1900 Appleby Line Mixed-Use Buildings Phase 1

Functional Servicing and Stormwater Management Report

Project Location:
1900 Appleby Line, Burlington, ON

Prepared for:
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28-1860 Appleby Line, Burlington, ON

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Engineers, Scientists, Surveyors.

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1.0 Introduction

1.1 Overview

MTE Consultants Inc. were retained by LJM Developments. to complete the site grading, servicing, and stormwater management design for the proposed mixed-used development located at 1900 Appleby Line in the City of Burlington (see Figure 1 for Location Plan). This report will outline a functional servicing and stormwater management strategy for the proposed development.

The site is located on a 2.1ha parcel of land bounded by Ironstone Drive to the north, Appleby Line to the east, commercial properties to the south, and Burlington Fire Department 8 to the west. This development has total three phases of the Uptown Center development. Phase 1 is located on the westerly portion of the site which has a parking lots and it will be developed as a 18 storeys mixed use building property and phase 2 & 3 is located in the existing commercial buildings and it will be developed as 16 storeys mixed use building property.

The property is currently an commercial building plaza and temporary parking lot in the north side of the site. The proponent plans to construct 3 mixed used buildings in 3 Phases. Each Phase will include the following:

- Phase 1: Building A, 18 storeys;
- Phase 2: Building B, 16 storeys; and,
- Phase 3: Building C, 16 storeys.

Commercial space will be provided within the main level of Buildings . Parking will be provided on site for the proposed buildings via two levels of underground (below grade) parking, and a few parking spaces will be available at grade for the commercial space along the southern site entrance.

A layby is proposed along the southern side of the building. adjacent to Building A to provide a drop off area for the proposed commercial space.

Existing municipal storm and sanitary sewers and watermain services are located on the abutting 1837 Ironstone Dr and right-of-way's (ROW) which will be utilized to service the proposed development.

The functional servicing described in this report will provide additional detailed information on the proposed servicing scheme for the site. Please refer to the site plan and the enclosed MTE drawings for additional information.

1.2 Background Information

The following documents were referenced in the preparation of this report:

- Ref 1: *Geotechnical Investigation Uptown Centre, Southwest Corner Appleby Line & Ironstone Drive*, Coffey Geotechnics, (February 10, 2009)
- Ref 2: Stormwater Management Report, Skira & Associated LTD (May 2010)
- Ref. 3: *Water and Wastewater Linear Design Manual (Version 4.0),* Regional Municipality of Halton (April 2019).
- Ref. 4: *Standard Drawing Manual*, City of Burlington Engineering Department (March 2014).

- Ref. 5: Ontario Building Code (2012).
- Ref. 6: *Low Impact Development Stormwater Management Planning and Design Guideline,* Credit Valley Conservation & Toronto and Region Conservation for the Living City, Version 1.0 (2010).
- Ref. 7: *Design Guidelines for Drinking-Water Systems*, Ministry of the Environment and Climate Change (2008).
- Ref. 8: *Erosion & Sediment Control Guideline for Urban Construction* (December, 2006).
- Ref. 9: *MOE Stormwater Management Practices Planning and Design Manual*, Ministry of Environment and Climate Change (March 2003).
- Ref. 10: *Water Supply for Public Fire Protection*, Fire Underwriters Survey (1999).
- Ref. 11: *Stormwater Management Design Guidelines*, City of Burlington (May 2020).

1.3 Geotechnical Investigation

A geotechnical investigation was undertaken by Coffey Geotechnics dated February 10, 2009 (Ref. 1). Eight (8) boreholes were advanced to various depths. Shale bedrock was encountered in BH3, BH5 and BH6 at depths varying between 1.5m and 2.3m below grade. All boreholes were dry after completion of drilling. Updated geotechnical recommendations will be required for the proposed development.

1.3.1 Hydrogeological Investigation

A hydrogeological investigation is underway to determine long term water levels. Once further information is obtained, impacts from short-term and long-term groundwater discharge will be quantified.

1.4 Site Grading

A topographic survey was performed by Barich Grenkie Surveying Ltd. dated March 31, 2017. The information indicates that the site is currently commercial buildings and parking lot. Commercial buildings and ground parking lot in the north of the site that will be demolished prior to the commencement of Phase 2 & 3 construction The northwest corner is the highest point with an overall grade difference on site of approximately 1.0m.

Under proposed conditions, site elevations will range from approx. 136.52m in the southwestern corner, to 137.30m along the northern exposed building foundation. The proposed phase 1 building first floor elevation (FFE) will be 136.70m and phase 2 and phase 3 FFE will be 137.02m and 137.26 each. The site is graded to self-contain drainage with the exceptions being the perimeter landscape which will drain uncontrolled off site (see Figure 3). The parking lot will we graded towards a series of area drains which will internally convey stormwater runoff to a proposed underground stormwater tank to be located within the underground parking structure.

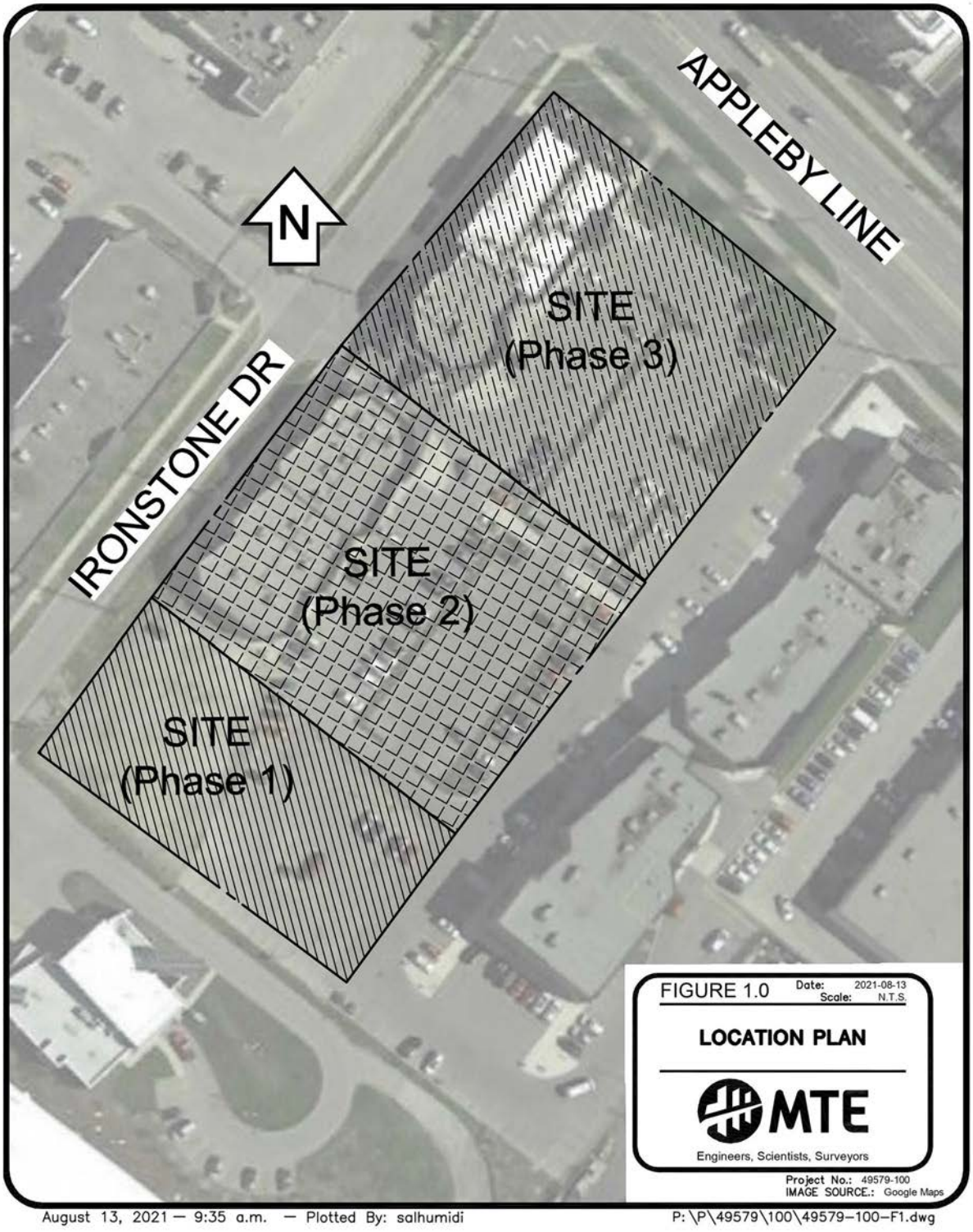


Figure 1: Location Plan

2.0 Stormwater Management

The following sections will describe the proposed stormwater management (SWM) plan for the proposed development.

2.1 Stormwater Management Criteria

The following stormwater management (SWM) criteria will be applied to the site:

2.1.1 Water Balance

Retain 5 mm of storm runoff volume on site through infiltration, evapotranspiration or rain water reuse for the increase in percent impervious from existing conditions.

2.1.2 Quantity Control

Attenuation of the post-development peak flows for the 2 through 100-year storm events to the 2-year to 100-year storm events existing conditions peak flow rate. Design Runoff Coefficients are provided by the City. The previously approved discharge rate for the site will be maintained or reduced. The previously approved discharge rate for the site is 0.210m3/s and is based on the capacity of the existing approved 450mm dia. outlet storm sewer outlet for the site.

2.1.3 Quality Control

An enhanced (Level 1) water quality treatment (80% TSS Removal) is required for all impacted surface runoff prior to discharging to the receiving system. In addition, the previously approved treatment level for the site will be maintained or enhanced.

2.2 Existing Conditions

Under existing conditions, the majority of the site is comprised of parking lots with commercial buildings and parking area. There is an existing 200mm, 300mm and 450mm diameter storm sewer, which flows southwest at 0.50 to 1.00% slope within the plaza. There is also an existing 750mm diameter storm sewer flowing west on Ironstone Drive, and a 300mm diameter storm sewer on Appleby Line, east of the site. The majority of the site is graded as parking lots for the commercial plaza. There is an existing underground structure on the phase 1 site is a 150mm diameter storm sewer stub which flows south at 2.0%. All existing storm sewers within the development of the site ultimately discharge to the existing 450mm diameter storm sewer at 0.50%, which outlets to the existing flood storage area located southwest of the site and connected to a storm pipe which goes along the fire station property . There is an STC 2000 Oil / Grit Separator on the southwest corner for water quality controls on site .

The existing conditions have been defined by one (1) catchment areas (see Table 2.1 and Figure 2). The existing percent impervious of the site was set at 79% (RC=0.75) based on comments from Stormwater Management Report (Ref. 2). A copy of the report can be found in Appendix A.

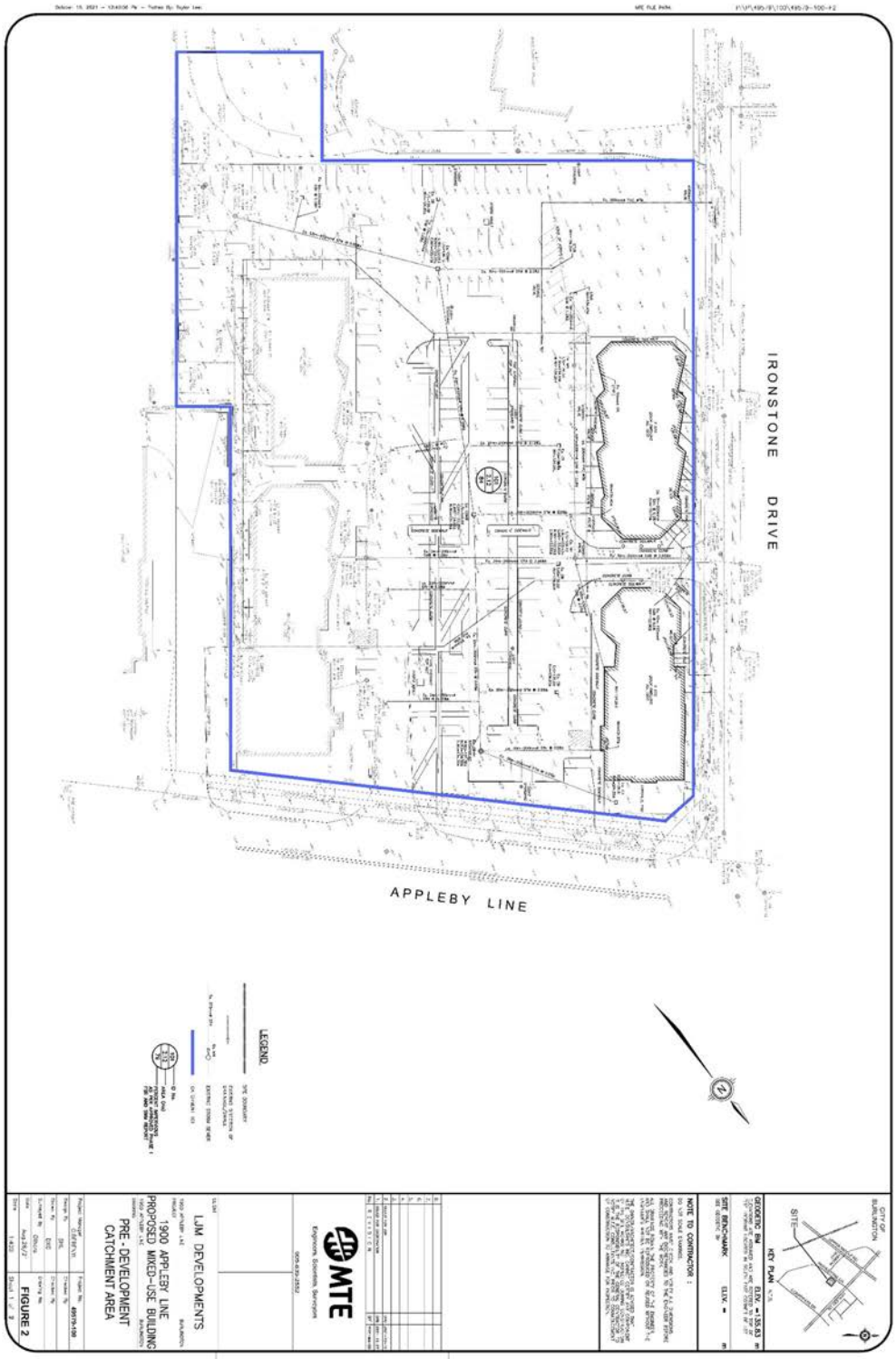


Figure 2: Existing Conditions Catchment Area

Table 2.1 - Existing Conditions Catchment Area

Catchment ID	Description	Area (ha)	% Imp. [A]
101	Existing Site	2.2	79
[A] As Provided by Skira & associates consulting SWM Report (Ref. 2).			

The existing conditions were based of the allowable release rate outlines by the Skira & associates consulting SWM Report (Ref. 2). The quantity controls for the existing site include surface storage and a 250mm dia orifice tube. Table 2.2 summarizes the allowable pre-development runoff rates.

Table 2.2 - Allowable Site Discharge

Catchment	Area (ha)	Allowable Site Discharge Rate (100-Year Storm)[A] (m^3/s)
101 (Existing Site)	2.2	0.210m3/s
[A] As Provided by Skira & associates consulting SWM Report (Ref. 2). Capacity of existing 450mm dia. storm sewer.		

2.3 Proposed Conditions

Under proposed conditions, the proponent plans to construct three new 16 to 18-storey mixed used buildings over three phases, with commercial space on the first and second floor. Surface and underground parking will be included for all buildings. The perimeter of the site will consist of walkways, landscaped and ground parking area.

The proposed conditions drainage pattern is delineated by two (2) catchment areas. Stormwater runoff from the site will be collected by a series of catch basins on the underground parking structure surface. These drains will connect to the internal plumbing of the building and therefore will be detailed by the mechanical consultant. The storm sewer outlet for the development will convey flows to the existing 450mm diameter located on Southwest corner of the site. Stormwater management controls in the form of an orifice and storm tank will be implemented to control post-development discharge rates to the allowable release rate for the site. Existing South commercial building areas' catch basins will be realigned and will be connected to a new manhole to be discharged into the existing OGS.

Table 2.3 provides a brief description of each catchment area as well as the size and impervious cover associated with each. Figure 3 provides an illustration of the post-development catchment areas. Appendix A contains detailed information pertaining to the stormwater management model.

Table 2.3 - Proposed Conditions Catchment Areas

Catchment ID	Description	Area (ha)	%Imp.
201	Proposed surrounding landscape (uncontrolled)	0.08	2
202	Proposed building area (Phase 1,2 & 3)	1.02	92
203	Proposed landscape area drains to existing commercial area	0.24	1
204	Existing Commercial buildings area	0.95	97
	Total	**2.2**	**78**

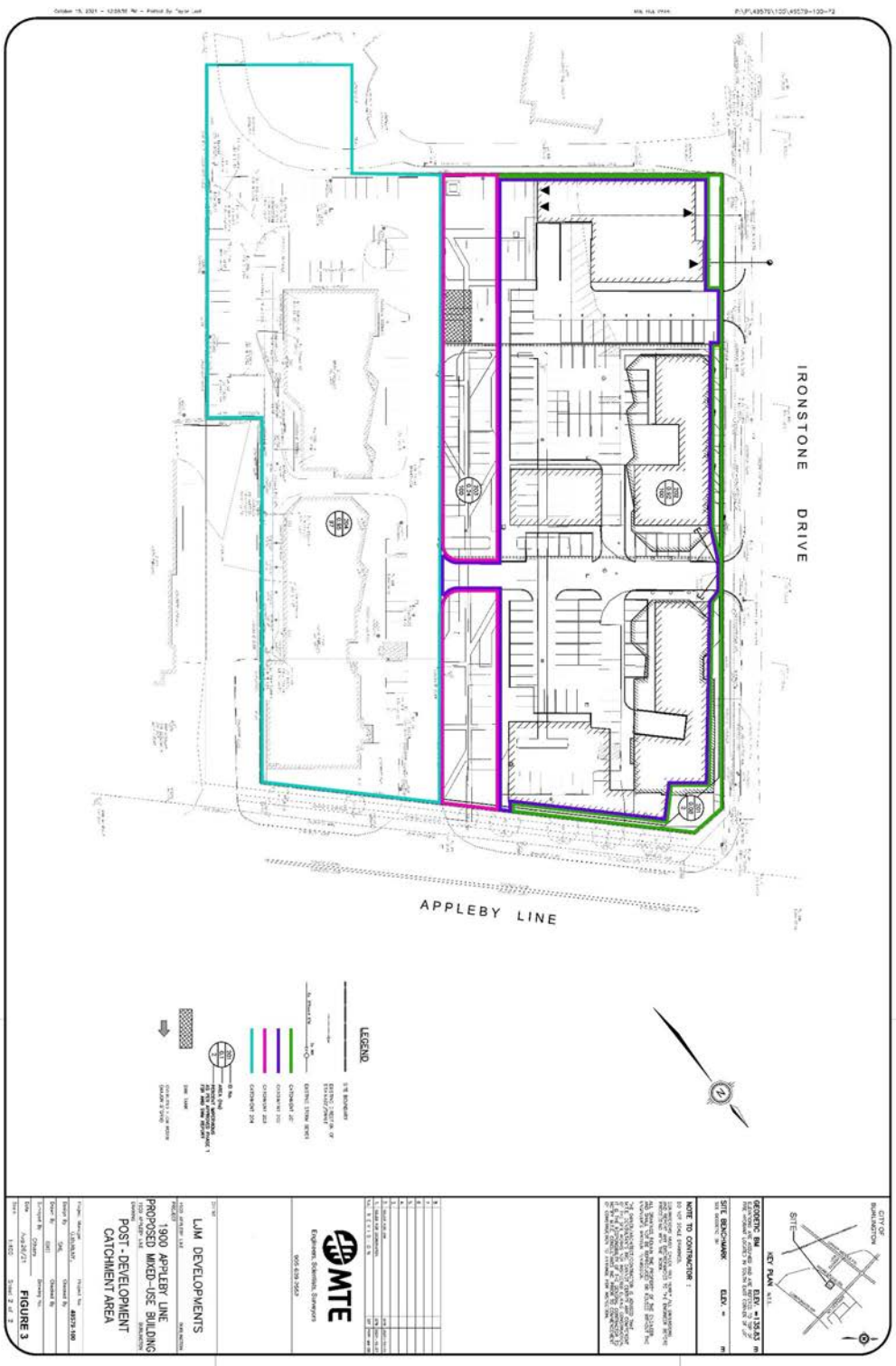


Figure 3: Proposed Conditions Catchment Areas

2.3.1 Water Balance

In order to meet the required stormwater retention of 5mm for the site (per Section 2.1), rain water harvesting will be implemented. The total require retention volume to be provided can be calculated as follows:

Retention Volume Required = Site Area x Increase in % Impervious x 5mm

= 22079 m^2 * 5mm

= 110.40m^3

Table 2.4 indicates the proposed infiltration and evaporation volumes for the site.

Table 2.4: Water Retention Summary

Surface Type	Area (m^2)	Initial Abstraction (mm)	Volume Achieved ($m^{\wedge 3}$)
Landscape Area	2361	5	11.81
Green Roof Area	2138	5	10.69
Conventional Roof	2138	1	2.14
Asphalt/Paved Areas	15442	1	15.44
Total	**22079**		**40.08**

Based on the above, additional water re-use measures such as using harvested rain water for landscape irrigation, process water in cooling towers, or for non-potable water uses will need to be considered at the detailed design stage and coordinated with the mechanical consultant. Approximately 70.32m^3 of additional stormwater re-use will be required.

2.3.2 Quantity Control

Stormwater management quantity controls for the site will be provided with an underground storage tank coupled with an orifice control located within the Phase 1 underground parking structure. The tank will form part of the underground parking structure and will require structural design and waterproofing (to be designed by others at the building permit stage). Backwater valves are to be installed within the building as indicated on drawing C1.3&1.4 for the proposed storm and sanitary services. Table 2.4 summarizes the stage-storage-discharge relationship of the proposed storm tank and orifice control. The full calculations can be found in Appendix B. This information is used in the hydrologic model.

Table 2.4 - Stage-Storage-Discharge Calculations for Underground Storm Tank

Elevation (m)	Head, H (m)	Cumulative Storage Volume (m^3)[A]	Discharge Q (m^3/s)[B]	Comments
133.98	0.00	0.0	0.0000	Inside Bottom of Tank
134.09	0.00	26.1	0.0000	C/L of orifice
134.52	0.43	125.3	0.0725	1/3 Storm Tank
135.06	0.97	250.6	0.1091	2/3 Storm Tank
135.20	1.10	281.9	0.1165	75% Storm Tank
135.60	1.51	375.8	0.1362	Top of Tank

[A] Storage volume based on a tank with internal footprint of 232m2 and an internal height of 1.51m for an overall internal storage volume of 375.8m3. See Appendix A and drawing C1.2 for more details.
[B] From orifice equation Q = CA (2gH)0.5 for a 226 mm diameter orifice
Where: C = 0.62, A=cross-sectional area, g=9.81, H=pressure head

The proposed conditions were assessed for the 5 year and 100-year City of Burlington 4-hour Chicago Distribution design storms. Appendix A contains detailed modeling parameters and input/output printouts for the proposed conditions. Table 2.5 summarizes the results.

Table 2.5 – Maximum Site Discharge and Storage

Storm Event	Cumulative Storage Volume (m^3)	Discharge Q (m^3/s)
5 year	71	0.210
100 year	353	0.210

The analysis indicates the following:

- For the 5-year to 100-year events, the total proposed conditions peak discharge rates from the site do not exceed the allowable release rate as illustrated in Table 2.5. This satisfies the stormwater management quantity control requirement set by the City of Burlington.
- Sufficient storage volume is provided within the storm tank and existing storm system to contain stormwater as illustrated within Table 2.4 and Appendix A.
- Overland flow will be contained on site to the allowable release rate for the site, as outlined in the approved and SWM report (Ref. 2).

2.3.3 Water Quality Control

Water quality control for the proposed development will be provided by the existing a Stormceptor 2000 oil/grit separator. A copy of the OGS sizing report is located in Appendix A. This OGS unit is located at the downstream end of the stormwater management system prior to connecting into the existing 450mm diameter storm sewer within the next site. The following parameters were used to size the oil/grit separator:

- Upstream Catchment Area = 2.14ha;
- % Impervious = 84%;
- Particle Distribution = FINE ; and
- Upstream storage = none

The sizing report indicates that the STC 2000 will provide 72% TSS Removal which meets the requirements for an “Normal” (Level 2 or 70% TSS removal) level of water quality protection.

Additional on-site controls will be required to achieve “Enhanced” (Level 1 or 80% TSS removal) level of water quality protection. Additional controls will include CB sediment shields and LID techniques in landscaped areas where possible.

In some localized areas around the building perimeter, stormwater runoff will flow overland uncontrolled to the abutting ROWs. Since these areas are comprised of walkways and landscaped areas, stormwater runoff is generally considered to be clean and therefore no water quality controls will be provided for these areas.

2.3.4 Private Storm Service Connection

Phase 1

Existing storm sewer pipes within Phase 1 will be removed to construct the proposed underground parking structure. The rest of the existing storm sewer pipes north of the existing commercial buildings on the south side of the property will be connected to a new 200mm diameter storm sewer at a 1%. Three proposed MH will be added and will flow from MH3 to MH2 to MH1. The proposed 430 cubic meter storm tank will also flow through a 4m 200mm pipe at a 0.5% slope into MH1. MH1 will then flow into the exciting STM MH#1 through a 46.2m 450mm pipe at a 0.8% slope with an existing 250mm orifice. There will also be a proposed catch basin located above the underground parking lot and will be internal routed into the stormwater tank. The existing 450mm diameter storm sewer has a full flow capacity of approximately 0.210m^3/s which is greater than the 100-year controlled peak discharge of 0.136 m^3/s from the tank orifice (see Table 2.4). Therefore, the proposed storm lateral will have sufficient capacity to convey the proposed 100-year controlled peak flow from the site. The proposed OGS and stormwater tank are designed for the ultimate buildout conditions and will be constructed during the first phase of construction. Therefore, the interim conditions are addressed. Please refer to Drawing C2.2 for further site servicing details.

Ultimate Phase

All the existing storm sewer pipes and structures which are in conflict with the proposed ultimate phase will be removed and storm water will be captured in a proposed storm water tank in the phase 1. Existing catch basins in front of the existing commercial building will be connected to a new pipe run. Along this new pipe run two new MH will be added and connected to the proposed MH1. The storm water will also be captured by two catch basing located above the underground parking lot and will be routed within the building into the storm water tank.

2.4 Sediment and Erosion Control

Sediment and erosion control measures will be implemented on site during construction and will conform to the Erosion & Sediment Control Guideline for Urban Construction (Ref. 8) and City of Burlington Standards.

Sediment and erosion control measures will include:

- Installation of silt control fencing at strategic locations around the perimeter of the site where feasible.
- Preventing silt or sediment laden water from entering inlets (catch basins / catch basin manholes) by wrapping their tops with filter fabric or installing silt sacks.
- Construction of 6m x 12m mud mat at the exit from the site to Ironstone Drive. to mitigate the transportation of sediments to the surrounding roads.
- Maintaining sediment and erosion control structures in good repair (including periodic cleaning as required) until such time that the Engineer or City of Burlington approves their removal. Erosion control measures to be inspected daily and after any rainfall event.

3.0 Sanitary Sewer Servicing

3.1 Existing Conditions

Based on plan & profile drawings provided by the City of Burlington ('the City') and Halton Region ('the Region'), the site has servicing available via an existing 450mm diameter mainline sewer in Ironstone Drive. Table 3.1 summarizes the existing sanitary sewers.

Table 3.1 – Summary of Mainline Sanitary Sewers fronting the Site

Street	Direction of Flow	Diameter	Slope	Capacity [A]
Ironstone Drive	West	450 mm	0.50%	201.5 L/s
[A] Full flow capacity calculated using Manning's Formula				

3.2 Sanitary Demands

The anticipated sanitary discharge from the proposed development was estimated using Halton Region design criteria and the Ontario Building Code (2012) based on the proposed building use. Table 3.2 provides an estimate of the total residential population and the number of units for phase 1 and Table 3.3 presents all phases. For a population breakdown by phase refer to Appendix. B.

Table 3.2 - Population Estimate – Phase 1

Unit Types	Total Number of Units [A]	People per unit [B]	Population (people) [C]
Studio	16	2	32
1 Bedroom	15	2	30
1 Bedroom + Den	77	3	231
2 Bedroom	32	4	128
2 Bedroom + Den	30	5	150
		Total Estimated Population	**571**

[A] Unit Count taken from Site plan by Core Architecture received August 17, 2021
[B] Population density based on OBC Occupancy Loads Section 3.1.7.17.1. clause 1b)
(2 persons per bedroom). Population Densities for 1 Bedroom + Den was estimated to be 3 persons per unit due to possibility of den being used at small bedroom.
[C] Population calculated as (Total # of Units) X (Persons per Unit)

Table 3.3 - Population Estimate – All Phases

Unit Types	Total Number of Units [A]	People per unit [B]	Population (people) [C]
Studio	43	2	86
1 Bedroom	171	2	342
1 Bedroom + Den	149	3	447
2 Bedroom	119	4	476
2 Bedroom + Den	78	5	390
		Total Estimated Population	**1741**

[A] Unit Count taken from Site plan by Core Architecture received August 17, 2021
[B] Population density based on OBC Occupancy Loads Section 3.1.7.17.1. clause 1b)
(2 persons per bedroom). Population Densities for 1 Bedroom + Den was estimated to be 3 persons per unit due to possibility of den being used at small bedroom.
[C] Population calculated as (Total # of Units) X (Persons per Unit)

In addition to the residential population of the buildings, the development will include a commercial area. To determine sanitary discharge rates from this use, the commercial floor area was used along with the Region of Halton Design Standards (Ref. 3). The sanitary sewer discharge rates from the development are summarized in Table 3.4 and detailed calculations and a breakdown by phase, are found in Appendix C.

Table 3.4 - Sanitary Sewer Discharge from Site – Phase 1

Land Use	Population (people) / Commercial Area (ha)	Average Flow (L/s)	Peak Flow (L/s)
Proposed Residential	571 [A]	1.82 [B]	7.17 [C]
Proposed Commercial Space	0.226	0.065 [D]	0.26 [C]
Ex. Commercial Space		1.27	5.09
		Total Peak Sanitary Demand for Site	12.52 [E]
Total Peak Sanitary Demand for Site (with infiltration allowance)			**12.61 [F]**

[A] Population Estimate: see Table 3.2
[B] Average flow based on 275 L/ca/day. Avg Flow = 275*571/(24*60*60) = 1.82 L/s
[C] Peak flow = Average Flow*PF, where Harmon Peaking Factor (PF) = 1+(14/(4+(P/1000)½)) = 1 + (14/(4+(571/1000)½)) = 3.94 (max 5.0)
[D] Average flow = (0.29 L/s/ha)*(0.226 ha floor space) = 0.065 L/s
[E] Total Peak flow = Peak flow from Residential + Commercial
[F] Total Peak flow with infiltration = Total Peak flow + infiltration allowance = 7.43+ 0.094 = 7.52 L/s
Where infiltration is based on 0.286 l/s/ha. Area reflects site area (0.33 ha), I = 0.286*0.33 = 0.094 L/s

Table 3.5 - Sanitary Sewer Discharge from Site – Ultimate Phase

Land Use	Population (people) / Commercial Area (ha)	Average Flow (L/s)	Peak Flow (L/s)
Proposed Residential	1741 [A]	5.54 [B]	21.83 [C]
Proposed Commercial Space	0.393	0.112 [D]	0.45 [C]
Ex. Commercial Space		1.01	4.03
		Total Peak Sanitary Demand for Site	26.31 [E]
Total Peak Sanitary Demand for Site (with infiltration allowance)			**26.67 [F]**

[A] Population Estimate: see Table 3.3
[B] Average flow based on 275 L/ca/day. Avg Flow = 275*1741/(24*60*60) = 5.54 L/s
[C] Peak flow = Average Flow*PF, where Harmon Peaking Factor (PF) = 1+(14/(4+(P/1000)½)) = 1 + (14/(4+(1741/1000)½)) = 3.63 (max 5.0)
[D] Average flow = (0.29 L/s/ha)*(0.393 ha floor space) = 0.112 L/s
[E] Total Peak flow = Peak flow from Residential + Commercial
[F] Total Peak flow with infiltration = Total Peak flow + infiltration allowance = 22.28+ 0.63 = 22.91 L/s
Where infiltration is based on 0.286 l/s/ha. Area reflects site area (0. ha), I = 0.286*2.2 = 0.63 L/s

3.3 Proposed Sanitary Servicing Plan and Capacity Analysis

As calculated in Table 3.3 & 3.4, the total peak sanitary discharge for phase 1 from the site is 7.52 L/s and all phases is 22.64 L/s. The proposed buildings will be serviced in three phases by three 200mm diameter sanitary services at 2.0% & 6.0% slope in each building (full flow capacity = 25.1 L/s (2%), 43.5L/s (6%)) that will connect to the Ironstone Drive. ROW's. sanitary sewer (see Drawing C1.3 & 1.4). Table 3.6. summarizes the sanitary sewers and their proposed loads from the site.

Table 3.6 – Summary of Proposed Loading on Mainline Sewers

SAN Sewer	Full Flow Capacity (L/s) [A]	Peak Load from Site (L/s) [B]	Percent Sewer Capacity Demand
1900 Appleby line–Phase 1	267.6	7.29	48.6%
1900 Appleby line– Ultimate Phase	267.6	22.64	53.9%

[A] Full flow capacity calculated using Manning's Formula
[B] Peak load from each phase calculated in Appendix B.

3.4 External Sanitary Sewer - Area Servicing Plan

A downstream sanitary sewer capacity analysis was conducted to assess the hydraulic capacity of the receiving municipal sewers based on the Region of Halton Water and Wastewater Linear Design Manual (Ref. 3). Based on sanitary drainage area plans and plan and profile drawings provided by the City of Burlington, this development falls within that drainage area. The sanitary drainage plan is delineated on MTE drawings SA1.1 and SA1.2 (see Appendix B).

The results of the analysis indicate that under proposed conditions, there is a surcharge in the existing downstream municipal sanitary sewer system. Based off of Halton Region Design Guidelines (Ref. 3), existing pipes less than or equal to 450mm diameters are required to be upsized if the pipe reaches 70% full. For proposed sewers pipes should be designed to no more than 60% full. Table 3.7 summarizes the impact of the proposed development on the existing sanitary sewer infrastructure. (See Appendix B for sanitary design sheets and catchment area figure)

Table 3.7 – Summary of Impacts of Proposed Development on Existing Sanitary Sewer Infrastructure on Ironstone Drive

Scenario	Ex. Diameter	Ex. Capacity [A]	Proposed Peak Flow [A]	Pipe Full	Upgrade Required?
Existing Conditions	450 mm	214.40 L/s	139.87 L/s	65.2%	No
Existing Conditions + Proposed Phase 1 Development			153.65 L/s	71.7%	Yes
Existing Conditions + Proposed Ultimate Phase Development			171.10	79.8%	Yes
[A] Full flow capacity calculated using Manning's Formula at worst case scenario sewer leg (from ex. MH 10 to ex. MH 11					

The above Table 3.7 indicates that the existing 450mm diameter existing sanitary sewer pipe on Ironstone drive is not sufficiently sized to accommodate Phase 1 development. Table 3.8 summarized the required sanitary sewer upgrades to accommodate the development during Phase 1 and Ultimate Phase.

Table 3.8 – Summary of Required Upgrades on Existing Sanitary Sewer Infrastructure on Ironstone Drive for Proposed Development

Scenario	Required Upgrades
Existing Conditions + Proposed Phase 1 Development	309.50m of 450mm diameter sewer to be upsized to 525mm diameter between MH5 and MH12
Existing Conditions + Proposed Ultimate Phase Development	328.3m of 450mm diameter sewer to be upsized to 525mm diameter between MH3 and MH12

4.0 Domestic and Fire Water Supply Servicing

4.1 Existing Conditions

The existing municipal water distribution system around the site consists of a 300mm diameter watermain within the Ironstone Drive. The proposed development is located in the B2 pressure zone. This pressure zone comprises the central portion of the City of Burlington, between Upper Middle Road and the QEW/ Fairview Street . The Washburn and Mount Forest Pumping Stations supply water to the Burlington B2 Pressure Zone.

4.2 Domestic Water Demands

The expected domestic water demands for the proposed development were estimated using Halton Region design criteria (Ref. 3). Table 4.1 summarizes the domestic water demand requirements for the Average Day, Maximum Day and Peak Hour demand scenarios and detailed calculations are provided in Appendix C. It should be noted that average day peak factor is 1.0, the max day peak factor is 2.25 and the peak hour factor is 4.0 for residential developments, and the peak hour factor is 2.0 for commercial developments, as in accordance with Halton Region standards. A summary of the demands for all phases are summarized in Table 4.1 below. Refer to Appendix C for water demands broken down by phase.

Table 4.1 - Domestic Water Demands

Residential Demands		
Population:	1741 people (see Table 3.1)	
Average Day Demand:	275 L/c/d x 1741 people =	5.54 L/s
Maximum Day Demand:	2.25 x 5.54 L/s =	12.47 L/s
Peak Hour Demand:	4.0 x 5.54 L/s =	22.16 L/s
Commercial Demands		
Floor area:	0.393 ha (see Appendix C)	
Average Day Demand:	0.29 L/ha/d x 0.393 ha =	0.113 L/s
Maximum Day Demand:	2.25 x 0.112 L/s =	0.254 L/s
Peak Hour Demand:	2.0 x 0.112 L/s =	0.254 L/s
Total Residential + Commercial Demands		**22.42 L/s**

4.3 Fire Flow Demands

Fire flow demands for the proposed development were also determined using the methodology outlined in Water Supply for Public Fire Protection (Ref. 10). The fire flows for each building were evaluated. The fire demands are summarized in Table 4.2 and detailed calculations are provided in Appendix C.

Table 4.2 - FUS Fire Flow Requirements

Building	Fire Underwriters Survey (FUS) Flow Rate
Phase 1- Building A	133 L/s (7,650 L/min)
Phase 2 - Building B	100 L/s (6,120 L/min)
Phase 3- Building C	83 L/s (4,590 L/min)

4.4 Proposed Water Servicing Plan and Analysis

Water servicing for the site will include the installation of a new water service for Phase 1. It should be noted that the existing water system is looped and services the existing plaza. At all times during construction, a detailed watermain staging plan will be need developed to maintain adequate water supply for both domestic and fire fighting purposes. The water servicing plan proposes to maintain two connections to the existing watermain on Ironstone Drive. Further discussion with the City and Region with respect to easements will be required.

The site will be serviced by the existing municipal hydrants located on the south side of Ironstone Drive, north of the proposed development. The proposed developments fire department connection will be within 45m of the fire hydrant located southeast of the proposed development. Existing on-site fire hydrants will be relocated to provide adequate water supply for fire fighting purposes.

A fire hydrant flow test will be required to confirm that there is adequate system pressure and flow. Based on a worst case Maximum Day + Fire Flow Demand of 137.57 L/s (133.33 L/s + 4.09 L/s + 0.15 from Appendix C and 4.2).

5.0 Conclusions and Recommendations

Based on the information provided herein, it is concluded that the development can be constructed to meet the requirements of the City of Burlington and Halton Region. Therefore, it is recommended that:

i. A stormwater storage tank complete with an orifice control to be provided to control proposed conditions stormwater site discharge rates to the allowable release rate as described in Section 2.3 of this report.

ii. Erosion and sediment controls be installed as described in Section 2.4 of this report.

iii. Sanitary servicing for the development and required external sanitary upgrades be advanced as described in Section 3.3 of this report.

iv. Water servicing for the development be installed as described in Section 4.4 of this report to meet OBC minimum water supply requirements.

v. The proposed stormwater management plan presented in this report and the site servicing works described in this report and as shown on Drawings C1.1, C2.1, C2.2, C2.3, C2.4, C2.5, C2.6 and C2.7 be accepted in support of the zoning approval process.

We trust the information enclosed herein is satisfactory. Should you have any questions please do not hesitate to contact our office.

All of which is respectfully submitted,

MTE Consultants Inc.



George Berenyi, P.Eng.
Project Manager
905-639-2552 ext. 2461
GBerenyi@mte85.com

Appendix A

Stormwater Management Calculations

SKIRA & ASSOCIATES LTD.
CONSULTING ENGINEERS

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Zenon Skira
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3464 Semenyk Court, Suite 100,
Mississauga, Ontario L5C 4P8
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info@skiraconsult.ca

May 17, 2010 File No. 205-BU168

RE: Stormwater Management Report
Proposed Commercial/Office Plaza Development
Ironstone/Appleby Line
City of Burlington

1.0 INTRODUCTION

The purpose of this report is to provide a suitable storm sewer solution for the proposed development site in order to comply with the City of Burlington requirements.

- An on-site stormwater management will be implemented in order to ensure that the **100 year** post development flows will not exceed the allowable release rates of the **5 year** storm with a runoff coefficient **C = existing conditions** (0.75).
- Maximum required storage volumes for the site was arrived at using the Modified Rational Method for the **100 year** storm event.
- The site is vacant subdivision block.

2.0 SITE DESCRIPTION

Site Area	=	21,417.0 Ha
Roof Area	=	5447.0 Ha
Landscaped Area	=	4282.0 Ha
Paved Area	=	11,688.0 Ha

3.0 SITE RETENTION

Therefore the maximum allowable discharge from the entire site will be as follows:

A =	2.1417 ha		
C =	0.75	Q_{5yr} =	CIA/360
Tc =	10.0 min	=	2.1417 x 0.75 x 88.09/360
I_{5yr} =	88.09 mm/hr	=	**0.393 m^3/s**

4.0 PROPOSED POST DEVELOPMENT CONDITIONS

Maximum allowable discharge from the subject property should not exceed **0.393 m^3/s** .

Existing storm sewer capacity 450mm diameter pipe @ 0.50% flow is only 0.210 m^3/s, therefore site will be restricted to 210 l/s flow.

3.0 SITE RETENTION - POST DEVELOPMENT CONDITIONS

3.1 Roof Storage - Not permitted

PAVED & LANDSCAPED AREA

Weighted C = (0.25 x 0.4282 + 0.90 x 1.1688 + 0.95 x 0.5447/2.1417

= 0.05 + 0.49 + 0.24

= **0.78**

YEAR STORM
100 YEAR
CITY
BURLINGTON

C = 0.780
A (ha) = 2.14170
Allow. Discharge Qa (m3/s) = 0.210000
Safety Factor Sf = 0%

Max. Required Detention (m3) = *293.17*

RAINFALL DURATION *Tc (min)*	RAINFALL INTENSITY *I (mm/hr)*	TOTAL UNCONTROLLED RUNOFF *Q=CIA/360 (m3/sec)*	INFLOW VOLUME Vi (m3)	OUTFLOW VOLUME Vo (m3)	REQUIRED DETENTION VOLUME (m3) *D=(Vi-Vo)*Sf*
5	193.18	0.8964	268.93	68.33	200.59
10	141.89	0.6584	395.06	126.00	269.06
15	113.99	0.5290	476.07	184.08	291.99
20	96.19	0.4464	535.63	242.45	**293.17**
25	83.73	0.3885	582.80	301.04	281.75
30	74.46	0.3455	621.94	359.80	262.14
35	67.27	0.3121	655.49	418.71	236.78
40	61.50	0.2854	684.91	477.73	207.17
45	56.76	0.2634	711.15	536.86	174.29
50	52.79	0.2450	734.88	596.08	138.80
55	49.41	0.2293	756.58	655.39	101.19
60	46.49	0.2157	776.59	714.76	61.82

3.2 Stormwater Detention Volumes

CATCHBASIN NO.	CATCHBASIN TOP ELEV. (m)	PONDING ELEVATION 100 YR (m)	PONDING DEPTH 100 YR (m)	STORAGE PROVIDED 100 YR (m^3)	STORAGE REQUIRED 100 YR (m^3)
4	136.05	136.25	0.20	32.0	
5	136.05	136.25	0.20	38.0	
6	136.05	136.25	0.20	13.0	
7	136.05	136.25	0.20	9.6	
8	136.05	136.25	0.20	41.8	
10	136.05	136.25	0.20	23.5	
11	136.05	136.25	0.20	16.0	
12	136.10	136.25	0.15	10.5	
13	136.15	136.25	0.10	13.0	
14	136.10	136.25	0.15	15.0	
15	136.10	136.25	0.15	18.0	
16	136.05	136.25	0.20	39.0	
17	136.05	136.25	0.20	27.0	
			TOTAL:	**296.40**	

3.3 Orifice & Weir Control

The maximum allowable discharge from the site will be controlled by means of an orifice restrictor tube installed between **STMH2** and the oil/grit separator. The size of the orifice restrictor tube is 250mm.

The orifice discharge rate was calculated by using FlowMaster computer program developed by Haestad Methods Inc. (USA) and an output report is attached.

ORIFICE DEPTH/DISCHARGE

Area	Ponding Elevation m	Orifice Centre Line Elevation m	Head on Orifice m	Discharge m^3/s	
(100 yr storm)	136.25	133.06	3.19	0.233	Orifice

4.0 STORMWATER QUALITY CONSIDERATIONS

Acceptable methods for stormwater treatment vary with size of the development, soil types and nature of proposed land use. All these must be balanced with the physical constrains imposed by the development site.

In our selection, we followed the guidelines and methods described in "Stormwater Management Practices Planning and Design Manual" prepared for M.O.E.. (March 2003).

Taking in consideration the size of the proposed development **(A = 2.14 ha)**, we have reviewed and selected the most suitable stormwater management practices (SWMP) to treat the stormwater runoff before it is released to the existing storm sewer system.

Oil/Grit Separators (O.G.S.)

Normally these facilities operate based on the principle of sedimentation of the grit and phase separation for the oil. They are most suitable for institutional/commercial/industrial areas where the level of concentrated pollutants is expected to be higher than residential areas. Being a residential development it is considered feasible to provide an oil/grit separator (O.G.S.) on the storm sewer line.

The stormwater runoff from the site will be intercepted and conveyed through the O.G.S. prior to being discharged to the existing storm sewer system along existing easement.

The proposed oil/grit separator is Type STC 2000 manufactured by Stormceptor. Attached to this report is an output file created by Stormceptor software.

This unit will provide Level 2 protection.

We trust that the above information will be satisfactory to you and if you have any comments and/or approval, please do not hesitate to contact our office.

Yours very truly,

SKIRA & ASSOCIATES LTD.



Michael Jozwik, P. Eng.
MJ:kb

Worksheet
Worksheet for Circular Orifice

Project Description	
Worksheet	Orifice - 1
Type	Circular Orific
Solve For	Discharge

Input Data		
Headwater Elevat	36.25	m
Centroid Elevatior	33.06	m
Tailwater Elevatio	32.78	m
Discharge Coeffic	0.60	
Diameter	250	mn

Results		
Discharge	ı.2330	m³/s
Headwater Height Above	3.19	m
Tailwater Height Above C	-0.28	m
Flow Area	1.9e-2	m²
Velocity	4.75	m/s

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SKIRA & ASSOCIATES LTD.
37 Brookside Road Waterbury, CT 06708 USA (203) 755-1666

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Stormceptor Sizing Detailed Report

PCSWMM for Stormceptor

Project Information

Date	6/25/2009
Project Name	Appleby Plaza
Project Number	205-BU168
Location	Burlington

Stormwater Quality Objective

This report outlines how Stormceptor System can achieve a defined water quality objective through the removal of total suspended solids (TSS). Attached to this report is the Stormceptor Sizing Summary.

Stormceptor System Recommendation

The Stormceptor System model STC 2000 achieves the water quality objective removing 72% TSS for a Fine (organics, silts and sand) particle size distribution.

The Stormceptor System

The Stormceptor oil and sediment separator is sized to treat stormwater runoff by removing pollutants through gravity separation and flotation. Stormceptor's patented design generates positive TSS removal for all rainfall events, including large storms. Significant levels of pollutants such as heavy metals, free oils and nutrients are prevented from entering natural water resources and the re-suspension of previously captured sediment (scour) does not occur.

Stormceptor provides a high level of TSS removal for small frequent storm events that represent the majority of annual rainfall volume and pollutant load. Positive treatment continues for large infrequent events, however, such events have little impact on the average annual TSS removal as they represent a small percentage of the total runoff volume and pollutant load.

Stormceptor is the only oil and sediment separator on the market sized to remove TSS for a wide range of particle sizes, including fine sediments (clays and silts), that are often overlooked in the design of other stormwater treatment devices.

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Small storms dominate hydrologic activity, US EPA reports

"Early efforts in stormwater management focused on flood events ranging from the 2-yr to the 100-yr storm. Increasingly stormwater professionals have come to realize that small storms (i.e. < 1 in. rainfall) dominate watershed hydrologic parameters typically associated with water quality management issues and BMP design. These small storms are responsible for most annual urban runoff and groundwater recharge. Likewise, with the exception of eroded sediment, they are responsible for most pollutant washoff from urban surfaces. Therefore, the small storms are of most concern for the stormwater management objectives of ground water recharge, water quality resource protection and thermal impacts control."

"Most rainfall events are much smaller than design storms used for urban drainage models. In any given area, most frequently recurrent rainfall events are small (less than 1 in. of daily rainfall)."

"Continuous simulation offers possibilities for designing and managing BMPs on an individual site-by-site basis that are not provided by other widely used simpler analysis methods. Therefore its application and use should be encouraged."

– US EPA Stormwater Best Management Practice Design Guide, Volume 1 – General Considerations, 2004

Design Methodology

Each Stormceptor system is sized using PCSWMM for Stormceptor, a continuous simulation model based on US EPA SWMM. The program calculates hydrology from up-to-date local historical rainfall data and specified site parameters. With US EPA SWMM's precision, every Stormceptor unit is designed to achieve a defined water quality objective.

The TSS removal data presented follows US EPA guidelines to reduce the average annual TSS load. Stormceptor's unit process for TSS removal is settling. The settling model calculates TSS removal by analyzing (summary of analysis presented in Appendix 2):

- Site parameters
- Continuous historical rainfall, including duration, distribution, peaks (Figure 1)
- Interevent periods
- Particle size distribution
- Particle settling velocities (Stokes Law, corrected for drag)
- TSS load (Figure 2)
- Detention time of the system

The Stormceptor System maintains continuous positive TSS removal for all influent flow rates. Figure 3 illustrates the continuous treatment by Stormceptor throughout the full range of storm events analyzed. It is clear that large events do not significantly impact the average annual TSS removal. There is no decline in cumulative TSS removal, indicating scour does not occur as the flow rate increases.

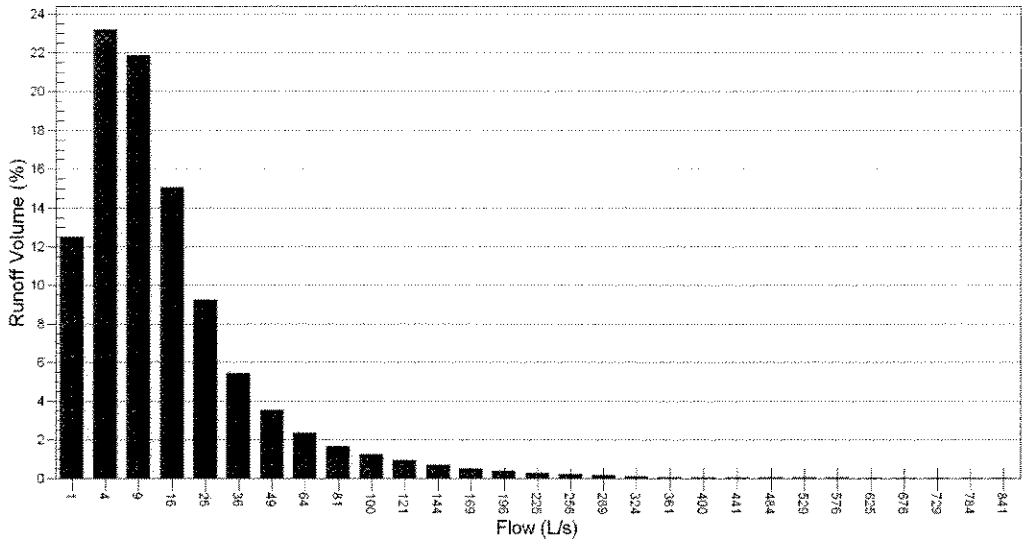


Figure 1. Runoff Volume by Flow Rate for TORONTO CENTRAL – ON 100, 1982 to 1999 for 2.14 ha, 84% impervious. Small frequent storm events represent the majority of annual rainfall volume. Large infrequent events have little impact on the average annual TSS removal, as they represent a small percentage of the total annual volume of runoff.

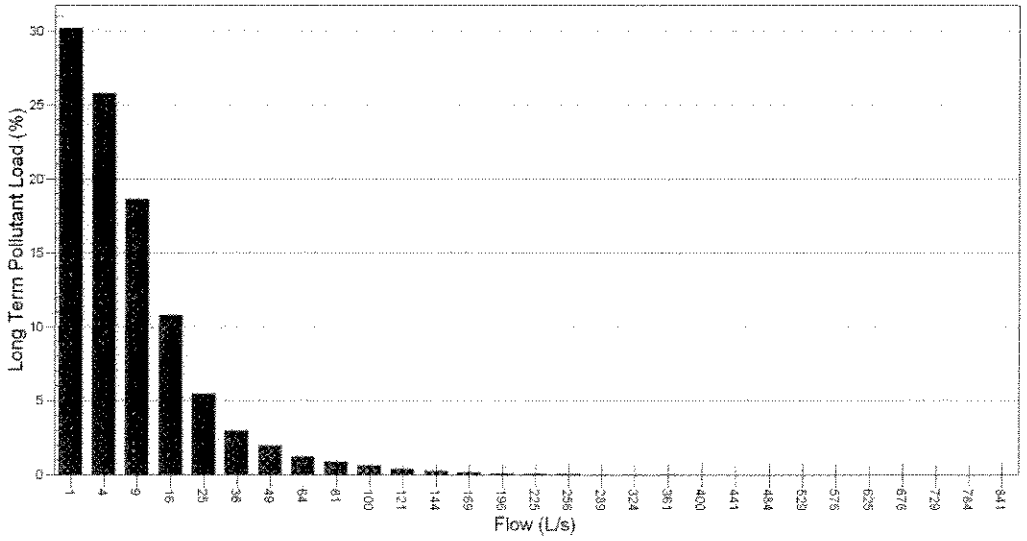
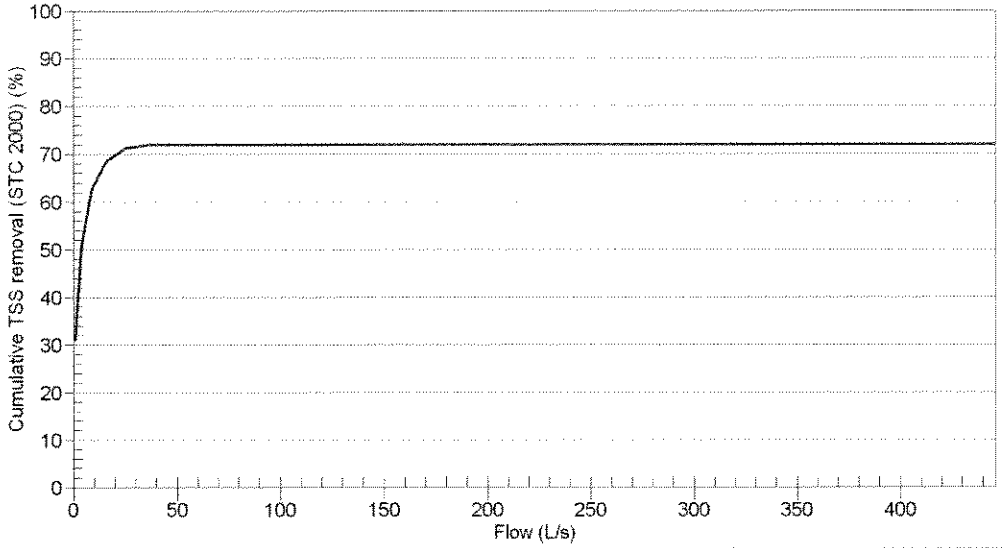


Figure 2. Long Term Pollutant Load by Flow Rate for TORONTO CENTRAL – 100, 1982 to 1999 for 2.14 ha, 84% impervious. The majority of the annual pollutant load is transported by small frequent storm events. Conversely, large infrequent events carry an insignificant percentage of the total annual pollutant load.



Stormceptor Model	STC 2000	Drainage Area (ha)	2.14
TSS Removal (%)	72	Impervious (%)	84

Figure 3. Cumulative TSS Removal by Flow Rate for TORONTO CENTRAL – 100, 1982 to 1999. Stormceptor continuously removes TSS throughout the full range of storm events analyzed. Note that large events do not significantly impact the average annual TSS removal. Therefore no decline in cumulative TSS removal indicates scour does not occur as the flow rate increases.

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Appendix 1
Stormceptor Design Summary

Project Information

Date	6/25/2009
Project Name	Appleby Plaza
Project Number	205-BU168
Location	Burlington

Designer Information

Company	Skira & Associates Ltd
Contact	Michael Jozwik

Notes

N/A

Drainage Area

Total Area (ha)	2.14
Imperviousness (%)	84

The Stormceptor System model STC 2000 achieves the water quality objective removing 72% TSS for a Fine (organics, silts and sand) particle size distribution.

Rainfall

Name	TORONTO CENTRAL
State	ON
ID	100
Years of Records	1982 to 1999
Latitude	45°30'N
Longitude	90°30'W

Water Quality Objective

TSS Removal (%)	70

Upstream Storage

Storage (ha-m)	Discharge (L/s)
0	0

Stormceptor Sizing Summary

Stormceptor Model	TSS Removal %
STC 300	53
STC 750	66
STC 1000	66
STC 1500	66
STC 2000	**72**
STC 3000	73
STC 4000	77
STC 5000	78
STC 6000	81
STC 9000	85
STC 10000	85
STC 14000	88

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Particle Size Distribution

Removing silt particles from runoff ensures that the majority of the pollutants, such as hydrocarbons and heavy metals that adhere to fine particles, are not discharged into our natural water courses. The table below lists the particle size distribution used to define the annual TSS removal.

Fine (organics, silts and sand)

Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s		Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
20	20	1.3	0.0004					
60	20	1.8	0.0016					
150	20	2.2	0.0108					
400	20	2.65	0.0647					
2000	20	2.65	0.2870					

Stormceptor Design Notes

- Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor version 1.0
- Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal.
- Only the STC 300 is adaptable to function with a catch basin inlet and/or inline pipes.
- Only the Stormceptor models STC 750 to STC 6000 may accommodate multiple inlet pipes.
- Inlet and outlet invert elevation differences are as follows:

Inlet and Outlet Pipe Invert Elevations Differences

Inlet Pipe Configuration	STC 300	STC 750 to STC 6000	STC 9000 to STC 14000
Single inlet pipe	75 mm	25 mm	75 mm
Multiple inlet pipes	75 mm	75 mm	Only one inlet pipe.

- Design estimates are based on stable site conditions only, after construction is completed.
- Design estimates assume that the storm drain is not submerged during zero flows. For submerged applications, please contact your local Stormceptor representative.
- Design estimates may be modified for specific spills controls. Please contact your local Stormceptor representative for further assistance.
- For pricing inquiries or assistance, please contact Hanson Pipe & Precast, 1-888-888-3222.

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Appendix 2
Summary of Design Assumptions

SITE DETAILS

Site Drainage Area

Total Area (ha)	2.14	Imperviousness (%)	84

Surface Characteristics

Width (m)	293
Slope (%)	2
Impervious Depression Storage (mm)	0.508
Pervious Depression Storage (mm)	5.08
Impervious Manning's n	0.015
Pervious Manning's n	0.25

Infiltration Parameters

Horton's equation is used to estimate infiltration	
Max. Infiltration Rate (mm/h)	61.98
Min. Infiltration Rate (mm/h)	10.16
Decay Rate (s^{-1})	0.00055
Regeneration Rate (s^{-1})	0.01

Maintenance Frequency

Sediment build-up reduces the storage volume for sedimentation. Frequency of maintenance is assumed for TSS removal calculations.	
Maintenance Frequency (months)	12

Evaporation

Daily Evaporation Rate (mm/day)	2.54

Dry Weather Flow

Dry Weather Flow (L/s)	No

Upstream Attenuation

Stage-storage and stage-discharge relationship used to model attenuation upstream of the Stormceptor System is identified in the table below.

Storage ha-m	Discharge L/s
0	0

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PARTICLE SIZE DISTRIBUTION

Particle Size Distribution

Removing fine particles from runoff ensures the majority of pollutants, such as heavy metals, hydrocarbons, free oils and nutrients are not discharged into natural water resources. The table below identifies the particle size distribution selected to define TSS removal for the design of the Stormceptor System.

Fine (organics, silts and sand)

Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s		Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
20	20	1.3	0.0004					
60	20	1.8	0.0016					
150	20	2.2	0.0108					
400	20	2.65	0.0647					
2000	20	2.65	0.2870					

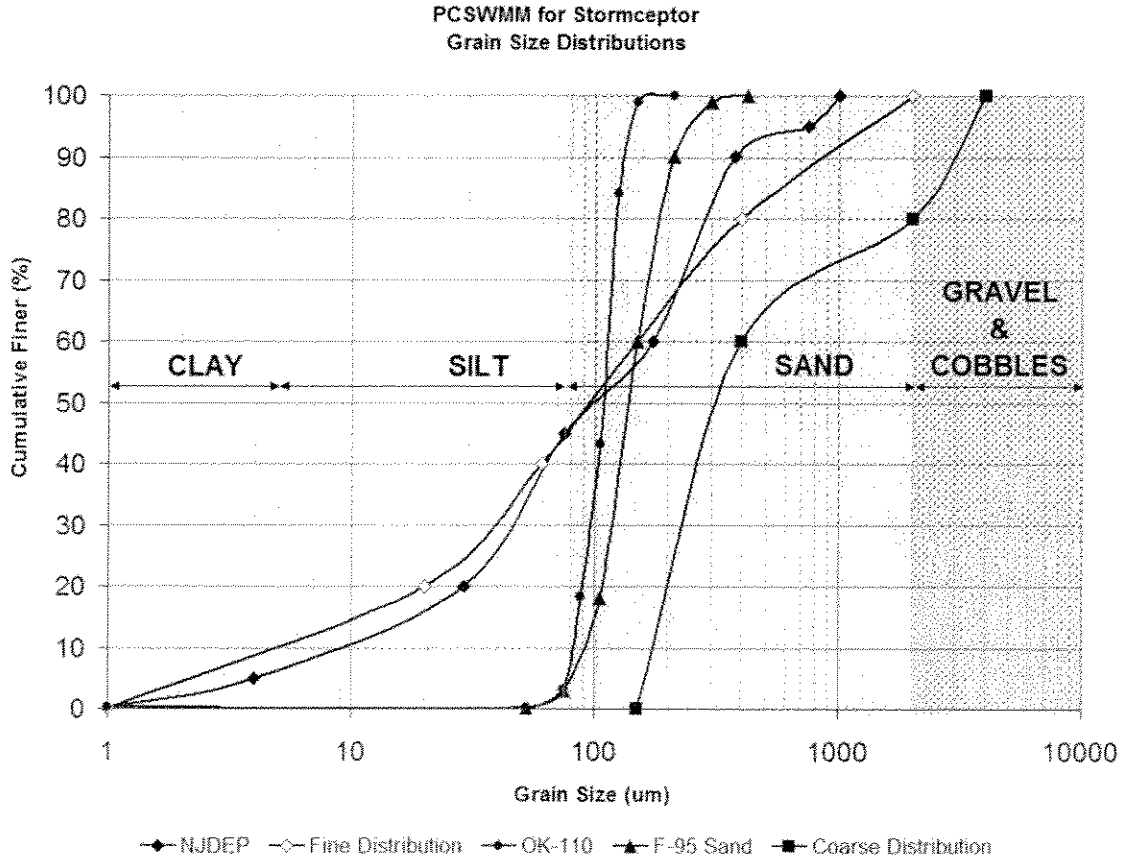


Figure 1. PCSWMM for Stormceptor standard design grain size distributions.

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TSS LOADING

TSS Loading Parameters

TSS Loading Function	Buildup / Washoff

Parameters

Target Event Mean Concentration (EMC) (mg/L)	125
Exponential Buildup Power	0.4
Exponential Washoff Exponential	0.2

HYDROLOGY ANALYSIS

PCSWMM for Stormceptor calculates annual hydrology with the US EPA SWMM and local continuous historical rainfall data. Performance calculations of the Stormceptor System are based on the average annual removal of TSS for the selected site parameters. The Stormceptor System is engineered to capture fine particles (silts and sands) by focusing on average annual runoff volume ensuring positive removal efficiency is maintained during all rainfall events, while preventing the opportunity for negative removal efficiency (scour).

Smaller recurring storms account for the majority of rainfall events and average annual runoff volume, as observed in the historical rainfall data analyses presented in this section.

Rainfall Station

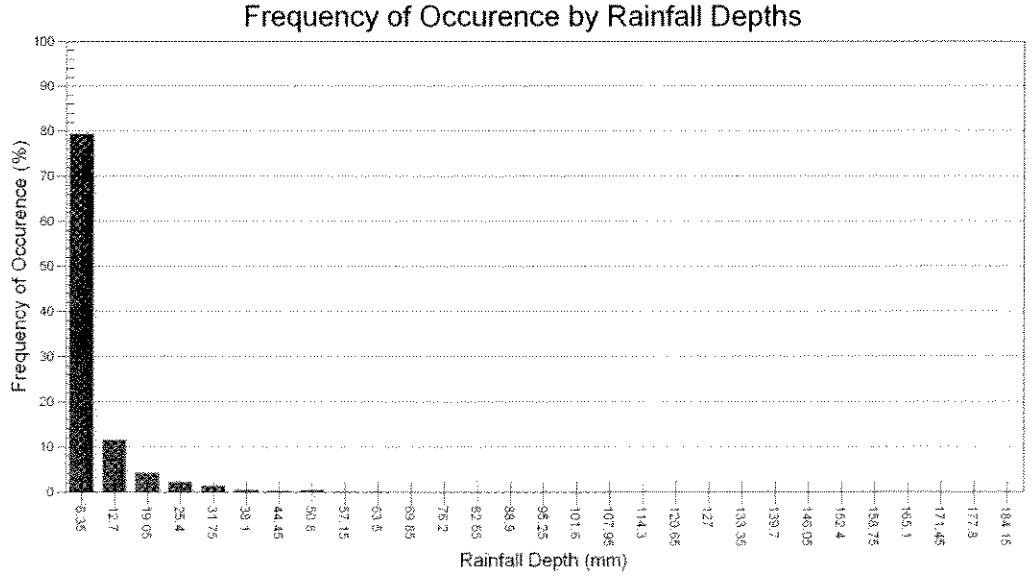
Rainfall Station	TORONTO CENTRAL		
Rainfall File Name	ON100.NDC	Total Number of Events	3020
Latitude	45°30'N	Total Rainfall (mm)	13190.7
Longitude	90°30'W	Average Annual Rainfall (mm)	732.8
Elevation (m)		Total Evaporation (mm)	1174.1
Rainfall Period of Record (y)	18	Total Infiltration (mm)	2101.3
Total Rainfall Period (y)	18	Percentage of Rainfall that is Runoff (%)	75.6

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Rainfall Event Analysis

Rainfall Depth mm	No. of Events	Percentage of Total Events %	Total Volume mm	Percentage of Annual Volume %
6.35	2398	79.4	3626	27.5
12.70	346	11.5	3182	24.1
19.05	130	4.3	2037	15.4
25.40	66	2.2	1432	10.9
31.75	38	1.3	1075	8.2
38.10	16	0.5	545	4.1
44.45	7	0.2	292	2.2
50.80	13	0.4	611	4.6
57.15	2	0.1	106	0.8
63.50	2	0.1	121	0.9
69.85	0	0.0	0	0.0
76.20	0	0.0	0	0.0
82.55	1	0.0	79	0.6
88.90	1	0.0	85	0.6
95.25	0	0.0	0	0.0
101.60	0	0.0	0	0.0
107.95	0	0.0	0	0.0
114.30	0	0.0	0	0.0
120.65	0	0.0	0	0.0
127.00	0	0.0	0	0.0
133.35	0	0.0	0	0.0
139.70	0	0.0	0	0.0
146.05	0	0.0	0	0.0
152.40	0	0.0	0	0.0
158.75	0	0.0	0	0.0
165.10	0	0.0	0	0.0
171.45	0	0.0	0	0.0
177.80	0	0.0	0	0.0
184.15	0	0.0	0	0.0
190.50	0	0.0	0	0.0
196.85	0	0.0	0	0.0
203.20	0	0.0	0	0.0
209.55	0	0.0	0	0.0
>209.55	0	0.0	0	0.0

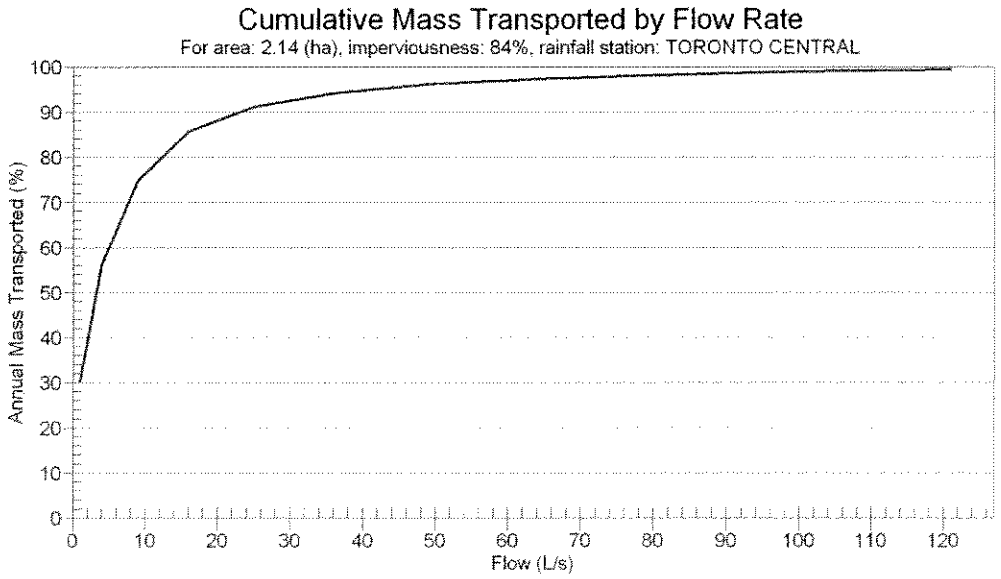


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Pollutograph

Flow Rate L/s	Cumulative Mass %
1	30.3
4	56.2
9	74.8
16	85.6
25	91.1
36	94.1
49	96.1
64	97.3
81	98.2
100	98.9
121	99.3
144	99.5
169	99.7
196	99.8
225	99.9
256	99.9
289	99.9
324	100.0
361	100.0
400	100.0
441	100.0
484	100.0
529	100.0
576	100.0
625	100.0
676	100.0
729	100.0
784	100.0
841	100.0
900	100.0



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STORMWATER STORAGE REQUIREMENTS

Project: 1900 APPLEBY LINE
CITY OF BURLINGTON

Chicago Storm Rainfall Information		
City/Town:	**Burlington**	
Return Period:	**5**	Years
A =	**802.04**	
B =	**5**	
C =	**0.764**	
Tc =	**10**	minutes
	600	seconds

Area of site being investigated (ha) =	**2.20**
Composite Runoff Coeff. (C) =	**0.77**
Release Rate - Q_{ALLOW} (m^3/s) =	**0.210**

(Max. discharge 100-year post-dev from Table 4 Phase 1 FSR)

Flows from site area calculated from area indicated above

Roof flows (Q_{ROOF}) added in as a constant flow rate into the orifice controlled system (if applicable)

Duration (T_D)		Rainfall Intensity		Post-Development Runoff			Runoff Volume	Release Volume	Storage Volume
				Site	Roof	Total "Q_{POST}"			
(min)	(sec)	(mm/hr)	(m/s)	(m^3/s)	(m^3/s)	(m^3/ha)	(m^3)	(m^3)	(m^3)
5	300	138.101	0.0000384	0.650	0.00000	0.6498	194.95	94.50	100.45
10	600	101.312	0.0000281	0.477	0.00000	0.4767	286.04	126.00	160.04
15	900	81.322	0.0000226	0.383	0.00000	0.3827	344.40	157.50	186.90
20	1200	68.576	0.0000190	0.323	0.00000	0.3227	387.22	189.00	198.22
25	1500	59.659	0.0000166	0.281	0.00000	0.2807	421.09	220.50	200.59
30	1800	53.031	0.0000147	0.250	0.00000	0.2495	449.17	252.00	197.17
35	2100	47.888	0.0000133	0.225	0.00000	0.2253	473.21	283.50	189.71
40	2400	43.766	0.0000122	0.206	0.00000	0.2059	494.27	315.00	179.27
45	2700	40.382	0.0000112	0.190	0.00000	0.1900	513.05	346.50	166.55
50	3000	37.546	0.0000104	0.177	0.00000	0.1767	530.02	378.00	152.02
55	3300	35.131	0.0000098	0.165	0.00000	0.1653	545.52	409.50	136.02
60	3600	33.047	0.0000092	0.156	0.00000	0.1555	559.81	441.00	118.81
65	3900	31.228	0.0000087	0.147	0.00000	0.1469	573.08	472.50	100.58
70	4200	29.624	0.0000082	0.139	0.00000	0.1394	585.48	504.00	81.48
75	4500	28.199	0.0000078	0.133	0.00000	0.1327	597.12	535.50	61.62
80	4800	26.923	0.0000075	0.127	0.00000	0.1267	608.10	567.00	41.10
85	5100	25.772	0.0000072	0.121	0.00000	0.1213	618.49	598.50	19.99
90	5400	24.729	0.0000069	0.116	0.00000	0.1164	628.38	630.00	-1.62
95	5700	23.779	0.0000066	0.112	0.00000	0.1119	637.80	661.50	-23.70
100	6000	22.909	0.0000064	0.108	0.00000	0.1078	646.80	693.00	-46.20
105	6300	22.109	0.0000061	0.104	0.00000	0.1040	655.43	724.50	-69.07
110	6600	21.371	0.0000059	0.101	0.00000	0.1006	663.71	756.00	-92.29
115	6900	20.687	0.0000057	0.097	0.00000	0.0973	671.68	787.50	-115.82
120	7200	20.052	0.0000056	0.094	0.00000	0.0944	679.36	819.00	-139.64
125	7500	19.460	0.0000054	0.092	0.00000	0.0916	686.78	850.50	-163.72
130	7800	18.907	0.0000053	0.089	0.00000	0.0890	693.95	882.00	-188.05
135	8100	18.389	0.0000051	0.087	0.00000	0.0865	700.89	913.50	-212.61
140	8400	17.902	0.0000050	0.084	0.00000	0.0842	707.62	945.00	-237.38
145	8700	17.445	0.0000048	0.082	0.00000	0.0821	714.15	976.50	-262.35
150	9000	17.013	0.0000047	0.080	0.00000	0.0801	720.50	1008.00	-287.50
155	9300	16.605	0.0000046	0.078	0.00000	0.0781	726.68	1039.50	-312.82
160	9600	16.220	0.0000045	0.076	0.00000	0.0763	732.69	1071.00	-338.31
165	9900	15.854	0.0000044	0.075	0.00000	0.0746	738.55	1102.50	-363.95
170	10200	15.507	0.0000043	0.073	0.00000	0.0730	744.26	1134.00	-389.74
175	10500	15.176	0.0000042	0.071	0.00000	0.0714	749.84	1165.50	-415.66
180	10800	14.862	0.0000041	0.070	0.00000	0.0699	755.29	1197.00	-441.71

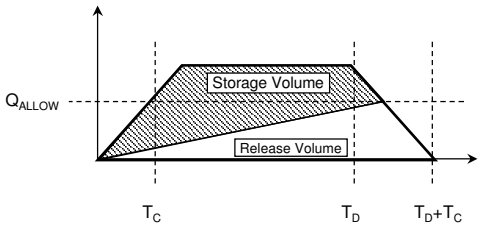
***Max. required storage volume =**	**70.59 m^3**

Q_{POST} = (C i A) x 10000 m^2/ha (Rational Method)

Runoff Volume = Area under trapezoidal hydrograph
= $(T_D - T_C)Q_{POST} + (T_C\ Q_{POST})$

Release Volume = Area under triangular outflow hydrograph
= ½ $(T_D + T_C)\ Q_{ALLOW}$

Storage Volume = Runoff Volume - Release Volume



* Approx. 130m3 of surface storage provided in existing parking lots to remain

STORMWATER STORAGE REQUIREMENTS

Project: 1900 APPLEBY LINE
CITY OF BURLINGTON

Chicago Storm Rainfall Information		
City/Town:	**Burlington**	
Return Period:	**100**	Years
A =	**1281.34**	
B =	**5**	
C =	**0.761**	
Tc =	**10**	minutes
	600	seconds

Area of site being investigated (ha) = **2.20**
Composite Runoff Coeff. (C) = **0.77**
Release Rate - Q_{ALLOW} (m^3/s) = **0.210** (Max. discharge 100-year post-dev from Table 4 Phase 1 FSR)

Flows from site area calculated from area indicated above

Roof flows (Q_{ROOF}) added in as a constant flow rate into the orifice controlled system (if applicable)

Duration (T_D)		Rainfall Intensity		Post-Development Runoff			Runoff Volume	Release Volume	Storage Volume
				Site	Roof	Total "Q_{POST}"			
(min)	(sec)	(mm/hr)	(m/s)	(m^3/s)	(m^3/s)	(m^3/ha)	(m^3)	(m^3)	(m^3)
5	300	222.159	0.0000617	1.045	0.00000	1.0454	313.61	94.50	219.11
10	600	163.177	0.0000453	0.768	0.00000	0.7678	460.70	126.00	334.70
15	900	131.093	0.0000364	0.617	0.00000	0.6169	555.18	157.50	397.68
20	1200	110.620	0.0000307	0.521	0.00000	0.5205	624.63	189.00	435.63
25	1500	96.289	0.0000267	0.453	0.00000	0.4531	679.64	220.50	459.14
30	1800	85.631	0.0000238	0.403	0.00000	0.4029	725.29	252.00	473.29
35	2100	77.356	0.0000215	0.364	0.00000	0.3640	764.41	283.50	480.91
40	2400	70.724	0.0000196	0.333	0.00000	0.3328	798.71	315.00	483.71
45	2700	65.275	0.0000181	0.307	0.00000	0.3072	829.32	346.50	482.82
50	3000	60.708	0.0000169	0.286	0.00000	0.2857	857.00	378.00	479.00
55	3300	56.819	0.0000158	0.267	0.00000	0.2674	882.30	409.50	472.80
60	3600	53.461	0.0000149	0.252	0.00000	0.2516	905.63	441.00	464.63
65	3900	50.529	0.0000140	0.238	0.00000	0.2378	927.30	472.50	454.80
70	4200	47.945	0.0000133	0.226	0.00000	0.2256	947.55	504.00	443.55
75	4500	45.647	0.0000127	0.215	0.00000	0.2148	966.58	535.50	431.08
80	4800	43.589	0.0000121	0.205	0.00000	0.2051	984.53	567.00	417.53
85	5100	41.734	0.0000116	0.196	0.00000	0.1964	1001.54	598.50	403.04
90	5400	40.051	0.0000111	0.188	0.00000	0.1885	1017.70	630.00	387.70
95	5700	38.518	0.0000107	0.181	0.00000	0.1812	1033.12	661.50	371.62
100	6000	37.114	0.0000103	0.175	0.00000	0.1746	1047.86	693.00	354.86
105	6300	35.823	0.0000100	0.169	0.00000	0.1686	1061.98	724.50	337.48
110	6600	34.632	0.0000096	0.163	0.00000	0.1630	1075.54	756.00	319.54
115	6900	33.528	0.0000093	0.158	0.00000	0.1578	1088.60	787.50	301.10
120	7200	32.502	0.0000090	0.153	0.00000	0.1529	1101.18	819.00	282.18
125	7500	31.547	0.0000088	0.148	0.00000	0.1484	1113.33	850.50	262.83
130	7800	30.654	0.0000085	0.144	0.00000	0.1442	1125.09	882.00	243.09
135	8100	29.817	0.0000083	0.140	0.00000	0.1403	1136.47	913.50	222.97
140	8400	29.031	0.0000081	0.137	0.00000	0.1366	1147.50	945.00	202.50
145	8700	28.292	0.0000079	0.133	0.00000	0.1331	1158.21	976.50	181.71
150	9000	27.594	0.0000077	0.130	0.00000	0.1298	1168.63	1008.00	160.63
155	9300	26.936	0.0000075	0.127	0.00000	0.1267	1178.75	1039.50	139.25
160	9600	26.312	0.0000073	0.124	0.00000	0.1238	1188.62	1071.00	117.62
165	9900	25.721	0.0000071	0.121	0.00000	0.1210	1198.23	1102.50	95.73
170	10200	25.160	0.0000070	0.118	0.00000	0.1184	1207.60	1134.00	73.60
175	10500	24.626	0.0000068	0.116	0.00000	0.1159	1216.75	1165.50	51.25
180	10800	24.118	0.0000067	0.113	0.00000	0.1135	1225.69	1197.00	28.69

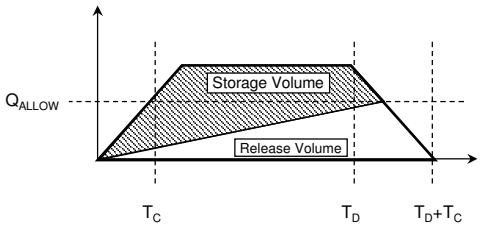
***Max. required storage volume = 353.71 m^3**

Q_{POST} = (C i A) x 10000 m^2/ha (Rational Method)

Runoff Volume = Area under trapezoidal hydrograph
= $(T_D - T_C)Q_{POST} + (T_C\ Q_{POST})$

Release Volume = Area under triangular outflow hydrograph
= ½ $(T_D + T_C)\ Q_{ALLOW}$

Storage Volume = Runoff Volume - Release Volume



* Approx. 130m3 of surface storage provided in existing parking lots to remain

1900 APPLEBY LINE
CITY OF BURLINGTON
STORMWATER MANAGEMENT

Project Number: 49579-100
Date: October 15, 2021
File:

Orifice Calculations for Catchment 201 +202 $Q_o=C_d*A_o*(2*g*H_o)^{0.5}$		
	Orifice	**Description**
C_d	0.63	Orifice plate
Invert (m)	133.98	
CL elevation (m)	134.11	
Diameter (mm)	250	
Type (H/V)	V	

STAGE-STORAGE-DISCHARGE RELATIONSHIP

Description	**Stage**	**Contour Area**	**Incremental Volume**	**Cumulative Volume**	**Orifice**		
					Orifice Area	**H_o**	**Flow**
	m	m^2	m^3	m^3	m^2	*m*	m^3/s
Inside Bottom of Tank	133.98	232.0	0.0	0.0	0.049	0.00	0.0000
C/L of orifice	134.11	232.0	29.0	29.0	0.049	0.00	0.0000
1/3 Storm Tank	134.52	232.0	96.3	125.3	0.049	0.41	0.0882
2/3 Storm Tank	135.06	232.0	125.3	250.6	0.049	0.96	0.1339
75% Storm Tank	135.20	232.0	31.3	281.9	0.049	1.09	0.1430
Top of Tank	135.60	232.0	94.0	375.8	0.049	1.50	0.1675

Appendix B

Sanitary Sewer Calculations

Mixed-Use Building
1900 Appleby Line
MTE Project #: 49579-100
Date: August 2021
By: SXL

Sanitary Demand Calculations

Location	Residential					Commercial		Total Residential and Commercial		
	Site Area (ha)	Units [6]	Population Density	Population (persons)	Demand [2] (L/s)	Floor Area (ha)	Demand [3] (L/s)	Total Demand (L/s)	Total Peaked Demand [4] (L/s)	Total Peaked Demand +Infiltration [5] (L/s)
As Desgined per Phase 1 FSR										
Entire Site	2.1		285	599	1.905			1.905	7.49	
Total Designed per Phase 1 FSR										8.09
Mixed-Use Tower										
Studio [1]		16	2	32	0.102				0.402	
1 Bedroom [1]		15	2	30	0.095				0.377	
1 Bedroom + Den [7]		77	3	231	0.735				2.900	
2 Bedroom [1]		32	4	128	0.407				1.607	
2 Bedroom + Den [7]		30	5	150	0.477				1.883	
Sub Total		170		571	1.817			1.817	**7.168**	
Commercial Area						0.226	0.065	0.065	**0.256**	
									Phase 1 Total	**7.518**
Mixed-Use Tower										
Studio [1]		15	2	30	0.095				0.375	
1 Bedroom [1]		78	2	156	0.497				1.951	
1 Bedroom + Den [7]		36	3	108	0.344				1.351	
2 Bedroom [1]		48	4	192	0.611				2.402	
2 Bedroom + Den [7]		24	5	120	0.382				1.501	
Sub Total		201		606	1.929			1.929	**7.580**	
Commercial Area						0.115	0.033	0.033	**0.130**	
									Phase 2 Total	**7.828**
Mixed-Use Tower										
Studio [1]		12	2	24	0.076				0.301	
1 Bedroom [1]		78	2	156	0.497				1.960	
1 Bedroom + Den [7]		36	3	108	0.344				1.357	
2 Bedroom [1]		39	4	156	0.497				1.960	
2 Bedroom + Den [7]		24	5	120	0.382				1.507	
Sub Total		189		564	1.795			1.795	**7.085**	
Commercial Area						0.052	0.015	0.015	**0.059**	
									Phase 3 Total	**7.291**
Total (Residential + Commercial) - All Phases				**1741**	**5.541**	**0.393**	**0.112**	**7.559**	**22.278**	**22.638**

Sanitary Demand		
Residential Daily Demands [2]	275	L/ca/day
	0.0032	L/ca/sec
Commercial Space Daily Demands [3]	24.75	m³/d/ha
	0.29	L/s/ha
Harmon PF [4] (Ex. Design)	3.93	
Harmon PF [4] (Proposed- Phase 1)	3.94	
Harmon PF [4] (Proposed- Phase 2)	3.93	
Harmon PF [4] (Proposed- Phase 3)	3.95	
Site Area- Phase 1	0.33	ha
Site Area- Phase 2	0.41	ha
Site Area- Phase 3	0.51	ha
Infiltration Allowance [5]	0.286	L/s/ha
Phase 1	0.094	L/s
Phase 2	0.118	L/s
Phase 3	0.147	L/s

0.360

Note 1: Population Density based on OBC Section 3.1.17.1, Clause 1(b). 2 Persons per bedroom
Note 2: Residential Demand based on Halton Region Design Standards
Note 3: Commercial Space Demand based on Halton Region Design Standards
Note 4:Harmon Peaking Factor = 1 + (14/(4+(P/1000)½)), 5.0 max (modified 0.8PF for commercial)
Note 5: Infiltration allowance from Halton Region Design Standards.
Note 6: Unit Count taken from Site plan by Core Architecture received March 15, 2021

Note 7: Population Densities for 1 Bedroom + Den was estimated to be 3 persons per unit due to possibility of den being used at small bedroom

1900 Appleby Line
CITY OF BURLINGTON

Project No. 49579-100
Date: October 15, 2021
Designed By: SXL
Checked By: GPB
File: Q:\49579\100\SAN\49579-100 Sanitary Sewer Design Sheet.xls

SANITARY SEWER DESIGN SHEET

ENGINEERING SERVICES

DESIGN PARAMETERS

Design Flow
Design flow factor: 275 L/c/d
Infiltration Factor: 0.286 L/s/d
Mannings n: 0.013 dia < 600mm
0.013 dia ≥ 600mm
Min Velocity: 0.6 m/s
Max Velocity: 3 m/s

Peaking Factor
Harmon Formula

$$M = K_{ex} \times \left(1 + \frac{14}{4 + \sqrt{P + P_i}}\right)$$

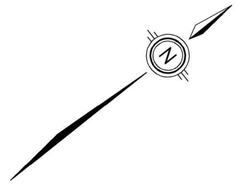
$$K_{ex} = \frac{A_x + 0.80 \times (A_i + A_c)}{A_x + A_i + A_c}$$

Max Peaking factor 4
Min Peaking Factor 2

LOCATION				SANITARY FLOW												DESIGN							
																Proposed Sewer Design					Actual	%	
Area No.	Street Name	From MH	To MH	Pop. Density [per/ha]	Incremental Area [ha]	Population Area [ha]	Cumulative Area [ha]	Total Area [ha]	Population Increment [per]	Cumulative Population [per]	Peaking Factor M	Average Flow l/s	Peak Flow l/s	Infil-tration l/s	Total Flow l/s	Diameter [mm]	Material n=0.015	Grade %	Capacity l/s	Velocity m/s	Velocity m/s	Pipe Full	Remarks
	Pre-Development																						
	Ironstone Drive	Ex.MH1	Ex.MH2					85.35	9693	9693	2.97	30.85	91.57	24.41	115.98	450	VC	0.60%	218.07	1.39	1.42	53.2%	
1 & 2	Ironstone Drive	Ex.MH2	Ex.MH3	90	0.15	2.55	2.70	88.05	230	230	4.00	0.73	2.92	0.77	119.68	450	VC	0.50%	199.07	1.27	1.33	60.1%	
3 & 4	Ironstone Drive	Ex.MH3	Ex.MH4	90	0.09	2.22	5.01	93.06	200	429	4.00	1.37	5.47	1.43	122.88	450	VC	0.50%	199.07	1.27	1.34	61.7%	
5	Ironstone Drive	Ex.MH4	Ex.MH5	90	0.13		5.14	98.20		429	4.00	1.37	5.47	1.47	122.92	450	VC	0.58%	214.40	1.37	1.42	57.3%	
	Heron Way		Ex.MH6					12.69	1040	1040	3.79	3.31	12.54	3.63	16.17	300	CONC	0.84%	88.12	1.25	0.97	18.3%	
6	Heron Way	Ex.MH6	Ex.MH5	90	0.17		0.17	12.86		0	4.00	0.05	0.20	0.05	16.42	300	CONC	1.00%	96.15	1.37	1.03	17.1%	
7	Ironstone Drive	Ex.MH5	Ex.MH7	90		0.53	5.84	116.90	48	477	3.98	1.52	6.05	1.67	139.87	450	VC	0.58%	214.40	1.37	1.46	65.2%	
8,9,10	Ironstone Drive	Ex.MH7	Ex.MH8	90	0.17	1.48	7.49	124.39	133	610	3.93	1.94	7.63	2.14	141.92	450	VC	0.67%	230.44	1.47	1.55	61.6%	
11 & 12	Ironstone Drive	Ex.MH8	Ex.MH9	90	0.08	0.92	8.49	132.88	83	693	3.90	2.21	8.60	2.43	143.18	450	VC	0.62%	221.67	1.41	1.50	64.6%	
13 & 14	Ironstone Drive	Ex.MH9	Ex.MH10	90	0.07	1.96	10.52	143.40	176	869	3.84	2.77	10.62	3.01	145.78	450	VC	0.61%	219.88	1.40	1.50	66.3%	
15,16,17	Ironstone Drive	Ex.MH10	Ex.MH11	90	0.20	2.01	12.73	156.13	181	1050	3.79	3.34	12.66	3.64	148.45	450	VC	0.58%	214.40	1.37	1.48	69.2%	
18,19,20	Ironstone Drive	Ex.MH11	Ex.MH12	90	0.10	1.60	14.43	170.56	144	1194	3.75	3.80	14.25	4.13	150.53	450	VC	2.34%	430.65	2.74	2.51	35.0%	
	Post - Development - Phase 1																						
	Ironstone Drive	Ex.MH1	Ex.MH2					85.35	9693	9693	2.97	30.85	91.57	24.41	115.98	450	VC	0.60%	218.07	1.39	1.42	53.2%	
1 & 2	Ironstone Drive	Ex.MH2	Ex.MH3	90	0.15	2.55	2.70	88.05	230	230	4.00	0.73	2.92	0.77	119.68	450	VC	0.50%	199.07	1.27	1.33	60.1%	
3 & 4	Ironstone Drive (Ex. Commercial)	Ex.MH3	Ex.MH4	90	0.09	1.89	4.68	92.73	170	400	4.00	1.27	5.09	1.34	122.41	450	CONC	0.50%	199.07	1.27	1.34	65.3%	
	Ironstone Drive (Prop Tower/s)	Ex.MH3	Ex.MH4			0.33									7.52								
5	Ironstone Drive	Ex.MH4	Ex.MH5	90	0.13	0.00	4.81	97.54	0	400	4.00	1.27	5.09	1.38	129.96	450	VC	0.58%	214.40	1.37	1.43	60.6%	
	Heron Way		Ex.MH6					12.69	1040	1040	3.79	3.31	12.54	3.63	16.17	300	CONC	0.84%	88.12	1.25	0.97	18.3%	
6	Heron Way	Ex.MH6	Ex.MH5	90	0.17		0.17	12.86		0	4.00	0.05	0.20	0.05	16.42	300	CONC	1.00%	96.15	1.37	1.03	17.1%	
7	Ironstone Drive	Ex.MH5	Ex.MH7	90		0.53	5.51	115.91	48	447	4.00	1.42	5.69	1.58	153.65	525	CONC	0.58%	332.52	1.51	1.49	46.2%	
8,9,10	Ironstone Drive	Ex.MH7	Ex.MH8	90	0.17	1.48	7.16	123.07	133	581	3.94	1.85	7.28	2.05	162.98	525	CONC	0.67%	357.39	1.63	1.60	45.6%	
11 & 12	Ironstone Drive	Ex.MH8	Ex.MH9	90	0.08	0.92	8.16	131.23	83	663	3.91	2.11	8.25	2.33	164.23	525	CONC	0.62%	343.80	1.56	1.55	47.8%	
13 & 14	Ironstone Drive	Ex.MH9	Ex.MH10	90	0.07	1.96	10.19	141.42	176	840	3.85	2.67	10.28	2.91	166.85	525	CONC	0.61%	341.01	1.55	1.55	48.9%	
15,16,17	Ironstone Drive	Ex.MH10	Ex.MH11	90	0.20	2.01	12.40	153.82	181	1021	3.79	3.25	12.33	3.55	169.52	525	CONC	0.58%	332.52	1.51	1.52	51.0%	
18,19,20	Ironstone Drive	Ex.MH11	Ex.MH12	90	0.10	1.60	14.10	167.92	144	1165	3.76	3.71	13.92	4.03	171.61	450	VC	2.34%	430.65	2.74	2.60	39.8%	
	Post - Development - Ultimate Phase																						
	Ironstone Drive	Ex.MH1	Ex.MH2					85.35	9693	9693	2.97	30.85	91.57	24.41	115.98	450	VC	0.60%	218.07	1.39	1.42	53.2%	
1 & 2	Ironstone Drive	Ex.MH2	Ex.MH3	90	0.15	2.55	2.70	88.05	230	230	4.00	0.73	2.92	0.77	119.68	450	VC	0.50%	199.07	1.27	1.33	60.1%	
3 & 4	Ironstone Drive (Ex. Commercial)	Ex.MH3	Ex.MH4	90	0.09	0.97	3.76	91.81	87	317	4.00	1.01	4.03	1.08	121.09	525	VC	0.50%	308.74	1.40	1.33	46.6%	
	Ironstone Drive (Prop Tower/s)	Ex.MH3	Ex.MH4			1.25									22.64								
5	Ironstone Drive	Ex.MH4	Ex.MH5	90	0.13	2.10	3.89	95.70		317	4.00	1.01	4.03	1.11	148.88	450	VC	0.58%	214.40	1.37	1.48	69.4%	
	Heron Way		Ex.MH6					12.69	1040	1040	3.79	3.31	12.54	3.63	16.17	300	CONC	0.84%	88.12	1.25	0.97	18.3%	
6	Heron Way	Ex.MH6	Ex.MH5	90	0.17		0.17	12.86		0	4.00	0.05	0.20	0.05	16.42	300	CONC	1.00%	96.15	1.37	1.03	17.1%	
7	Ironstone Drive	Ex.MH5	Ex.MH7	90		0.53	4.59	113.15	48	365	4.00	1.16	4.64	1.31	171.10	525	VC	0.58%	332.52	1.51	1.53	51.5%	
8,9,10	Ironstone Drive	Ex.MH7	Ex.MH8	90	0.17	1.48	6.24	119.39	133	498	3.98	1.58	6.30	1.78	179.18	525	VC	0.67%	357.39	1.63	1.63	50.1%	
11 & 12	Ironstone Drive	Ex.MH8	Ex.MH9	90	0.08	0.92	7.24	126.63	83	581	3.94	1.85	7.28	2.07	188.53	525	VC	0.62%	343.80	1.56	1.60	54.8%	
13 & 14	Ironstone Drive	Ex.MH9	Ex.MH10	90	0.07	1.96	9.27	135.90	176	757	3.87	2.41	9.33	2.65	200.51	525	VC	0.61%	341.01	1.55	1.62	58.8%	
15,16,17	Ironstone Drive	Ex.MH10	Ex.MH11	90	0.20	2.01	11.48	147.38	181	938	3.82	2.98	11.40	3.28	215.19	525	VC	0.58%	332.52	1.51	1.61	64.7%	
18,19,20	Ironstone Drive	Ex.MH11	Ex.MH12	90	0.10	1.60	13.18	160.56	144	1082	3.78	3.44	13.01	3.77	231.97	450	VC	2.34%	430.65	2.74	2.81	53.9%	

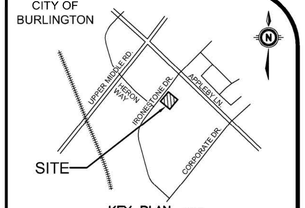
Legend = ***Proposed Upgraded Pipe***





LEGEND

- SITE BOUNDARY
- EXISTING DIRECTION OF DRAINAGE/SWALE
- Ex. 375mmø STM — Ex. MH — EXISTING STORM SEWER
- CATCHMENT 201
- CATCHMENT 202
- CATCHMENT 203
- CATCHMENT 204
- 201 / 0.1 / 2 — ID No. / AREA (Ha) / PERCENT IMPERVIOUS AS PER APPROVED PHASE 1 FSR AND SWM REPORT
- SWM TANK
- OVERLAND FLOW ROUTE (MAJOR STORM)



KEY PLAN N.T.S.

GEODETIC BM ELEV. =135.83 m
ELEVATIONS ARE ASSUMED AND ARE REFERED TO TOP OF FIRE HYDRANT LOCATED IN SOUTH EAST CORNER OF LOT

SITE BENCHMARK ELEV. = m
SEE GEODETIC BM

NOTE TO CONTRACTOR :

DO NOT SCALE DRAWINGS.

CONTRACTORS MUST CHECK AND VERIFY ALL DIMENSIONS AND REPORT ANY DISCREPANCIES TO THE ENGINEER BEFORE PROCEEDING WITH THE WORK.

ALL DRAWINGS REMAIN THE PROPERTY OF THE ENGINEER AND SHALL NOT BE REPRODUCED OR REUSED WITHOUT THE ENGINEER'S WRITTEN PERMISSION.

THE OWNER/ARCHITECT/CONTRACTOR IS ADVISED THAT M.T.E. CONSULTANTS INC. CANNOT CERTIFY ANY COMPONENT OF THE SITE WORKS NOT INSPECTED DURING CONSTRUCTION. IT IS THE RESPONSIBILITY OF THE GENERAL CONTRACTOR TO NOTIFY M.T.E. CONSULTANTS INC. PRIOR TO COMMENCEMENT OF CONSTRUCTION TO ARRANGE FOR INSPECTION.

No.	REVISION	BY	YYYY-MM-DD
8.			
7.			
6.			
5.			
4.			
3.			
2.	ISSUED FOR ZBA	GPB	2021-10-15
1.	ISSUED FOR COORDINATION	GPB	2021-10-07

MTE
Engineers, Scientists, Surveyors
905-639-2552

CLIENT
LJM DEVELOPMENTS
1900 APPLEBY LINE BURLINGTON

PROJECT
1900 APPLEBY LINE
PROPOSED MIXED-USE BUILDING
1900 APPLEBY LINE BURLINGTON

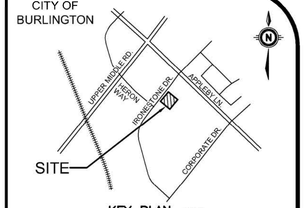
DRAWING
EXTERNAL SANITARY DRAINAGE AREA PLAN EXCISTING

Project Manager G.BERENYL	Project No. 49579-100
Design By TJL	Checked By
Drawn By TJL	Checked By
Surveyed By Others	Drawing No. SA1.1
Date Aug.26/21	
Scale NTS	Sheet 2 of 2

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MTE FILE PATH:
October 15, 2021 - 12:37:03 PM - Plotted By: Taylor Leet



CITY OF BURLINGTON



KEY PLAN N.T.S.

GEODETIC BM ELEV. =135.83 m
ELEVATIONS ARE ASSUMED AND ARE REFERED TO TOP OF FIRE HYDRANT LOCATED IN SOUTH EAST CORNER OF LOT

SITE BENCHMARK ELEV. = m
SEE GEODETIC BM

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LEGEND

- SITE BOUNDARY
- EXISTING DIRECTION OF DRAINAGE/SWALE
- Ex. 375mmø STM — Ex. MH — EXISTING STORM SEWER
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- OVERLAND FLOW ROUTE (MAJOR STORM)

No.	REVISION	BY	YYYY-MM-DD
8.			
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5.			
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2.	ISSUED FOR ZBA	GPB	2021-10-15
1.	ISSUED FOR COORDINATION	GPB	2021-10-07

MTE
Engineers, Scientists, Surveyors
905-639-2552

CLIENT

LJM DEVELOPMENTS

1900 APPLEBY LINE BURLINGTON

PROJECT

1900 APPLEBY LINE
PROPOSED MIXED-USE BUILDING

1900 APPLEBY LINE BURLINGTON

DRAWING

EXTERNAL SANITARY DRAINAGE AREA PLAN PROPOSED

Project Manager G.BERENYL	Project No. 49579-100
Design By TJL	Checked By
Drawn By TJL	Checked By
Surveyed By Others	Drawing No. SA1.2
Date Aug.26/21	
Scale NTS	Sheet 2 of 2

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Appendix C

Water Demand Calculations

Mixed-Use Building
1900 Appleby Line
MTE Project #: 49579-100
Date: July 2021

FIRE FLOW DEMAND REQUIREMENTS - FIRE UNDERWRITERS SURVEY (FUS GUIDELINES)

Fire flow demands for the FUS method is based on information and guidance provided in "Water Supply for Public Protection" (Fire Underwriters Survey, 1999).

An estimate of the fire flow required is given by the following formula:

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

where:

F = the required fire flow in litres per minute
C = coefficient related to the type of construction
= 1.5 for wood frame construction (structure essentially all combustible).
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
= 0.8 for non-combustible construction (unprotected metal structural components, masonry or metal walls)
= 0.6 for fire-resistive construction (fully protected frame, floors, roof)
A = Gross floor area in square metres. For fire-resistive buildings, consider the two largest adjoining floors plus 50 percent of each of any floors immediately above them up to eight

Adjustments to the calculated fire flow can be made based on occupancy, sprinkler protection and exposure to other structures. The table below summarizes the adjustments made to the basic fire flow demand.

			(1)		(2)		(3)		(4)		**Final Adjusted**		
Phase	Area "A" [A]	C [B]	Fire Flow "F"		Occupancy		Sprinkler		Exposure		**Fire Flow**		
	(m^2)		(l/min)	(l/s)	%	Adjusted Fire Flow (L/min)	%	Adjustment (L/min)	%	Adjustment (L/min)	**(L/min)**	**Rounded(L/min)**	**(L/s)**
Phase 1 - Building	4,376.00	0.6	9,000	150.0	-15	7,650	-40	-3,060	40	3,060	7,650	8,000	**133**
Phase 2 - Building	4,670.00	0.6	9,000	150.0	-15	7,650	-40	-3,060	20	1,530	6,120	6,000	**100**
Phase 3 - Building	4,700.00	0.6	9,000	150.0	-15	7,650	-40	-3,060	0	0	4,590	5,000	**83**
												Max required:	**133**

Note [A]: Area "A" represents the Gross Floor Area as indicated on the unit matrix.

Note [B]: Construction type confirmed by the Architect

Is there any Firewall?

(2) Occupancy

Non-Combustible	-25%
Limited Combustible	-15%
Combustible	No charge
Free Burning	15%
Rapid Burning	25%

(3) Sprinkler

40% credit for adequately designed system per NFPA 13. Additional 10% if water supply standard for both the system and fire department hose lines.

(4) Exposure

0 to 3m	25%	
3.1 to 10m	20%	Calculate for all
10.1 to 20m	15%	sides. Maximum
20.1 to 30m	10%	charge shall not
30.1 to 45m	5%	exceed 75%

Phase 1

N	20.1 to 30m	10
S	20.1 to 30m	10
E	20.1 to 30m	10
W	20.1 to 30m	10
	TOTAL	40

Phase 2

N	30.1 to 45r	5
S	30.1 to 45r	5
E	30.1 to 45r	5
W	30.1 to 45r	5
	TOTAL	20

Phase 3

N	> 45m	0
S	> 45m	0
E	> 45m	0
W	> 45m	0
	TOTAL	0

Note: 10% exposure for fire wall.

Mixed-Use Building
1900 Appleby Line
MTE Project #: 49579-100
Date: July 2021

Residential Peaking Factors[1]:		Commercial Peaking Factors[1]:	
Avg. Day	1.0	Avg. Day	1.0
Max. Day	2.25	Max. Day	2.25
Peak Hour	4.0	Peak Hour	2.25

Water Demand Calculations

	Residential				Commerical		Final Demand					
Location	Site Area (ha)	Population Density (ppha)	Population[3] (persons)	Demand (L/s)	Floor Area (ha)	Demand[3] (L/s)	Avg Day Demand Qavg (L/s)	Max Day Demand Qmax.day (L/s)	Peak Hour Demand Qpeak (L/s)			
Designed Per Phase 1												
Entire Site	2.2	285	627	2.00			2.00	4.49	7.98			
Total Designed for Phase 1	2.2		627	2.00			2.00	4.49	7.98			
Proposed Phase												
Phase 1			571	1.817			1.817	4.088	7.268	5.541	12.467	22.164
Phase 1- Commercial					0.2260	0.0647	0.0650	0.146	0.146	0.1130	0.2543	0.2543
Phase 2			606	1.929			1.929	4.340	7.716			
Phase 2- Commercial					0.1146	0.0328	0.0330	0.074	0.074			
Phase 3			564	1.795			1.795	4.039	7.180			
Phase 3- Commercial					0.0520	0.0149	0.0150	0.034	0.034			
Total	0.95		1741	5.541	0.3926	0.1125	**5.654**	**12.722**	**22.42**			

Water Demand[4]		
Average Residential Daily Demands	275	L/d/person
	0.0032	L/s/person
Average Commercial Daily Demands	24.75	m^3/d/ha
	0.29	L/s/ha

Fire Flow[5]		
Fire Flow- Phase 1	133.33	L/s
Fire Flow- Phase 1	100.00	L/s
Fire Flow- Phase 1	83.33	L/s

Max Day + Fire Flow Demand		
Qmax.day+fire- Phase 1	137.57	L/s
Qmax.day+fire- Phase 1	104.41	L/s
Qmax.day+fire- Phase 1	87.41	L/s

Note 1: Peaking factors based on Halton Region Design Guidelines
Note 2: Residential Population Density based on OBC 3.1.17.1.1(c)(i)
Note 3: Refer to Sanitary in Appendix B for population calcualtions.
Note 4: Water Demands based on Halton Region Design Guidelines
Note 5: Fire flows calculated using FUS (1999) guidelines - See attached worksheet, maximum fire flow demand of the three phases used