.P. 807 DRIVE
PLAN C.T.A. BLOCK 'B'
1 RESERVE
EASEMENT-57836 VS(M)
BLK.C
CHURCH
52286 V.S.(S)
99069 V.S.-(S)
124036(S)
DRIVE
R.P. 566 159054(M-Pt.)
BLK. 'I'
CRESCENT
HONEYSUCKLE AVENUE
AVENUE
R.P. 566
BLK. H
MINOTOLA AVENUE R.P. 566
73 Lots @ 41
3808
1(C) CONSUMERS GAS Co. R/W 105951
105951
105949(Pt.)
362
SONJA ROAD
CRESCENT
30 @ 41
GOREWAY DRIVE
CAPRICORN
159054(M-Pt.)
DORCAS STREET
1A
2A
184948
29 Lots @ 41
R.P. 566
DRIVE
3A
75800 VS(S)
75798 VS(S)
75799 VS(S)
80975-V.S.(S)
75 Esmnt 102820
Esmnt. 7816 V.S.(M)
169562(S-M)
20' Esmnt-171827(S-M)
20' Easement
108145
Esmnt 170097(S-M)
BLK. E
80319(Pt)
159923
3913 V.S.
159487
39650 V.S.
BLOCK A
159868
68834 V.S.
234939 VS(S)
BLK 'D'
20' Easement-171828(S-M)
DRIVE
1 Reserve 102819(M-Pt.)
82051
99599(Pt)

Appendix D

Stormwater Management Calculations

SWM TANK ALLOWABLE RELEASE RATE CALCULATION

Existing Site Peak Flow Rates

Returning Period	Area (ha)	Runoff Coefficient C	Intensity (mm/hr)	Discharge Q (L/s)
2	0.765	0.50	59.89	63.7
5	0.765	0.50	80.51	85.6
10	0.765	0.50	99.17	105.5
25	0.765	0.55	113.89	133.2
50	0.765	0.60	127.13	162.2
100	0.765	0.63	140.69	187.0

Post-Development Uncontrolled Release Rate to Goreway

Returning Period	Area (ha)	Runoff Coefficient C	Intensity (mm/hr)	Discharge Q (L/s)
2	0.023	0.85	59.89	3.3
5	0.023	0.85	80.51	4.4
10	0.023	0.85	99.17	5.4
25	0.023	0.94	113.89	6.8
50	0.023	1.00	127.13	8.1
100	0.023	1.00	140.69	9.0

Post-Development SWM Tank Allowable Release Rate

Returning Period	Area (ha)	Runoff Coefficient C	Tank Allowable Release Rate (L/s)
2	0.742	0.85	60.4
5	0.742	0.85	81.2
10	0.742	0.85	100.1
25	0.742	0.94	126.4
50	0.742	1.00	154.1
100	0.742	1.00	178.0

100-year Required Storage

Project: 4866

Modified Rational Method

Internal Area	Controlled Area (ha) =	0.742
	100 year C =	1.00
	100yr Allowable Release Rate (l/s) =	178.0
	Actual Release Rate (l/s) =	60.4
External Area	Area (ha) =	0.000
	C =	0.00
	100-year C =	0.00
Roof Storage	Release Rate from roof(l/s) =	0.00

100 Year Storm

Design Storm =	City of Mississauga
A =	1450
B =	4.9
C =	0.78

	100 Year					Total	Maximum	Required
Time		Total	Rooftop	External	Total	Runoff	Release	Storage
(min)	Intensity	Runoff	Runoff	Runoff	Runoff	Volume	Volume	Volume
15	140.69	290.21	0.00	0.00	290.21	261.19	54.36	206.83
20	118.12	243.66	0.00	0.00	243.66	292.39	72.48	219.91
25	102.41	211.25	0.00	0.00	211.25	316.87	90.60	226.27
30	90.77	187.25	0.00	0.00	187.25	337.04	108.72	228.32
35	81.77	168.68	0.00	0.00	168.68	354.22	126.84	227.38
40	74.58	153.84	0.00	0.00	153.84	369.21	144.96	224.25
45	68.68	141.68	0.00	0.00	141.68	382.53	163.08	219.45
50	63.75	131.51	0.00	0.00	131.51	394.52	181.20	213.32
55	59.56	122.86	0.00	0.00	122.86	405.45	199.32	206.13
60	55.95	115.42	0.00	0.00	115.42	415.50	217.44	198.06
65	52.81	108.92	0.00	0.00	108.92	424.80	235.56	189.24
70	50.03	103.21	0.00	0.00	103.21	433.48	253.68	179.80
75	47.58	98.14	0.00	0.00	98.14	441.61	271.80	169.81
80	45.38	93.60	0.00	0.00	93.60	449.27	289.92	159.35
85	43.39	89.51	0.00	0.00	89.51	456.51	308.04	148.47
90	41.60	85.81	0.00	0.00	85.81	463.39	326.16	137.23
95	39.97	82.44	0.00	0.00	82.44	469.93	344.28	125.65
100	38.47	79.36	0.00	0.00	79.36	476.17	362.40	113.77
105	37.10	76.53	0.00	0.00	76.53	482.15	380.52	101.63
110	35.84	73.92	0.00	0.00	73.92	487.88	398.64	89.24
115	34.66	71.51	0.00	0.00	71.51	493.39	416.76	76.63
120	33.58	69.26	0.00	0.00	69.26	498.69	434.88	63.81

Required Storage (m^3): 228

City of Mississauga
Orifice Plate

Allowable Release Rate = 0.060 m^3/s

CALCULATE DIAMETER KNOWING Q & H	
Q(m^3/s)=	0.000
Td(m) =	0.27
Approx A=	0.0000
Approx D=	0
A(m^2) =	0.000
D(mm) =	**0**

Control Manhole Orifice Plate

DIA (mm)=	**172**	
AREA m^2=	0.023	
COEFF =	**0.62**	
GRAVITY =	9.81	
K =	1.0	
D/S HGL=	N/A	m
Orifice Inv.=	163.95	m

Effective Head m	Depth Water At CTL MH m	Qp m^3/s	TOTAL FLOW Qp m^3/s	ELEVATION of Water m	
0.00	0.086	0.000	0.000	164.04	
0.320	0.406	0.036	0.036	164.36	
0.400	0.486	0.040	0.040	164.44	
0.895	**0.981**	**0.060**	**0.060**	**164.93**	**100-year**
2.000	2.086	0.090	0.090	166.04	
2.320	2.406	0.097	0.097	166.36	
3.000	3.086	0.111	0.111	167.04	

ORIFICE FLOW $Q(m^3/s)=$ COEF*AREA*(2*GRAVITY*HEAD/K)^0.5

WEIR FLOW $Q(m^3/s)=$ CLH^1.5 C=1.5

Schaeffers Consulting Engineers

Printed: 19-May-22

STANDARD OFFLINE
Jellyfish Filter Sizing Report

Project Information

Date	Wednesday, May 11, 2022
Project Name	Mississauga
Project Number	4866
Location	Mississauga

Jellyfish Filter Design Overview

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

Please see ***www.ImbriumSystems.com*** *for more information.*

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF6-5-1 is recommended to meet the water quality objective by treating a flow of 27.8 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 313 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF6-5-1	5	1	1.8	27.8	313

The Jellyfish Filter System

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

Maintenance

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

Please see ***www.ImbriumSystems.com*** *for more information.*

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

Performance

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

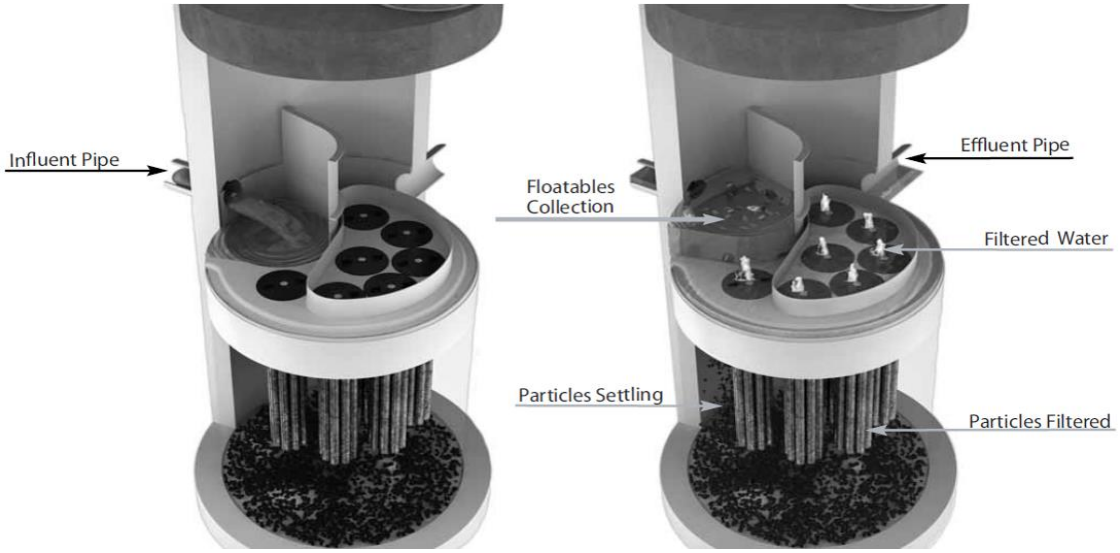
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 77% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash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

Field Proven Peformance

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

- A median TSS removal efficiency of 89%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitotred storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 77%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Wednesday, May 11, 2022
Project Name:	Mississauga
Project Number:	4866
Location:	Mississauga

Designer Information

Company:	Schaeffers Consulting Engineers
Contact:	Giancarlo Volpe
Phone #:	

Notes

Rainfall

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

Drainage Area

Total Area:	0.792 ha
Imperviousness:	93%

Upstream Detention

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

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 18 years of TORONTO CENTRAL rainfall data:	**20.2 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 4393 m³, with a suspended sediment concentration of 60 mg/L.	**264 kg***

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

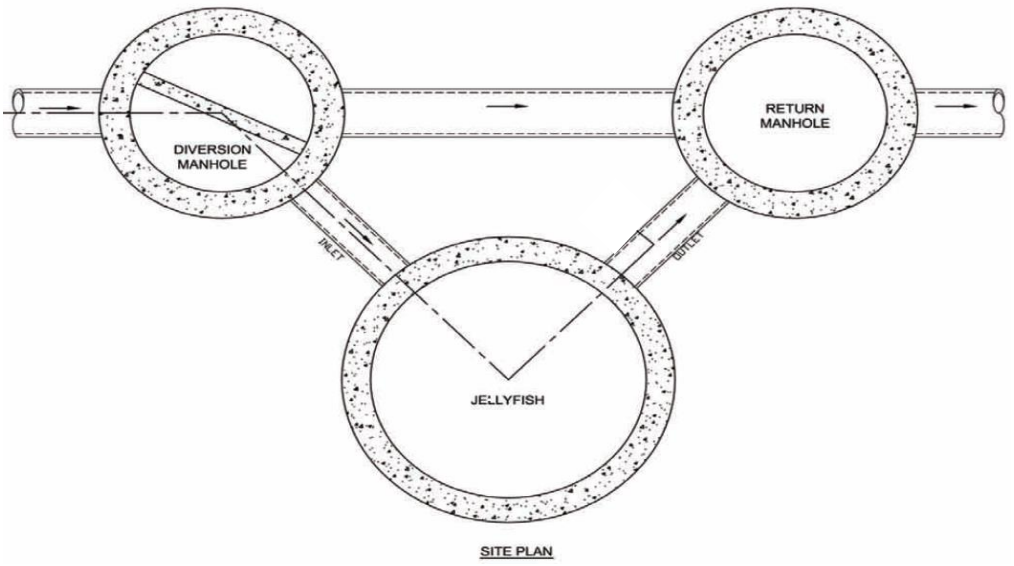
Recommendation

The Jellyfish Filter model JF6-5-1 is recommended to meet the water quality objective by treating a flow of 27.8 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 313 kg, which meets or exceeds the estimated average annual sediment load.

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

Jellyfish Filter Design Notes

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



Jellyfish Filter Typical Layout

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

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

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

STANDARD SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

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

1.2 REFERENCE STANDARDS

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

CAN/CSA-A257.4-M92
Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92
Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

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

1.4 PRODUCT SUBSTITUTIONS

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

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.

2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft² (0.142 lps/m²).

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

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft2 / m2)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

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

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

2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.

2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.

2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

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

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

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

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

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

2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.

2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.

2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.

2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).

3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.

3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.

3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.

3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

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

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.

3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.

3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.

3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).

3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.

3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 INSTALLATION

4.1.1 PRECAST DEVICE CONSTRUCTION SEQUENCE

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

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

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

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

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

4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.

4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

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

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

PART 5 – QUALITY ASSURANCE

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

5.2 INSPECTION AND MAINTENANCE

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

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

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

END OF SECTION

TABLE 2.01.03.03c: STORMWATER QUANTITY CONTROL REQUIREMENTS

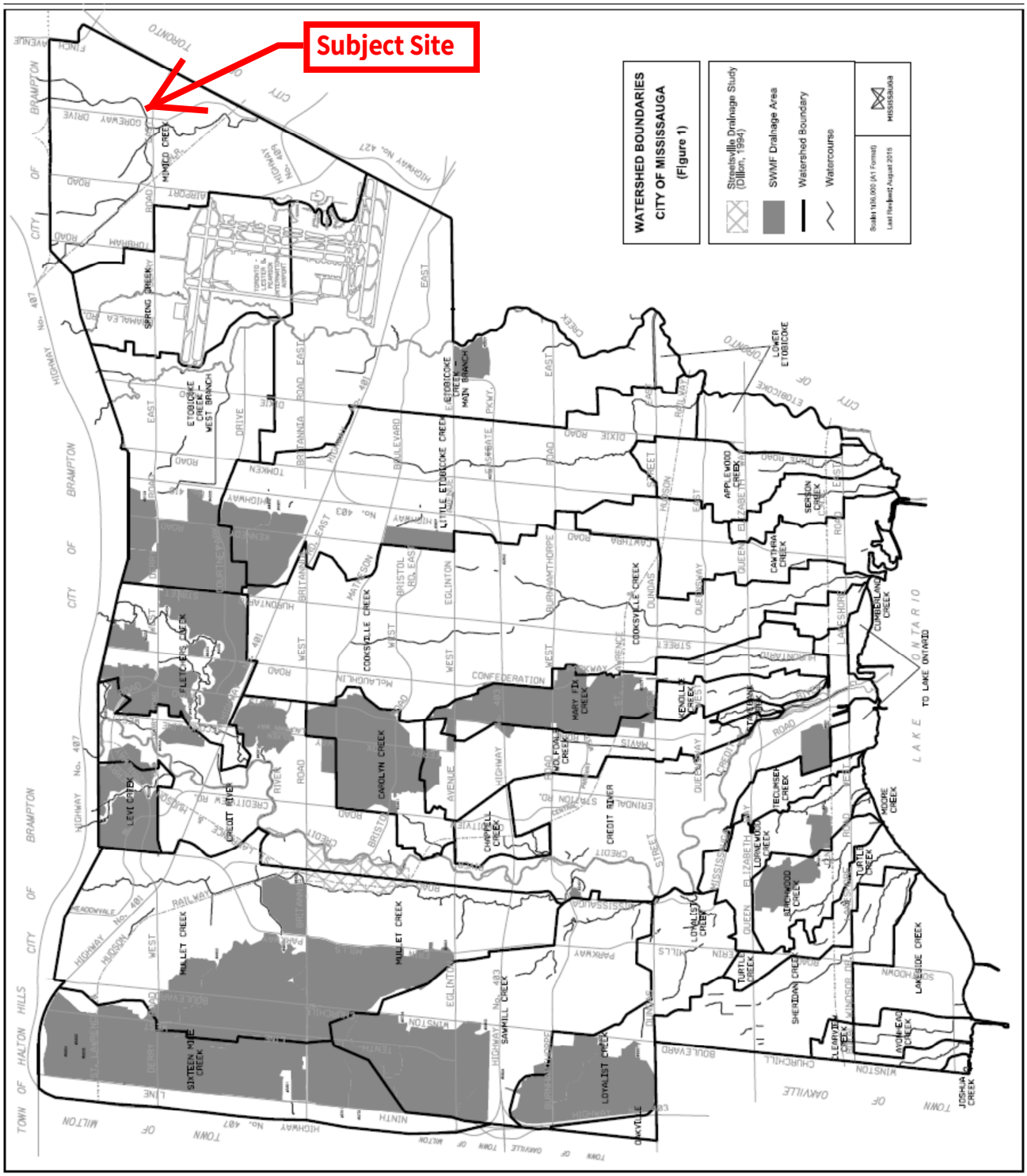
Note 1: In all cases, the storm sewer capacity constraints may govern

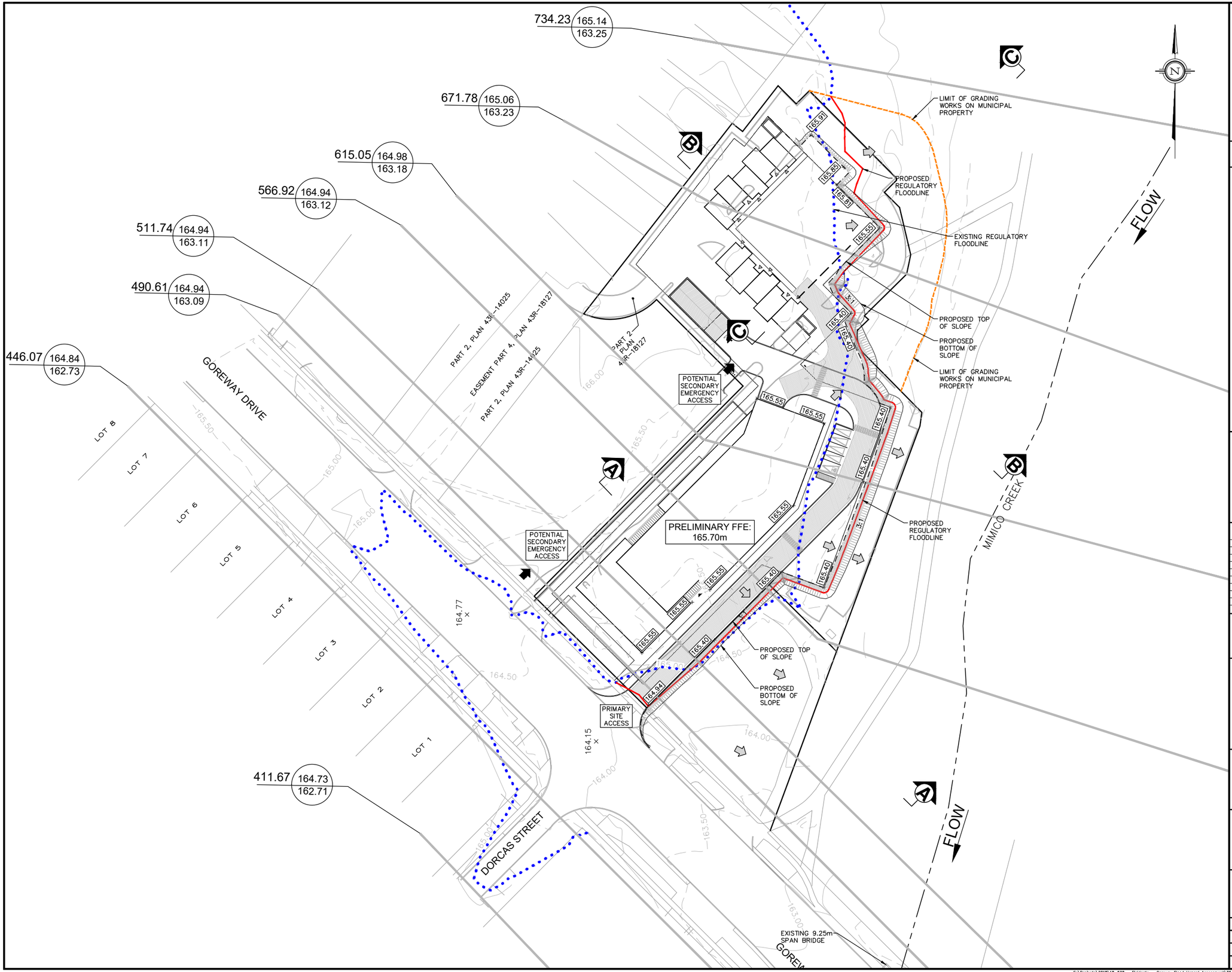
Note 2: Where "pre-development" is listed as part of the requirement, it is implied as raw land for which the run-off co-efficient=0.25 but will not exceed 0.50 for a site that may already be developed

Note 3: CVC-Credit Valley Conservation, TRCA-Toronto Region Conservation Authority, CH-Conservation Halton

Subwatershed Name (Conservation Authority)	Quantity Control Criteria	References & Notes
Loyalist Creek (CVC)	East of Winston Churchill Blvd - Provide post to pre control for only 10 year design storm	Loyalist Creek Watershed Study (CBCL Limited, 1980)
	West of Winston Churchill Blvd - Provide post to pre control for all storms (i.e. 2,5,10,25,50 & 100 year)	Erin Mills West Loyalist Creek Drainage Report (Proctor & Redfern Group, 1985)
Mary Fix Creek (CVC)	10 Year Post to 2 Year Pre-development Control	-
Mimico Creek (TRCA)	Provide post to pre control for all storms (i.e. 2,5,10,25,50 & 100 year)	Hydrologic Model: VISUAL OTTHYMO-Return period peak flows based on the AES - 12 hour design storm Hydrology Study:Mimico Hydrology Update (Marshall Macklin Monaghan, 2009)
Moore Creek (CVC)	No control required	-
Mullet Creek (CVC)	Provide post to pre control for all storms (i.e. 2,5,10,25,50 & 100 year) & Regional storm	Hydrologic Model: GAWSER Model-Return period peak flows based on 24 hour SCS Type II distribution
	Consider storm sewer constraints outlined in Streetsville Area Drainage Study (Dillon, 1994)	Gateway West Subwatershed Study (Gartner Lee Limited & Cosburn Patterson Mather, 1999)
	□	Gateway West Subwatershed Study Update by Kidd Consulting (Update in Progress)
Sawmill Creek (CVC)	Provide post to pre control for all storms (i.e. 2,5,10,25,50 & 100 year)	Hydrologic Model: GAWSER Model-Return period peak flows based on 24 hour SCS Type II distribution Sawmill Creek Subwatershed Study (Proctor & Redfern Limited, 1993)
Serson Creek (CVC)	100 Year Post to 2 Year Pre-development Control	Large number of buildings (> 150) in the regulated flood plain

A-1 - Watershed Boundaries





E:\Projects\2018\18-528 - Fieldgate - Goreway Flood Hazard Assessment\05_Design\051_DRAWINGS\528_BASE.dwg

Date Plotted: Feb 23, 2022 - 2:03pm

KEY PLAN
N.T.S.

LEGEND

- PROPERTY LINE
- EXISTING RIVER
- EX. MAJOR CONTOUR
- EX. MINOR CONTOUR
- GRECK EXISTING CONDITIONS REGULATORY FLOODLINE
- PROPOSED REGULATORY FLOODLINE
- PROPOSED BOTTOM OF SLOPE
- PROPOSED TOP OF SLOPE
- OVERLAND FLOW
- × 165.55 PRELIMINARY PROPOSED GRADES (TO BE CONFIRMED DURING DETAILED DESIGN)

GRECK REGULATORY FLOODLINE
165.06 / 163.23 — 671.78 HEC-RAS ID
100yr FLOODLINE
SECTION LINE

BENCHMARK(S)
BENCHMARK No. BM448
ELEVATION = 162.55m
LOCATION: MISSISSAUGA, ON
DESCRIPTION:

COMPLETED BY:
KORMAR ONTARIO LAND SURVEYORS
1137 CENTRE ST. THORNHILL, ONTARIO
COMPLETED: JUNE 20, 2017 (905) 738-0053

NO.	REVISION	DATE	BY	APPROVED

Greck
5770 Highway 7, Woodbridge, Ontario, L4L 1T8

CLIENT NAME:
7085 GOREWAY DEVELOPMENTS LIMITED

7085 GOREWAY DRIVE MISSISSAUGA, ONTARIO

FLOODLINE HAZARD ASSESSMENT PROPOSED CONCEPTUAL DESIGN

DESIGNED BY: S.S.	SCALES:		PROJECT No. 18-528
CHECKED BY: E.G.	HORIZONTAL:	1:1000	DRAWING No. FPM
DRAWN BY: J.N.	VERTICAL:	N/A	SHEET No. 01
DATE: JULY 20, 2018			

Appendix E

Engineering Drawings

Project: 7085 Goreway Drive
(CITY OF MISSISSAUGA)
Consultant: Schaeffer & Associates Ltd.
DRAINAGE AREA PLAN NO.: N/A

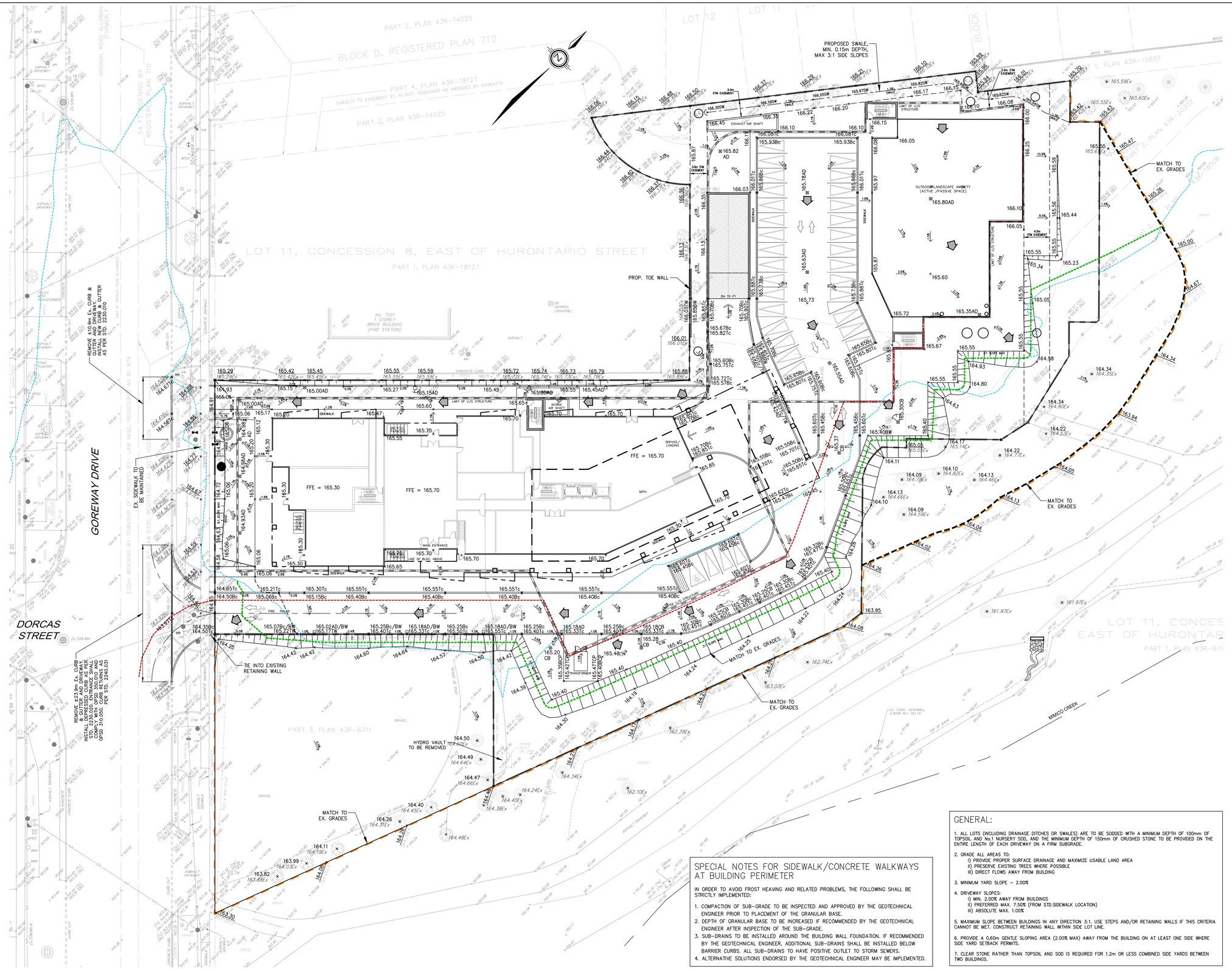
STORM SEWER DESIGN SHEET

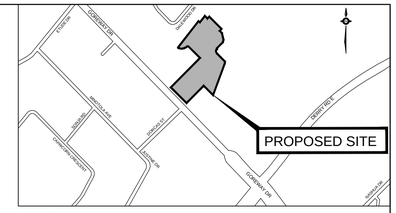
CITY OF MISSISSAUGA
7085 Goreway Drive - Post Development

PROJECT No.: 2019-4866
DESIGNED BY: H.H.T.
CHECKED BY:
DATE: May 26, 2022

AREA NO	LOCATION	LAND USE	UPSTREAM		DOWNSTREAM		NO. OF HECTARES		AREA x STORM C0-EFF.				TIME OF CONCENTRATION		$I10_{YR}$	$I100_{YR}$	Q_{10}=2.78 x CIA / 1000 (m^3/s)	Q_{100}=2.78 x CIA / 1000 (m^3/s)	PIPE						
			MH	INV	MH	INV	IN AREA	TOTAL	C	INCR AxC	TOTAL SECT AxC	TOTAL AxCx2.78	IN AREA	TOT					Length (m)	SIZE NOM (mm)	SIZE ACT (mm)	GRADE	TYPE OF PIPE	CAPACITY (m^3/s)	VELOCITY (m/s)
	Fire Hall				1		0.340	0.34	0.94	0.319	0.319	0.886		15.00		140.69		**0.125**	= 100-yr capture rate from Fire Hall						
	Easement - 3.0m	BY-PASS	1		2		0.00	0.00	0.00	0.000	0.000	0.000	0.81	15.00	99.17		0.000	0.125	41.6	450	457	0.30	CONC	0.141	0.86
	Easement - 3.0m	BY-PASS	2		3		0.00	0.00	0.00	0.000	0.000	0.000	0.92	15.81	96.10		0.000	0.125	47.5	450	457	0.30	CONC	0.141	0.86
	Easement - 3.0m	BY-PASS	3		4		0.00	0.00	0.00	0.000	0.000	0.000	0.17	16.73	92.85		0.000	0.125	3.6	450	457	0.30	CONC	0.141	0.86
	Easement - 3.0m	BY-PASS	4		5		0.00	0.00	0.00	0.000	0.000	0.000	0.55	16.90	92.26		0.000	0.125	11.3	450	457	0.30	CONC	0.141	0.86
	Easement - 4.5m	BY-PASS	5		6		0.00	0.00	0.00	0.000	0.000	0.000	0.80	16.90	92.26		0.000	0.125	41.3	450	457	0.30	CONC	0.141	0.86
								0.00			**0.000**			**17.70**											
	7085 Goreway Drive		7		6								100-YR Controlled Flow From The Tank =					**0.0637**	8.6	250	254	2.00	UR-PVC	0.076	1.50
	Easement - 4.5m	BY-PASS	6		HW.1		0.000	0.00	0.00	0.000	0.000	0.000	0.72	15.00	142.37	140.69	0.000	**0.188**	53.2	525	533	0.50	CONC	0.275	1.23

$I10_{YR}=35.1(T)^{-0.695}$, T in hours
Inlet Time = 15 min





NOTES

1. THE LOCATION OF ALL UNDERGROUND AND ABOVE GROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON CONTRACT DRAWINGS, AND WHERE SHOWN THE ACCURACY OF THE LOCATION AND ELEVATION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. PRIOR TO COMMENCING CONSTRUCTION, THE CONTRACTOR SHALL VERIFY EXACT LOCATION AND ELEVATION OF SUCH UTILITIES AND STRUCTURES AND SHALL ASSUME ALL LIABILITIES OF DAMAGE.
2. ANY CONFLICTS WITH EXISTING SERVICES AND/OR UTILITIES SHALL BE REPORTED TO THE ENGINEER FOR REVIEW AND ADVICE PRIOR TO COMMENCEMENT OF CONSTRUCTION.
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LEGEND

- DENOTES HYDRANT
- AD DENOTES AREA DRAIN
- DENOTES CATCHBASIN
- DENOTES STORM MANHOLE
- DENOTES SANITARY MANHOLE
- VC DENOTES VALVE & CHAMBER
- VB DENOTES VALVE & BOX
- DENOTES EXISTING REGULATORY FLOODPLAIN
- DENOTES LINE OF FLOODPLAIN (REGIONAL)
- DENOTES 6.0m SETBACK FROM PROPOSED ENGINEERED LONG TERM STABLE TOP OF SLOPE
- DENOTES EXISTING PROPERTY LINE
- DENOTES PROPOSED PROPERTY LINE
- DENOTES LINE OF U/G
- OHR DENOTES EXISTING OVERHEAD HYDRO
- DENOTES LIMIT OF GRADING
- + 131.48 DENOTES PROPOSED ELEVATION
- + 131.48TC DENOTES PROPOSED TOP OF CURB
- + 131.48BC DENOTES PROPOSED BOTTOM OF CURB
- + 131.48TW DENOTES PROPOSED TOP OF WALL
- + 131.48BW DENOTES PROPOSED BOTTOM OF WALL
- + 131.48SW DENOTES PROPOSED SWALE
- + *130.29EX* DENOTES EXISTING ELEVATION
- DENOTES EMERGENCY OVERLAND FLOW ARROW

BENCHMARK NOTE

ELEVATIONS SHOWN HEREON ARE GEODETIC(1928) AND ARE RELATED TO CITY OF MISSISSAUGA BENCH MARK No. BM448 HAVING PUBLISHED ELEVATION OF 162.55 METRES.

No.	Date	Issued for
4.	JULY.07.2023	ISSUED FOR ZBA SUBMISSION
3.	MAR.08.2023	ISSUED FOR ZBA SUBMISSION
2.	JUNE.06.2022	ISSUED FOR ZBA SUBMISSION
1.	APR.24.2020	ISSUED FOR ZBA SUBMISSION

REGISTERED PROFESSIONAL ENGINEER
H. H. TOZCU
100130655
JULY.07.2023
PROVINCE OF ONTARIO

SPECIAL NOTES FOR SIDEWALK/CONCRETE WALKWAYS AT BUILDING PERIMETER

IN ORDER TO AVOID FROST HEAVING AND RELATED PROBLEMS, THE FOLLOWING SHALL BE STRICTLY IMPLEMENTED:

1. COMPACTION OF SUB-GRADE TO BE INSPECTED AND APPROVED BY THE GEOTECHNICAL ENGINEER PRIOR TO PLACEMENT OF THE GRANULAR BASE.
2. DEPTH OF GRANULAR BASE TO BE INCREASED IF RECOMMENDED BY THE GEOTECHNICAL ENGINEER AFTER INSPECTION OF THE SUB-GRADE.
3. SUB-DRAINS TO BE INSTALLED AROUND THE BUILDING WALL FOUNDATION. IF RECOMMENDED BY THE GEOTECHNICAL ENGINEER, ADDITIONAL SUB-DRAINS SHALL BE INSTALLED BELOW BARRIER CURBS. ALL SUB-DRAINS TO HAVE POSITIVE OUTLET TO STORM SEWERS.
4. ALTERNATIVE SOLUTIONS ENDORSED BY THE GEOTECHNICAL ENGINEER MAY BE IMPLEMENTED.

GENERAL:

1. ALL LOTS (INCLUDING DRAINAGE DITCHES OR SWALES) ARE TO BE SODDED WITH A MINIMUM DEPTH OF 100mm OF TOPSOIL AND No.1 NURSERY SOD, AND THE MINIMUM DEPTH OF 150mm OF CRUSHED STONE TO BE PROVIDED ON THE ENTIRE LENGTH OF EACH DRIVEWAY ON A FIRM SUBGRADE.
2. GRADE ALL AREAS TO:
   I) PROVIDE PROPER SURFACE DRAINAGE AND MAXIMIZE USABLE LAND AREA
   II) PRESERVE EXISTING TREES WHERE POSSIBLE
   III) DIRECT FLOWS AWAY FROM BUILDING
3. MINIMUM YARD SLOPE – 2.00%
4. DRIVEWAY SLOPES:
   I) MIN. 2.00% AWAY FROM BUILDINGS
   II) PREFERRED MAX. 7.50% (FROM STD.SIDEWALK LOCATION)
   III) ABSOLUTE MAX. 1.00%
5. MAXIMUM SLOPE BETWEEN BUILDINGS IN ANY DIRECTION 3:1. USE STEPS AND/OR RETAINING WALLS IF THIS CRITERIA CANNOT BE MET. CONSTRUCT RETAINING WALL WITHIN SIDE LOT LINE.
6. PROVIDE A 0.60m GENTLE SLOPING AREA (2.00% MAX) AWAY FROM THE BUILDING ON AT LEAST ONE SIDE WHERE SIDE YARD SETBACK PERMITS.
7. CLEAR STONE RATHER THAN TOPSOIL AND SOD IS REQUIRED FOR 1.2m OR LESS COMBINED SIDE YARDS BETWEEN TWO BUILDINGS.

REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

CITY OF MISSISSAUGA

SCHAEFFERS
CONSULTING ENGINEERS
SCHAEFFER & ASSOCIATES LTD.
6 Ronrose Drive, Concord, Ontario L4K 4R3
Tel: (905) 738-6100
Fax: (905) 738-6875
E-mail: design@schaeffers.com

PRELIMINARY SITE GRADING PLAN

DRAWN BY: M.P.	DESIGNED BY: M.P.	CHECKED BY: H.S.
SCALE: 1:300	DATE: APR. 2020	
PROJECT No. 2019-4866	DRAWING No. SG-1	

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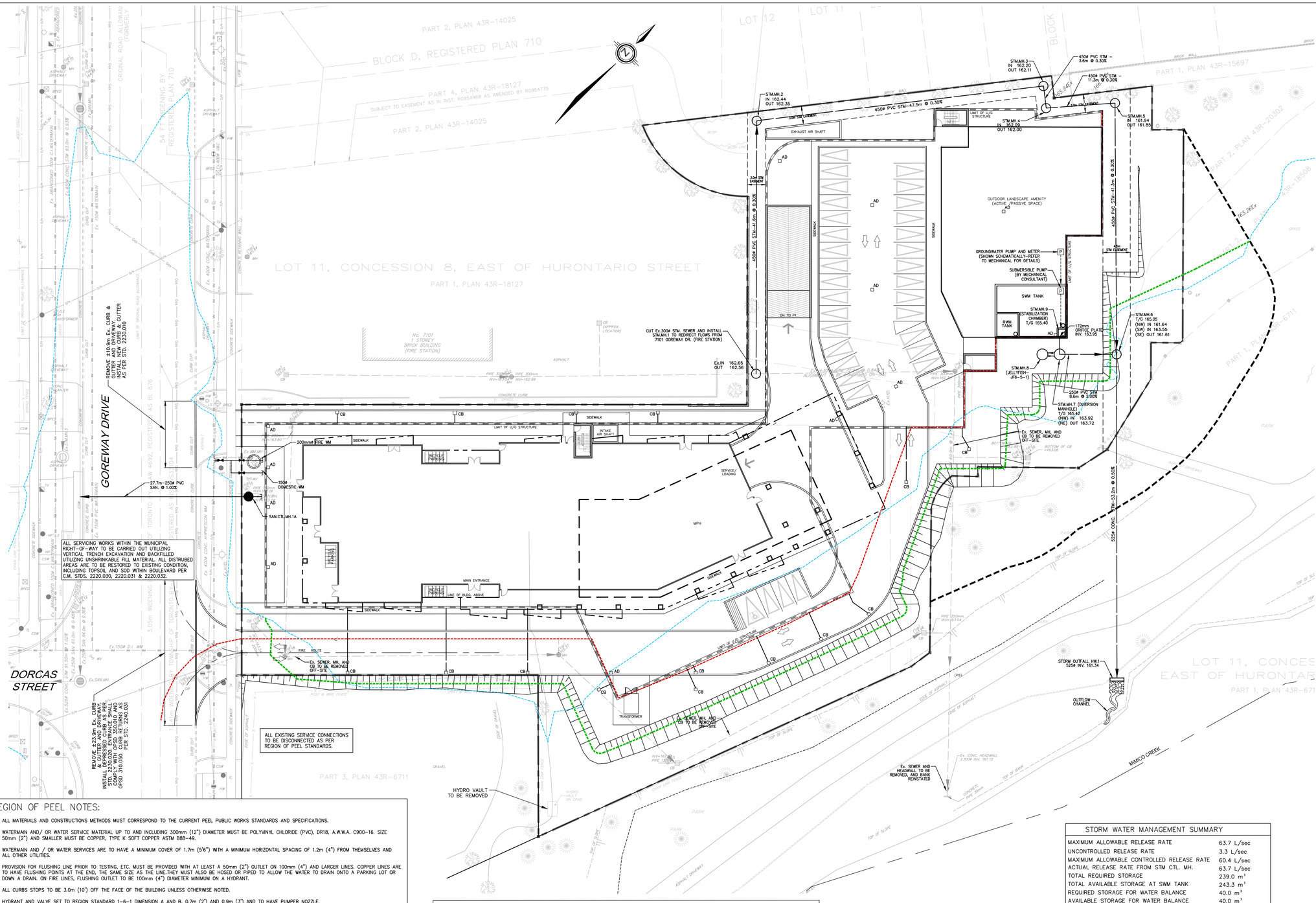

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LEGEND

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- DENOTES SANITARY MANHOLE
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- VB — DENOTES VALVE & BOX
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- DENOTES LINE OF FLOODPLAIN (REGIONAL)
- DENOTES 6.0m SETBACK FROM PROPOSED ENGINEERED LONG TERM STABLE TOP OF SLOPE
- DENOTES EXISTING PROPERTY LINE
- DENOTES PROPOSED PROPERTY LINE
- DENOTES LINE OF U/G
- OHR — DENOTES EXISTING OVERHEAD HYDRO
- Gas — DENOTES EXISTING GAS MAIN
- B — DENOTES EXISTING BELL CABLE

BENCHMARK NOTE

ELEVATIONS SHOWN HEREON ARE GEODETIC(1928) AND ARE RELATED TO CITY OF MISSISSAUGA BENCH MARK No. BM448 HAVING PUBLISHED ELEVATION OF 162.55 METRES.

No.	Date	Issued for
4.	JULY.07.2023	ISSUED FOR ZBA SUBMISSION
3.	MAR.08.2023	ISSUED FOR ZBA SUBMISSION
2.	JUNE.06.2022	ISSUED FOR ZBA SUBMISSION
1.	APR.24.2020	ISSUED FOR ZBA SUBMISSION

REGISTERED PROFESSIONAL ENGINEER
H./H. TOZCU
100130655
JULY.07.2023
PROVINCE OF ONTARIO

REDWOOD ON GOREWAY
7085 GOREWAY DRIVE

CITY OF MISSISSAUGA

SCHAEFFERS
CONSULTING ENGINEERS
SCHAEFFER & ASSOCIATES LTD.
6 Ronrose Drive, Concord, Ontario L4K 4R3
Tel: (905) 738-6100
Fax: (905) 738-6875
E-mail: design@schaeffers.com

PRELIMINARY SITE SERVICING PLAN

DRAWN BY: M.P.	DESIGNED BY: M.P.	CHECKED BY: H.S.
SCALE: 1:300	DATE: APR. 2020	
PROJECT No. 2019-4866	DRAWING No. SS-1	

REGION OF PEEL NOTES:

1. ALL MATERIALS AND CONSTRUCTIONS METHODS MUST CORRESPOND TO THE CURRENT PEEL PUBLIC WORKS STANDARDS AND SPECIFICATIONS.
2. WATERMAIN AND/ OR WATER SERVICE MATERIAL UP TO AND INCLUDING 300mm (12") DIAMETER MUST BE POLYVINYL CHLORIDE (PVC), DR18, A.W.W.A. C900-16. SIZE 50mm (2") AND SMALLER MUST BE COPPER, TYPE K SOFT COPPER ASTM B88-49.
3. WATERMAIN AND / OR WATER SERVICES ARE TO HAVE A MINIMUM COVER OF 1.7m (5'6") WITH A MINIMUM HORIZONTAL SPACING OF 1.2m (4") FROM THEMSELVES AND ALL OTHER UTILITIES.
4. PROVISION FOR FLUSHING LINE PRIOR TO TESTING, ETC. MUST BE PROVIDED WITH AT LEAST A 50mm (2") OUTLET ON 100mm (4") AND LARGER LINES. COPPER LINES ARE TO HAVE FLUSHING POINTS AT THE END, THE SAME SIZE AS THE LINE.THEY MUST ALSO BE HOSED OR PIPED TO ALLOW THE WATER TO DRAIN ONTO A PARKING LOT OR DOWN A DRAIN. ON FIRE LINES, FLUSHING OUTLET TO BE 100mm (4") DIAMETER MINIMUM ON A HYDRANT.
5. ALL CURBS STOPS TO BE 3.0m (10') OFF THE FACE OF THE BUILDING UNLESS OTHERWISE NOTED.
6. HYDRANT AND VALVE SET TO REGION STANDARD 1-6-1 DIMENSION A AND B, 0.7m (2') AND 0.9m (3') AND TO HAVE PUMPER NOZZLE.
7. WATERMAIN TO BE INSTALLED TO GRADES AS SHOWN ON APPROVED SITE PLAN. COPY OF GRADE SHEET MUST BE SUPPLIED TO INSPECTOR PRIOR TO COMMENCEMENT OF WORK, WHERE REQUESTED BY INSPECTOR.
8. WATERMAIN MUST HAVE A MINIMUM VERTICAL CLEARANCE OF 0.30m (12") OVER / 0.5m (20") UNDER SEWERS AND ALL OTHERS UTILITIES WHEN CROSSING.
9. ALL PROPOSED WATER PIPING MUST BE ISOLATED FROM EXISTING LINES IN ORDER TO ALLOW INDEPENDENT PRESSURE TESTING AND CHLORINATING FROM EXISTING SYSTEMS.
10. ALL LIVE TAPPING AND OPERATION OF REGION WATER VALVES SHALL BE ARRANGED THROUGH THE REGIONAL INSPECTOR ASSIGNED OR BY CONTACTING THE OPERATIONS AND MAINTENANCE DIVISION.
11. ALL PROPOSED WATER PIPING MUST BE ISOLATED THROUGH A TEMPORARY CONNECTION THAT SHALL INCLUDE AN APPROPRIATE CROSS-CONNECTION CONTROL DEVICE, CONSISTENT WITH THE DEGREE OF HAZARD, FOR BACKFLOW PREVENTION OF THE ACTIVE DISTRIBUTION SYSTEM, CONFORMING TO REGION OF PEEL STANDARD 1-7-7 OR 1-7-8.
12. LOCATION OF ALL EXISTING UTILITIES IN THE FIELD TO BE ESTABLISHED BY THE CONTRACTOR.
13. THE CONTRACTOR(S) SHALL BE SOLELY RESPONSIBLE FOR LOCATES, EXPOSING, SUPPORTING AND PROTECTING OF ALL UNDERGROUND AND OVERHEAD UTILITIES AND STRUCTURES EXISTING AT THE TIME OF CONSTRUCTION IN THE AREA OF THEIR WORK WHETHER SHOWN ON THE PLANS OR NOT AND FOR ALL REPAIRS AND CONSEQUENCES RESULTING FROM DAMAGE TO SAME.
14. THE CONTRACTOR(S) SHALL BE SOLELY RESPONSIBLE TO GIVE 72 HOURS WRITTEN NOTICE TO THE UTILITIES PRIOR TO CROSSING SUCH UTILITIES, FOR THE PURPOSE OF INSPECTION BY THE CONCERNED UTILITY. THIS INSPECTION WILL BE FOR THE DURATION OF THE CONSTRUCTION, WITH THE CONTRACTOR RESPONSIBLE FOR ALL COSTS ARISING FROM SUCH INSPECTION.

GENERAL NOTES

1. ALL CONCRETE AND PLASTIC SEWER PIPE SHALL HAVE RUBBER GASKET JOINTS.
2. ALL SEWERS SHALL BE CONSTRUCTED WITH BEDDING IN ACCORDANCE WITH OPSD 802.03 CLASS "B" UNLESS OTHERWISE NOTED.
3. PLASTIC SEWER PIPES SHALL BE CONSTRUCTED WITH ULTRA RIB OR APPROVED EQUAL UP TO THE MAXIMUM DIAMETER OF 600mm.
4. ALL WORKS SHALL BE CONSTRUCTED IN ACCORDANCE WITH CURRENT CITY OF MISSISSAUGA AND OPSD STANDARD DRAWINGS AND SPECIFICATIONS.
5. DOUBLE CATCHBASIN LEADS TO BE 300mm UNLESS OTHERWISE NOTED. ALL CATCHBASIN LEADS TO BE EITHER C-14-ES MINIMUM OR P.V.C. TYPE S.D.R. 28.
6. ALL BACKFILL FOR SEWERS, WATERMAINS AND UTILITIES ON PAVED AREAS MUST BE MECHANICALLY COMPACTED TO 95% STANDARD PROCTOR DENSITY.
7. INVERTS, ELEVATIONS AND EXACT LOCATIONS OF ALL EXISTING UNDERGROUND SERVICES TO BE VERIFIED IN THE FIELD BEFORE COMMENCING ANY WORK.
8. ALL AREAS DISTURBED DURING CONSTRUCTION TO BE RESTORED TO ORIGINAL CONDITION OR BETTER TO THE SATISFACTION OF THE CITY OF MISSISSAUGA AND REGION OF PEEL.
9. GRASSED AREAS TO BE TOPPED WITH 150mm TOPSOIL AND SODDED WITH No.1 NURSERY SOD.
10. THE CONTRACTOR SHALL BE RESPONSIBLE FOR CONTROLLING MUD AND DUST ON ALL PUBLIC ROADS TO THE SATISFACTION OF THE CITY AND REGION.

STORM WATER MANAGEMENT SUMMARY

MAXIMUM ALLOWABLE RELEASE RATE	63.7 L/sec
UNCONTROLLED RELEASE RATE	3.3 L/sec
MAXIMUM ALLOWABLE CONTROLLED RELEASE RATE	60.4 L/sec
ACTUAL RELEASE RATE FROM STM CTL. MH.	63.7 L/sec
TOTAL REQUIRED STORAGE	239.0 m^3
TOTAL AVAILABLE STORAGE AT SWM TANK	243.3 m^3
REQUIRED STORAGE FOR WATER BALANCE	40.0 m^3
AVAILABLE STORAGE FOR WATER BALANCE	40.0 m^3
100 YR. HWL AT ORIFICE	164.93 m
ORIFICE PLATE SIZE	172mm DIA.
STORM TANK MAXIMUM WATER DEPTH	3.08 m

PLUMBING NOTES:

1. ALL AD/CB/TRENCH DRAINS DESIGNED TO CAPTURE 100 YR STM. EVENT AND DIRECTED TO INTERNAL PLUMBING SYSTEM – PLUMBING TO BE DESIGNED BY MECHANICAL ENGINEER.
2. THE BUILDING SANITARY AND STORM SYSTEM MUST BE DESIGNED TO BE ABLE TO OPERATE UNDER MUNICIPAL SEWER SURCHARGE CONDITIONS.
3. ALL AREA DRAINS TO BE CONNECTED TO SWM TANK – PLUMBING TO BE DESIGNED BY MECHANICAL ENGINEER.
4. ROOF DRAINS TO BE CONNECTED TO RWH TANK – PLUMBING TO BE DESIGNED BY MECHANICAL ENGINEER.