URBANTECH®

FUNCTIONAL SERVICING REPORT

MILLCROFT GOLF CLUB

CITY OF BURLINGTON

REGION OF HALTON

PREPARED FOR
MILLCROFT GREENS CORPORATION

Urbantech File No.: 17-526

2nd SUBMISSION – APRIL 2022

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TABLE OF CONTENTS

1 EXECUTIVE SUMMARY 1

1.1. SANITARY SERVICING 1
1.2. WATER SERVICING 1
1.3. STORM SERVICING AND STORMWATER MANAGEMENT 1
1.4. GRADING 2

2 INTRODUCTION 3

2.1. BACKGROUND 3
2.2. SUBJECT PROPERTY 3

3 DESIGN CRITERIA AND OBJECTIVES 5

3.1. SANITARY DESIGN CRITERIA 5
3.2. WATER DESIGN CRITERIA 6
3.3. STORM DESIGN CRITERIA 7
3.4. STORMWATER MANAGEMENT DESIGN CRITERIA 9
3.5. GRADING DESIGN OBJECTIVES 10

4 AREAS A & B 11

4.1. EXISTING CONDITIONS 11
4.1.1. LAND USE & TOPOGRAPHY 11
4.1.2. SOIL CONDITIONS 12
4.1.3. GEOTECHNICAL RECOMMENDATIONS 12
4.1.4. HYDROGEOLOGY RECOMMENDATIONS 13
4.1.5. EXISTING MINOR SYSTEM DRAINAGE 13
4.1.6. EXISTING MAJOR SYSTEM DRAINAGE – AREA A 15
4.1.7. EXISTING MILL POND 16
4.1.8. EXISTING MAJOR SYSTEM DRAINAGE – AREA B 17
4.2. DEVELOPMENT CONCEPT 18
4.2.1. LAND USE & RIGHT-OF-WAYS 18
4.2.2. GRADING 19
4.3. SANITARY SERVICING 20
4.3.1. EXISTING 20
4.3.2. PROPOSED 20
4.4. WATER SERVICING 22
4.4.1. EXISTING 22
4.4.2. PROPOSED 22
4.5. STORM DRAINAGE 24
4.5.1. PROPOSED MINOR SYSTEM 24
4.5.2. PROPOSED MAJOR SYSTEM 24
4.5.3. CHANNEL REALIGNMENT / RECONFIGURED IRRIGATION POND 26
4.5.4. EXISTING & PROPOSED FLOOD ELEVATIONS 28
4.5.5. RIPARIAN STORAGE ANALYSIS 30
4.6. STORMWATER MANAGEMENT 33
4.6.1. DESIGN 33
4.6.2. QUALITY CONTROL 33
4.6.3. QUANTITY CONTROL 33

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4.6.4. WATER BALANCE 34
4.7. IMPLEMENTATION 35
4.8. NOISE AND VIBRATION FEASIBILITY STUDY 35
4.9. SUMMARY 36

5 AREA C 37

5.1. EXISTING CONDITIONS 37
5.1.1. LAND USE & TOPOGRAPHY 37
5.1.2. SOIL CONDITIONS 37
5.1.3. GEOTECHNICAL RECOMMENDATIONS 38
5.1.4. HYDROGEOLOGY RECOMMENDATIONS 38
5.1.5. EXISTING MINOR SYSTEM DRAINAGE 39
5.1.6. EXISTING MAJOR SYSTEM DRAINAGE 39
5.2. DEVELOPMENT CONCEPT 40
5.2.1. LAND USE & RIGHT-OF-WAYS 40
5.2.2. GRADING 41
5.3. SANITARY SERVICING 43
5.3.1. EXISTING 43
5.3.2. PROPOSED 43
5.4. WATER SERVICING 44
5.4.1. EXISTING 44
5.4.2. PROPOSED 44
5.5. STORM SERVICING 46
5.5.1. PROPOSED MINOR SYSTEM 46
5.5.2. PROPOSED MAJOR SYSTEM 46
5.6. STORMWATER MANAGEMENT 47
5.6.1. QUALITY CONTROL 47
5.6.2. QUANTITY CONTROL 47
5.6.3. WATER BALANCE 48
5.7. IMPLEMENTATION 49
5.8. NOISE AND VIBRATION FEASIBILITY STUDY 49
5.9. SUMMARY 50

6 AREA D 51

6.1. EXISTING CONDITIONS 51
6.1.1. LAND USE & TOPOGRAPHY 51
6.1.2. SOIL CONDITIONS 51
6.1.3. GEOTECHNICAL RECOMMENDATIONS 52
6.1.4. HYDROGEOLOGY RECOMMENDATIONS 52
6.1.5. EXISTING MINOR SYSTEM DRAINAGE 54
6.1.6. EXISTING MAJOR SYSTEM DRAINAGE 54
6.2. DEVELOPMENT CONCEPT 54
6.2.1. LAND USE & RIGHT-OF-WAYS 54
6.2.2. GRADING 55
6.3. SANITARY SERVICING 57
6.3.1. EXISTING 57
6.3.2. PROPOSED 57
6.4. WATER SERVICING 58
6.4.1. EXISTING 58
6.4.2. PROPOSED 58

6.5. STORM SERVICING 60
6.5.1. PROPOSED MINOR SYSTEM 60
6.5.2. PROPOSED MAJOR SYSTEM 60
6.6. STORMWATER MANAGEMENT 61
6.6.1. DESIGN 61
6.6.2. QUALITY CONTROL 61
6.6.3. QUANTITY CONTROL 61
6.6.4. WATER BALANCE 62
6.7. IMPLEMENTATION 63
6.8. NOISE AND VIBRATION FEASIBILITY STUDY 63
6.9. SUMMARY 64

7 CONCLUSION 65

TABLE OF TABLES

Table 2-1 Study Areas 4
Table 3-1 Residential Dry Weather Flow 5
Table 3-2 Average Daily Consumption Rates for each Land Use 6
Table 3-3 Runoff Coefficients (City of Burlington[1]) 8
Table 3-4A IDF Values for City of Burlington (2020 design standards) 8
Table 3-5B IDF Values for City of Burlington (1999 design standards) 8
Table 4-1 Existing Drainage Areas - Area A 15
Table 4-2 Average Daily Turf Irrigation Requirements 16
Table 4-3 Existing Drainage Areas - Area B 17
Table 4-4 Sanitary Servicing Criteria - Areas A and B 20
Table 4-5 Sanitary Servicing Summary - Area A & Area B 21
Table 4-6 Water Servicing Criteria and Flow Demands 22
Table 4-7 Modelled Service Pressures and Fire Flows 22
Table 4-8 Existing and Proposed Flood Elevations 28
Table 4-9A Summary of Riparian Storage Volumes 30
Table 4-10B Summary of Flood Storage Volumes 32
Table 5-1 Existing Drainage Areas - Area C 40
Table 5-2 Sanitary Servicing Criteria - Area C 43
Table 5-3 Sanitary Servicing Summary - Area C 43
Table 5-4 Water Servicing and Flow Demands 44
Table 5-5 Modelled Service Pressure and Fire Flows 44
Table 6-1 Existing Drainage Areas - Area D 54
Table 6-2 Sanitary Servicing Criteria - Area D 57
Table 6-3 Sanitary Servicing Summary - Area D 57
Table 6-4 Water Servicing and Flow Demands 58
Table 6-5 Modelled Service Pressure and Fire Flows 58

APPENDICES

Appendix A – Geotechnical Report
Appendix B – Hydrogeology Report
Appendix C – Foundation Weeping Tiles & Groundwater Quality Evaluation
Appendix D – As-Constructed Drawings
Appendix E – Sanitary Sewer Design Sheets
Appendix F – Preliminary Water Distribution Modelling Report
Appendix G – STM Calculations
Appendix H – SWM Calculations
Appendix I – Channel Re-alignment & Hydraulics
Appendix J – PTTW Application
Appendix K – Noise Study
Appendix L – Arborist Report

DRAWINGS & FIGURES

Concept Plan SK-74 (Gerrard Design)
Draft Plan of Subdivision (Wellings Planning Consultants Inc.)

Area A

SEC-A1 – Area A Road Cross Section (8.5m ROW; w/ ex. switchgear)
SEC-A2 – Area A Road Cross Section (8.5m ROW)
SEC-A3 – Area A Road Cross Section (10.6m ROW)
SEC-A4 – Area A Road Cross Section (island)
GP-A – General Plan
GR-A1 – Grading Plan
GR-A2 – Grading Plan
EX-AB – Existing Drainage Plan (Areas A & B)
STM-A – Storm Drainage Plan
SAN-A – Sanitary Drainage Plan
WM-A – Water Distribution Plan
LID-A1 – Conceptual LID Plan
LID A2 – Conceptual LID Plan
CMMP-A – Construction Management & Mobility Plan
Figure 1 – Constraint Mapping
Figure 2 – Sections
Figure 3 – Riparian Wetland Catchment Area
FP-1 – Existing Floodplain
FP-2 – Proposed Floodplain
LU-AB – Land Use (Areas A & B)

Area B

SEC-B1 – Area B Road Cross Section (10.6m ROW)
SEC-B2 – Area B Road Cross Section (island)
GP-B – General Plan
GR-B – Grading Plan
STM-B – Storm Drainage Plan
SAN-B – Sanitary Drainage Plan
WM-B – Water Distribution Plan
LID-B – Conceptual LID Plan
CMMP-B – Construction Management & Mobility Plan

Area C

SEC-C1 – Area C Road Cross Section (11.2m ROW)
GP-C – General Plan
GR-C – Grading Plan
EX-C – Existing Drainage Plan)
STM-C – Storm Drainage Plan
SAN-C – Sanitary Drainage Plan
WM-C – Water Distribution Plan
CMMP-C – Construction Management & Mobility Plan
LU-C – Land Use

Area D

SEC-D1 – Area D Road Cross Section (9.1m ROW)
GP-D – General Plan
GR-D – Grading Plan
EX-D – Existing Drainage Plan)
STM-D – Storm Drainage Plan
SAN-D – Sanitary Drainage Plan
WM-D – Water Distribution Plan
CMMP-D – Construction Management & Mobility Pla
LU-D – Land Use

1 EXECUTIVE SUMMARY

Urbantech® Consulting has been retained by Millcroft Greens Corporation to prepare a Functional Servicing Report (FSR) in support of the proposed development within the Millcroft Golf Club site in the City of Burlington, Region of Halton. The following is a summary of our conclusions for four (4) of the five (5) development Areas under consideration (Areas A & B, C and D). A separate FSR has been prepared for the 5th development area under consideration is referred to as Area E (Urbantech Consulting, March 2022). Note, reporting included under separate cover to this FSR submission consider Areas A & B as one (1) "study area" for a total of four (4) study areas.

1.1. SANITARY SERVICING

Areas A, B, C and D will connect to existing sanitary infrastructure within the existing Millcroft subdivision adjacent to the individual Area limits.

1.2. WATER SERVICING

Areas A, B, C and D will connect to existing water infrastructure within the existing Millcroft subdivision adjacent to the individual Area limits.

1.3. STORM SERVICING AND STORMWATER MANAGEMENT

The original drainage design for the Millcroft community complied with standards and studies that were in force in the 1980s. The design concepts presented in this study comply with the **current** City, Conservation and Provincial design guidelines. The proposed development parcels will rely on existing municipal infrastructure where possible and new or improved infrastructure where necessary. Following are the general drainage design principles applied to the proposed development lands.

1. New storm sewers have been sized in compliance with the City's guidelines including the 2020 rainfall (IDF) parameters.
2. All new development will include oil grit separators and CB Shields to improve water quality control.
3. The existing on-line Mill Pond will be reconfigured and taken offline from the Appleby Creek as part of the draft plan of subdivision and subsequent condominium approval processes. As part of the Mill Pond reconfiguration works, Mill Pond will continue to serve as a source for irrigation and will not be used for SWM controls. Any proposed works within CH regulated area will require a Permit pursuant to Ontario Regulation 162/06.
4. Runoff from the development parcels will be controlled to the lesser of pre-development flows (Areas A and B) or target flows (Areas C and D) that have been established relating to the original design capacity of existing storm sewer systems. The target / pre-development flows are based on the original City of Burlington IDF parameters which would have been used at the time of the subdivision design (1999 City standards which reference the original IDF parameters on Standard Drawing S-IDF).
5. Privately owned, on-site underground storage in the form of super pipes or vaults will be utilized to achieve quantity control objectives on all sites.
6. Storm sewer re-routing is proposed for the existing outfall for Parklane Crescent. The 100-year flows from the existing subdivision will be piped in a new storm sewer to the proposed irrigation pond in Area A.
7. The existing storm sewer that serves Hadfield Court will be intercepted and augmented to provide service to Areas A and B without increasing the hydraulic grade line within Hadfield Court.

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8. Where existing drainage paths are altered by the proposed development, measures have been taken to maintain existing flows to existing outlets. These measures include 100-year capture pipes and diversion swales. Flows to existing rear yards and municipal sewers will **not** increase.

1.4. GRADING

The proposed grading meets the City's requirements for private site development. Existing grades will be maintained along the property lines. A 6.0m buffer has been provided in addition to the minimum lot depth between existing and proposed lots. Some transitional grading is proposed within these 6.0m buffers. A maximum 10% slope is used within the 7.5m development setback to the proposed floodplain limits within Area A. Overall, the proposed grading design minimizes the use of retaining walls and will be optimized to minimize earthworks import/export where practical.

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2 INTRODUCTION

2.1. BACKGROUND

This report supports planning applications for the proposed residential development Areas (A-D) located at 2155 Country Club Drive within the Millcroft Golf Club site in the City of Burlington, Region of Halton (hereafter referred to as the subject property). This report fulfils the submission requirements and addresses City of Burlington & Conservation Halton comments related to grading, servicing, drainage, and stormwater management. The servicing and development strategies presented in this report have been developed in conjunction with the greater consulting team and should be considered in conjunction with their work. The following studies are included in the appendices:

- Geotechnical Investigation Reports– DS Consultants Ltd. (DSCL, December 2020);
- Proposed – DS Consultants Ltd. (DSCL, March 2022)
- Hydrogeological Assessment – R.J Burnside & Associates Ltd. (RJB, November 2020);
- Revised Water Balance Calculations - R.J Burnside & Associates Ltd. (RJB, March 2022);
- Foundation Weeping Tiles and Groundwater Quality Evaluation - R.J Burnside & Associates Ltd. (RJB, February 2022);
- Preliminary Water Distribution Modelling Report – Municipal Engineering Solutions (MES, March 2020);
- Noise and Vibration Feasibility Study – Howe Gastmeier Chapnik Limited (HGC, February 2022); and
- Excerpt from Urban-Area Flood Vulnerability, Prioritization & Mitigation Study – Amec Foster Wheeler Environment & Infrastructure (AMEC, July 2017)
- Arborist Report & Tree Protection Plan - Beacon Environmental (Beacon, March 2022)

In addition to the above, the findings of this study have been incorporated into the Benefit Analysis Report by Jennifer Lawrence & Associates (JLA, 2022) and Environmental Evaluation Report (EER) by Beacon Environmental (Beacon, 2022), both included under separate cover in this submission. This study presents the recommended stormwater management and municipal servicing scheme for the development of the subject property. This report is also applicable for any future revisions to the plan of subdivision, assuming the revisions are minor and in general conformance with the concepts outlined herein.

The information presented in this report conforms to the following guidelines:

- City of Burlington (City) Standard Specifications (March 2020)
- City of Burlington Stormwater Management Design Guidelines (May 2020)
- Conservation Halton (CH) Policies and Guidelines for the Administration of Ontario Regulation 162/06 and Land Use Planning Policy Document (November 2020)
- Conservation Halton (CH) Guidelines for Stormwater Management Engineering Submissions (November 2021)
- Regional Municipality of Halton (Region) Water and Wastewater Linear Design Manual (October 2019)
- Ministry of the Environment, Conservation and Parks (MECP) Stormwater Management Planning and Design Manual (March 2003)

2.2. SUBJECT PROPERTY

The subject property comprises of 3 distinct study areas for the overall FSR and total approximately 11.7 ha in size. The site currently consists mainly of golf course lands (Millcroft Golf Club). The individual study

areas are shown in the Preliminary Development Concept by Gerrard Design (Gerrard) as **Figure SK-74** and are identified in this report as follows (note that Areas A and B have been grouped into one study area as they have common servicing outlets and drainage interdependences):

Table 2-1 Study Areas

Study Area	Area	Current Land Use	Location
Areas A & B	9.48 ha	Hole 6 & 7	East side of Millcroft Park Drive, south of Parklane Crescent and north of Country Club Drive and Upper Middle Road
Area C	1.46 ha	Hole 1	North side of Country Club Drive, east of Berwick Drive
Area D	0.76 ha	Hole 16	East side of Millcroft Park Drive, south of Latimer Crescent and north of Turnberry Road

The subject property is generally bounded by:

- Dundas Street to the north; and,
- Appleby Line / CNR tracks to the east; and,
- Walkers Line to the west; and,
- Upper Middle Road to the south.

3 DESIGN CRITERIA AND OBJECTIVES

3.1. SANITARY DESIGN CRITERIA

The sanitary sewer flows for the subject lands site will be calculated using the following criteria outlined in the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019):

Design Flow (L/s) = Average Dry Weather Flow x M + Inflow/Infiltration Allowance Eq. [1]

Table 3-1 Residential Dry Weather Flow

Type of Development	Equivalent Population Density (people/ha)	Equivalent Population Density (People/Unit)	Average Dry Weather Flow (L/capita/d)
Single Family	55	3.52	275
Semi-detached duplex and 4-plex	100	3.52	
Townhouse, Maisonette (6 storey apt. or less)	135	2.67	
Apartments (over 6 stories high)	285	1.58	

Due to the relatively small size of the development Areas, the precise number of units is known. The proposed development will consist of single-detached homes of comparable size to those in the existing Millcroft subdivision. Based on measurements of several one (1) hectare areas in the existing subdivision, approximately 15 homes occupy a 1.0-hectare area. Assuming 55 people / ha, this results in a population per unit (PPU) of approximately 3.52 as calculated by MES in **Appendix F** (March 2020) for the purposes of the water distribution analysis. This assumption can be used for sanitary design as well. The following design criteria are used to size the sanitary sewers:

- Infiltration / Inflow – 0.286 L/s/ha (minimum for all land use types)
- Average Domestic Flow = 275 L/c/day
- Minimum Velocity = 0.60 m/s
- Maximum Velocity = 3.00 m/s
- Factor of Safety = 15%

Residential Peak Wastewater Flow Factor (M): $M = 1 + 14 / (4 + P1/2)$

Where:

- M = Harmon peaking factor Eq. [2]
  - Minimum factor = 2.00
  - Maximum factor = 4.50
- P = Population in thousands

In 2017, the Region of Halton provided results from their InfoSewer model (2016 and 2031 population and infrastructure scenarios). The Region's summary indicated the reserve capacity in the system downstream of the proposed development areas. In all cases, the sanitary sewers downstream of the connection points have at minimum 100 L/s reserve capacity, up to over 340 L/s at the trunk downstream of the subject lands. All sewers are approximately 50% full (depth) at the connection points. Sanitary capacity is not a constraint to the development of the subject lands. Refer to **Appendix E** for the sanitary capacity analysis report.

3.2. WATER DESIGN CRITERIA

The watermain distribution for each of the development phases will be calculated using the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019)

Table 3-2 Average Daily Consumption Rates for each Land Use

Type of Development	Equivalent Population Density (people/ha)	Equivalent Population Density (People/Unit)	Average Dry Weather Flow (L/capita/d)
Single Family	55	3.52	15.125
Semi-detached duplex and 4-plex	100	3.52	27.5
Townhouse, Maisonette (6 storey apt. or less)	135	2.67	37.125
Apartments (over 6 stories high)	285	1.58	78.375
Community Services	40	-	11.00

Water Supply System Pressures:

- Maximum sustaining operating pressure – 700 kPa
- Minimum pressure under simultaneous maximum day and fire flow demands – 140 kPa
- Minimum pressure under normal operating conditions – 275 kPa

If the static pressure exceeds 690 kPa, a pressure reducing valve must be installed to limit the maximum static pressure so it does not exceed 690 kPa and areas that may be occupied. Further, in accordance with Ontario Building Code (OBC) 7.6.3.3 and where the static pressure at fixture may exceed 550 kPa, a pressure reducing valve conforming to OBC 7.2.10.12 shall be installed to limit the maximum static pressure at the fixture to 550 kPa.

3.3. STORM DESIGN CRITERIA

New storm sewers are to be designed using the criteria outlined in the City of Burlington Stormwater Management Design Guidelines (May 2020):

Determining Return Period Storm Event Flow (Q):

$$Q = K \times C \times I \times A$$

Where:

A = Contributing Drainage Area (ha) Eq. [3]
Q = Return Period Storm Event Flow (m3/s)
K = Conversion Factor (0.00278)
C = Runoff Coefficient
I = Rainfall intensity (mm/hour)

Determining Rainfall Intensity (I):

$$I = A / (t_d + B)^C$$

To be calculated as per the City of Burlington's approved IDF equation where:

Eq. [4]

t_d = time duration (minutes)

*Refer to **Table 3-4** for all A, B & C constants.*

Runoff Coefficient (C):

- In urban areas, the runoff coefficient will be increased for high magnitude storms
- Runoff coefficients should be multiplied by the Antecedent Precipitation Index (Ca) for the 25, 50 and 100-year storms (maximum C x Ca is 1.00)
    - 25-year storm = 1.10
    - 50-year storm = 1.20
    - 100-year storm = 1.25

In accordance with the City standards, the minimum Time to Concentration (Tc) of 10 minutes shall be used. The actual expected time of concentration is to be calculated based on the contributing drainage area to the point of interest. Runoff coefficients for the City of Burlington are listed in **Table 3-3** below.

Table 3-3 Runoff Coefficients (City of Burlington[1])

Area Type	Runoff Coefficient (C)
Where Impervious/Pervious Areas are directly measured:	
Impervious Areas (Asphalt, Concrete Rooftop, and Water Surfaces)	0.95
Gravel Surfaces	0.80
Permeable Pavement	0.80[1]
Pervious Areas (Grass, Meadow, Forest)	0.25[2]
For Use in Design Without Detailed Site Plans/Plot Plans:	
Parks	0.35
Commercial	0.90
Industrial	0.90
Institutional (Schools and Churches)	0.75
Greenfield Residential:	
Single Family (under 30 ft width)	0.66
Single Family (between 30 and 50 ft width)	0.60
Single Family (greater than 50 ft width)	Consult City Staff
Semi-Detached	0.70
Row Housing, Town Houses	0.75
Apartments	0.85
Infill Residential	Calculated from proposed plans

[1]City of Burlington Stormwater Management Design Guidelines (May 2020)
[2]May vary depending on type of application; to be reviewed and confirmed with City of Burlington

A summary of the IDF values and intensities (assuming a 10-minute time of concentration) for the City of Burlington are listed in **Table 3-4A and Table 3-4B** below:

Table 3-4A IDF Values for City of Burlington (2020 design standards)

Variable	Type	2 Year	5 Year	10 Year	25 Year	50 Year	100-year
a =	Constant =	681.52	802.04	918.28	1065.95	1172.34	1281.34
b =	Constant =	6	5	5	5	5	5
c =	Constant =	0.780	0.764	0.763	0.762	0.761	0.761
t =	Time of Conc.(min) =	10	10	10	10	10	10
I =	Intensity (mm/hr)	78.39	101.31	116.31	135.38	149.30	163.18

Table 3-5B IDF Values for City of Burlington (1999 design standards)

Variable	Type	2 Year	5 Year	10 Year	25 Year	50 Year	100-year
a =	Constant =	592.60	697.40	798.50	926.90	1019.40	1114.20
b =	Constant =	6	5	5	5	5	5
c =	Constant =	0.780	0.764	0.763	0.762	0.761	0.761
t =	Time of Conc. (min) =	10	10	10	10	10	10
I =	Intensity (mm/hr)	68.16	88.09	101.14	117.72	129.82	141.89

Note that the target flows for areas draining to existing storm sewer systems are based on the previous standards (1999) / City Standard Drawing S-IDF. It was assumed that the capacity of the existing system was based on the older IDF parameters, and that new infrastructure should be designed according to the updated IDF parameters.

3.4. STORMWATER MANAGEMENT DESIGN CRITERIA

The latest City of Burlington Stormwater Management Design Guidelines, prepared by Wood in May of 2020, states that:

"*Proponents are thus required to provide SWM control in accordance with the specified level of control outlined within a Watershed Study, Subwatershed Study (SWS), Master Drainage Plan (MDP), or Master Environmental Servicing Plan (MESP) that encompasses the proponent's site.*"

While the Tuck, Shoreacres, Appleby and Sheldon Creeks Watershed Study (WS) prepared by the City of Burlington in June 1985 recommends that quality, quantity and erosion controls are not required, the City's current standards and comments received to date will take precedence with respect to SWM controls.

With respect to discharge of post-development flows into existing sewer systems, adequate storage / flow control will be provided where necessary to ensure that the existing flows in the storm sewer system are not exceeded. In general, discharge to the existing storm sewer system will be limited to the 5-year pre-development flow (calculated using the 1999 IDF parameters). In most cases, this involves over control of the 100-year post development flow (calculated based on 2020 IDF parameters) to the 5-year pre-development flow. Discharge directly to Appleby Creek will be designed on a post- to pre-basis.

Site recharge / water balance considerations have been considered in the 2020 hydrogeological study by RJB included in **Appendix B** as detailed in **Section 4.1.4**. Specific findings and recommendations are noted in the individual Area summaries in the following sections.

3.5. GRADING DESIGN OBJECTIVES

In general, grading shall be completed to safely direct surface water flows towards safe outlets. Safe outlets may consist of surface drainage features (i.e., overland flow routes, storm sewers, etc.) or creek systems. The following outlines the requirements from the latest City of Burlington Stormwater Management Design Guidelines, prepared by Wood (May 2020):

- *All developments (excluding single family residential) shall design site grading to provide an overland flow route to an acceptable receiver, generally the roadway right-of-way. All grading and drainage shall consider the City of Burlington By-Law 52-2018 (A by-law to regulate the altering of grades or drainage on Low Density Residential Lands). Reference is also made to City Standards S-118 (Lot Grading Standards, Exclusive of Current Subdivision) and S-119 (Lot Grading Standards for Subdivision Lots). Outside grades shall be a minimum of 0.15 m below the top of the foundation wall.*

- *Site grading is to be completed to facilitate a positive slope away from buildings and other features that may be at risk of damage caused by flooding. Lot grading shall be completed to provide grades between 2% - 5% away from the building foundation for a minimum apron length of 5 m. Reduced lot grades (i.e., less than 2%) may be considered by the City of Burlington as part of an overall stormwater management (SWM) system in order to slow overland flows and promote infiltration. Steeper lot grades may be considered by the City of Burlington where constrained, to a maximum slope of 3H:1V.*

- *Lot grading for development sites shall be self-contained and prevent spills onto adjacent properties, while also blending into existing topography and grades surrounding the site. Where external areas drain through a subject property, the proponent shall be required to design on site grading and drainage systems to safely convey these flows (up to the 100-year post development condition) and prevent ponding or negative impacts to the adjacent properties.*

- *For residential lots, the City of Burlington generally supports both back to front and split drainage. Where split drainage is proposed, a rear-yard drainage system must be designed to the satisfaction of the City of Burlington.*

- *There shall be a 0.6 m wide pathway (at a 2% cross fall slope away from the foundation) around one side of the building to allow access to the rear yard area, except where side yard setbacks from lot lines do not permit such a pathway.*

- *Driveways must slope away from the building with positive drainage towards the roadway right-of-way, with slopes of between 1% (minimum) and 5% (maximum). Reverse sloped driveways are strongly discouraged, unless no other feasible grading solution exists. Where a reverse sloped driveway is proposed, a complete hydraulic grade line (HGL) analysis will be required by a qualified Professional Engineer (P.Eng.) to demonstrate no negative effects for all storm events up to and including the 100-year storm.*

- *Upon completion of the development works, the owner shall supply the City of Burlington with an as-built grading plan, stamped by a qualified Civil Engineer (P.Eng.), certifying that the grading conforms to the approved grading plans and that there are no adverse impacts on neighboring properties.*

4 AREAS A & B

LOCATION: East side of Millcroft Park Drive, south of Parklane Crescent and north of Country Club Drive and Upper Middle Road

AREA: 9.48 ha

CURRENT LAND USE:
Holes No. 6 & 7



4.1. EXISTING CONDITIONS

4.1.1.LAND USE & TOPOGRAPHY

The site is currently being used as Holes 6 & 7 for the Millcroft Golf Club. The golf course will remain operational, and the two Holes will be re-designed to accommodate future development. Currently, the site is covered with greens/fairways/and bunkers, low lying vegetation and trees, an irrigation pond, and golf cart trails. A reach of Appleby Creek also traverses the southwest side of the site.

Topographical surveys of the subject lands have been completed by Rady-Pentek & Edward Surveying Ltd. ('R-PE') in August 2017 and October 2018.

The site generally falls from northeast to the southwest with a maximum grade change of approximately 5 m across both Areas.

Figure EX-AB *illustrates the existing site features, topography, and drainage patterns.*

4.1.2. SOIL CONDITIONS

DS Consultants Limited ('DSCL') was retained by Millcroft Greens Corporation to undertake the preliminary geotechnical investigation for the proposed development. The geotechnical report indicates the following subsurface conditions:

- Topsoil – 130 mm to 350 mm thick surficial layer of topsoil
- Fill material – ranging from 0.8 m to 1.9 m below grade consisting mainly of clayey silt, trace rootlets and trace topsoil.
- Upper soils – Clayey silt to sandy silt till
- Lower soils – Clayey silt till
- Bedrock encountered at depths ranging from 1.5 mbgs (adjacent to mill pond) to 6.1 mbgs (adjacent to Millcroft Park Drive in Area B)

DSCL's preliminary investigation indicated that the soil stratigraphy of the subject property consists of fill underlain by native deposits of clayey silt, sandy silt till, clayey silt till and a shale complex overlying shale bedrock. The report also establishes that no groundwater was found in the boreholes during drilling or upon completion of drilling except for BH18-7, where the groundwater level was measured at 2.1 m below existing grade. The stabilized groundwater levels observed in the monitoring wells installed in the boreholes range from depths below ground surface of 1.1-1.4m (BH18-2) to 4.2-4.3m (BH18-1).

Highly weathered shale bedrock of the Queenston Formation was encountered below depths ranging from 1.5 to 6.1m, however was not proven through core drilling. Methane gas is anticipated in the bedrock, and appropriate care and monitoring is essential in all confined bedrock excavations. For the purposes of this investigation, it is assumed that any remediation work which may be required to deal with potential contamination will be completed prior to earthworks and servicing. The complete study in **Appendix A** provides a detailed discussion of geotechnical site conditions. The report further discusses recommendations pertaining to site alteration procedures as well as the construction of roads, sewers, foundations, and drainage systems based on the subsoil conditions.

4.1.3. GEOTECHNICAL RECOMMENDATIONS

The geotechnical report recommended the following minimum pavement makeup for all internal site roads based on the assumed residential minor local or local traffic usage conditions:

- 40 mm HL3 Asphaltic Concrete
- 50 mm HL8 Asphaltic Concrete
- 150 mm Granular 'A'
- 350 mm Granular 'B'

The Area A proposed design grade at road surface will be at Elevation 140 to 141m. Assuming basement depth of 3m below the ground surface, the basement floors will be at about Elev. 137 to 138m. Bedrock elevation in the boreholes varies from 135.6 to 136.3m as per the geotechnical report prepared by DS Consultants Ltd in **Appendix A**.

The Area B proposed design grade at road surface will be at Elevation 141.5 to 142.4m. Assuming basement depth of 3m, the basement floors will be at about Elev. 138.5 to 139.4m. Bedrock elevation in the boreholes varies from 136 to 138.3m as per the geotechnical report prepared by DS Consultants Ltd in **Appendix A**.

Area A & Area B basements floor will generally be constructed in glacial till deposits. However, in any areas where basements are to be constructed below the bedrock surface, methane gas monitoring should be carried out during excavation. If methane gas is encountered during excavation, gas venting systems may be required depending upon the gas concentration readings.

*Refer to **Appendix A** for the Geotechnical Report for Areas A through D as completed by DSCL (December 2020).*

4.1.4. HYDROGEOLOGY RECOMMENDATIONS

A hydrogeological report investigation was completed by RJB in November 2020 and has been included in **Appendix B**.

The field and laboratory work included the installation of seven monitoring wells across the subject property (three in Area A and four in Area B) and a piezometer in Appleby Creek to monitor shallow groundwater levels and evaluate the potential groundwater/surface water interaction. Groundwater levels across Areas A and B have been measured monthly from January 2019 through to November 2021 and will continue to be monitored quarterly thereafter.

A review of the groundwater level data suggests the following:

- Groundwater levels in monitoring wells were observed to have varied about 136m above sea level (masl), with seasonal fluctuations ranging from 0.5m to 4.0m range.
- Shallow groundwater levels typically exhibit seasonal fluctuations, with the highest elevations generally observed in the spring and the lowest in the summer months. To date, seasonal high levels were observed in January 2020 due to the mild winter conditions.

*Refer to **Appendix B** for the Hydrogeological Report Investigation completed by RJB (November 2020).*

A conventional foundation weeping tile drain system and sump pumps for the proposed detached lots in Areas A & B will address seasonal high groundwater elevations and hydraulic gradeline. Based on the Foundation Weeping Tiles and Groundwater Quality Evaluation by RJB provided in **Appendix C**, groundwater flows in Areas A & B are estimated as 4.8×10^{-5} m^3/s & 6.6×10^{-5} m^3/s, respectively. These groundwater flows are seven (7) orders of magnitude less than the 100-year flows that will be captured and conveyed to each respective minor system. These flows will not impact the allowable release rate or storage volumes. Groundwater samples were collected and submitted for analysis to compare to the City's storm sewer discharge criteria. The results of the groundwater analysis in comparison to the City's storm sewer discharge criteria and discussion are presented by RJB in **Appendix C**. For discussion on the estimated groundwater flow impacts to the minor storm sewer system, refer to **Section 4.5.1**.

*Refer to **Appendix C** for the Foundation Weeping Tiles & Groundwater Quality Evaluation completed by RJB (February 2022).*

4.1.5. EXISTING MINOR SYSTEM DRAINAGE

An existing 675mm storm sewer traverses an easement from Parklane Crescent through the eastern limit of Area B to a 750mm storm sewer that passes through an easement to Hadfield Court. This storm sewer system travels west to an 825mm storm sewer that navigates through another easement that ultimately outlets at the existing pond at Upper Middle Road and Country Club Drive. A rear yard catchbasin ('RYCB') located at the south limit of Area B collects overland flow from the rear yards of houses on Parklane Crescent and approximately half of Area B. This RYCB is connected to the existing storm sewer within the Hadfield Court ROW with a 300mm pipe. Another RYCB located at the southeast limit of Area

B collects flows from the rear-yards of Parklane Crescent and the remaining drainage from Area B and is connected to the storm sewer that traverses Area B. Refer to **Figure EX-AB** for details.

4.1.6. EXISTING MAJOR SYSTEM DRAINAGE – AREA A

Areas A & B generally drains north to south, ultimately making its way to the existing pond at the southwest corner of Area A. Note, Area A is separated from Area B by an existing residential development (Hadfield Court).

Existing drainage from Area A generally drains from the northeast to southwest with an average slope of approximately 1.0%. Area A has been divided into five (5) existing drainage sub-catchments and associated with five (5) external existing drainage sub-catchments as depicted in **Figure EX-AB,** summarized in **Table 4-1** and further discussed below.

Table 4-1 Existing Drainage Areas - Area A

Area A			External to Area A		
Area ID	Area (ha)	RC (-)	Area ID	Area (ha)	RC (-)
S-01A	1.60	0.30	E-01A	0.31	0.40
S-02A	2.56	0.30	E-02A	0.46	0.40
S-03A	0.40	0.30	E-03A	0.37	0.40
S-04A	0.27	0.30	E-04A	0.08	0.40
S-05A (Mill Pond)	1.00	0.95	E-05A	3.11	0.40
Total	5.83	0.41	Total	4.33	0.40

- Catchment S-01A currently sheet drains south overland to the adjacent watercourse, Appleby Creek.
- Catchment S-02A currently sheet drains south overland to the existing pond, online with Appleby Creek.
- Catchment S-03A currently sheet drains north overland to the adjacent watercourse, Appleby Creek.
- Catchment S-04A currently sheet drains north overland to the existing pond, online with Appleby Creek.
- Catchment S-05A is the existing Mill Pond, online with Appleby Creek.

The total existing drainage from Area A is 5.83 ha, with an average runoff coefficient of 0.41.

There are five (5) external drainage sub-catchments that are conveyed through Area A.

- Catchment E-01A consists of existing rear lot residential along Hadfield Court, draining directly overland to Catchment S-01A.
- Catchment E-02A consists of existing rear lot residential along Hadfield Court, draining directly overland to Catchment S-02A.
- Catchment E-03A consists of existing rear lot residential along Country Club Drive, draining directly overland to Catchment S-03A.
- Catchment E-04A consists of existing rear lot residential along Country Club Drive, draining directly overland to Catchment S-04A.
- Catchment E05A consists of existing residential lots along Hadfield Court, draining major system overland flow to Catchment S-02A via a servicing easement.

The total external drainage conveyed through Area A is 4.33 ha with an average runoff coefficient of 0.40.

*Please refer to the existing site drainage for Area A as depicted in **Figure EX-AB** and calculations in **Appendix H**.*

4.1.7. EXISTING MILL POND

Located in the south-eastern corner of Area A, adjacent to Upper Middle Road, the Mill Pond is an online pond that receives drainage from the surrounding residential development and the East Branch of Appleby Creek. Note, Appleby Creek is classified as a second order watercourse and flows through three additional online ponds located upstream of the Mill Pond. Appleby Creek flows openly into the Mill Pond via an engineered channel. A concrete weir structure is located at the downstream end of the Mill Pond (at the Upper Middle Road crossing), which controls the normal water level in the pond to 136.30 m. The pond is approximately 100 m wide and 150 m long, with a storage volume of approximately 67,500 m^3 (below the weir crest elevation). The drainage area upstream of the Mill Pond is estimated at 43.91 ha, which includes the Appleby Creek drainage area as well as areas captured by the existing storm sewer drainage network.

The existing golf course uses the Mill Pond for irrigation. The current irrigation distribution network irrigates all greens, tee, fairways, and designated portions of rough as illustrated in **Appendix J**. RJB was retained by Millcroft Golf Club to prepare / secure a renewal of the Permit to Take Water (PTTW) from the MECP. The existing irrigation needs are summarized in **Table 4-2** below as per the PTTW Renewal Application by RJB included in **Appendix J**.

Table 4-2 Average Daily Turf Irrigation Requirements

	Avg.	Time Period (months)							
		APR.	MAY	JUN.	JUL.	AUG.	SEPT.	OCT.	NOV.
mm/day	1.565	0.181	0.907	1.814	3.629	3.629	1.814	0.363	0.181
gal/day	44,059	5,108	25,542	51,083	102,166	102,166	51,083	10,217	5,108
L/day	200,238	23,000	116,100	232,200	464,500	464,500	232,200	46,400	23,000
m^3/day	~200	~23	~116	~232	~465	~465	~232	~46	~23

*Please refer to **Appendix J** for the Permit to Take Water information & Existing Irrigation Line Network.*

4.1.8. EXISTING MAJOR SYSTEM DRAINAGE – AREA B

The existing drainage from Area B generally drains from the northeast to southwest with an average slope of approximately 1.0%. Area B has been divided into two (2) existing drainage subcatchments and associated with five (5) external existing drainage areas as depicted in **Figure EX-AB**, summarized in **Table 4-3** and further discussed below.

Table 4-3 Existing Drainage Areas - Area B

Area B			External to Area B		
Outlet #1B (Ex. RYCB)					
Area ID	Area (ha)	RC (-)	Area ID	Area (ha)	RC (-)
S-01B	1.26	0.30	E-02B	0.26	0.40
			E-04B	0.19	0.40
			E-05B*	5.74	0.40
Total	1.26	0.30	Total**	0.45	0.40
Outlet #2B (Ex. RYCB)					
S-02B	2.36	0.30	E-01B	0.30	0.40
			E-03B	0.36	0.40
Total	2.36	0.30	Total	0.66	0.40
Total	**3.62**	**0.30**	**Total**	**6.85**	**0.40**

*Minor & Major system discharge is only routed through Catchment S-01B and ultimately discharges to Mill Pond.
**E-05B Catchment is not included in external area total for Outlet #1B.

- Catchment S-01B currently sheet drains overland to a rear-yard catchbasin located at the rear of the existing development located at 2043 Hadfield Court (as Outlet #1B**)**.
- Catchment S-02B currently sheet drains overland to a rear-yard catchbasin located at the rear of the existing development located at 2061-2063 Hadfield Court (as Outlet #2B).

The total existing drainage area from Area B is 3.62 ha with an average runoff coefficient of 0.30. There are five (5) external drainage subcatchments that are conveyed through Area B.

- Catchment E-01B consists of existing rear lot residential, draining directly overland to Catchment S-02B, ultimately being captured by Outlet #2B.
- Catchment E-02B consists of existing rear lot residential, draining directly overland to Catchment S-01, ultimately being captured by Outlet #1B.
- Catchment E-03B consists of existing rear lot residential, draining directly overland to Catchment S-02, ultimately being captured by Outlet #2B.
- Catchment E-04B consists of existing rear lot residential, draining directly overland to Catchment S-01, ultimately being captured by Outlet #1B.
- Catchment E-05B consists of existing residential lots along Parkland Crescent, draining major system overland flow via swale through Catchment S-01B & lands abutting CNR corridor from existing servicing easement on Parklane Crescent. The minor system for Catchment E-05B is piped to a servicing easement on Hadfield Court. Note, both minor and major system ultimately discharge to Mill Pond.

The total external drainage area conveyed through Area B is 6.85 ha with an average runoff coefficient of 0.40. *Please refer to the existing site drainage for Area B as depicted in **Figure EX-AB** and calculated in **Appendix H**.*

4.2. DEVELOPMENT CONCEPT

4.2.1. LAND USE & RIGHT-OF-WAYS

As shown conceptually in **Figure SK-74** & calculated in the **Draft Plan of Subdivision**, by Gerrard Design & Wellings Planning Consultants Inc. respectively, the proposed Area A & Area B developments consist of the following land use components:

- Detached Homes: Lots 1-29 (Area A) and Lots 30-69 (Area B)
- Linear Park: Block 98 (Area A) and Block 97 (Area B)
- Open Space: Blocks 99 & 100 (Area A)
- CN Berm Area: Block 95 (Area A) and Block 96 (Area B)
- Right-of-Ways (ROW): Block 101 (Area A) and Block 102 (Area B)
- Servicing Easements: Blocks 92 & 93 (Area A) and Block 94 (Area B)
- Appleby Creek / Irrigation Pond: Block 91

The proposed developments will be advanced through the draft plan of subdivision and subsequent condominium approval processes.

Area A will consist of 29 residential-detached lots, and Block 101 includes a 10.6 m ROW with 8.1 m of pavement width and 8.5m & 7.0m ROWs with 6.0m pavement widths as illustrated in road cross-sections **SEC-A1** to **SEC-A4**. The 10.6m ROW includes, but is not limited to, a 1.5m sidewalk, 6.0m two-way travel lanes and a 2.4m parking lane as illustrated in **SEC-A3**. A portion of Block 101 is split by an island that varies in width as flanked by 7.0m and 8.5m ROWs illustrated in road cross-section **SEC-A4**. The 8.5m ROW to the south of the island includes a 2.4m parking lane and 1.5m sidewalk. A portion of Block 101, closest to the intersection with Millcroft Park Drive, consists of an 8.5m ROW that does not include a parking lane as illustrated in **SEC-A1** & **SEC-A2**. Refer to the Traffic Impact Study by C.F. Crozier & Associates Consulting Engineers submitted under separate cover with this submission as it relates to the placement of a guide rail between the sidewalk & retaining wall. Generally, residential-detached lot depths are approximately 28.0 m to 34.0m along with widths of 15.24 m or 18.30 m. The non-developable area consists of the existing Appleby Creek valley system as part of Block 91, the proposed low flow channel reinstatement, the existing online irrigation pond (to be taken offline) as part of Block 91, CN Rail Berm area (located along the east side of the site, parallel to the railway) as Block 95 and Linear Park as Block 98. Open Space Block 99, as referenced in **SEC-A1** through **SEC-A3**, interfaces the various ROW & development limits and includes a 1.2m high decorative fence along the top of a 1.2m high retaining wall as discussed further in **Section 4.5.3**. Open Space Block 100 contains a proposed underground storage tank as discussed further in **Section 4.6.3**

Area B will consist of 40 residential lots and Block 102, including a 10.6m right-of-way width with 8.1 m pavement width as illustrated in road cross-sections **SEC-B1**. Similar to Area A's Block 102, a portion of Block 102 is split by an island that varies in width as flanked by 7.0m and 8.5m ROWs illustrated on road cross-section **SEC-B2.** The 8.5m ROW to the south of the island includes a 2.4m parking lane and 1.5m sidewalk. Residential-detached lot depths range from 28 m to 40 m, with widths of 15.24 m to 18.30 m.

The non-developable areas consists of a linear park as Blocks 97 (Area B) & 98 (Area A) and CNR berm area as Block 96 (Areas A & B).

Note, the above-mentioned ROWs within Area A's Block 101 and Area B's Block 102 have been developed in keeping with the master plan vision to support the proposed surface treatment and will be further refined in consultation with the required approval agencies & utility companies. As such, the cross-sections are conceptual and are to be discussed further with the City of Burlington. Note, the ROW cross-section was developed in consultation with RTG Systems Inc with respect to utilities.

*Refer to **Drawings GP-A & GP-B** for Area A and Area B general plans.*

Note, many of the detached lots include a 6.0 m buffer within the rear yards for the proposed residential lots adjacent to the existing residential development in Areas A and B. This buffer will be landscaped to act as a privacy feature between existing and proposed residential lots and is illustrated in cross-section view by NAK Design Strategies as part of the Conceptual Open Space package submitted under separate cover with this submission. Existing trees within the 6.0m buffer will be preserved wherever possible as illustrated in Arborist Report Figures TP-1 & TP-2 by Beacon included in **Appendix L**. Note, this tree preservation plan is included in **Drawings GR-A1, GR-A2 & GR-B**. For details on ownership and condition ratings, refer to Arborist Report Figures TP-1 & TP-2.

*Refer to **Appendix L** for the Arborist Report for Areas A and B.*

4.2.2. GRADING

The proposed conceptual grading for the development will be designed in accordance with City of Burlington standards. Grading is generally governed by the existing boundary conditions. Site grading has also been designed to ensure that adequate cover over proposed services is maintained. No external grading works are proposed.

A preliminary grading concept plan has been prepared for the subject lands based on the following engineering constraints:

- Storm outlet elevations
- Major system drainage paths
- Provision of minimum cover over services
- Proposed road patterns and land use
- Elevations along boundary roads, property lines and the CNR slopes
- Floodplain impacts and riparian storage requirements
- Environmental features and hazards (i.e., wetlands and meander belt within channel) and their associated buffers
- Application of the City of Burlington standards

The grading plans, **Drawings GR-A1, GR-A2 & GR-B**, is consistent with the City standards. In general, grading of all proposed roads and blocks adjacent to the surrounding development and roads matches the existing grades. Additionally, significant effort has been exercised to ensure that the grades of the proposed lands transitions gradually from northeast to southwest and that proposed grades minimize impacts to existing trees within the 6m buffer adjacent to existing properties. Note, trees for preservation and associated protection fencing by Beacon is included on **Drawings GR-A1, GR-A2 & GR-B** as per Drawing TP-1 included in **Appendix L**. Some grading within the proposed 6.0m buffers may be necessary to provide for efficient drainage and to transition from existing to proposed conditions. Grading with the buffers will be restored with appropriate ground cover & trees. The Preliminary Landscape Design for Area A & Area B, as prepared by NAK Design Strategies included under separate cover in this submission, is refleced on **Drawings GR-A1, GR-A2 & GR-B**.

*Refer to **Drawings GR-A1, GR-A2 & GR-B** for Area A and Area B grading plans.*

4.3. SANITARY SERVICING

The sanitary servicing criteria for Areas A and B is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 4-4**.

Table 4-4 Sanitary Servicing Criteria - Areas A and B

Parameter	Value	Parameter	Value
Average Domestic Flow	275 L/capita/day	Population (Single Family)	3.52 person per unit
Infiltration / Inflow	0.286 L/s/ha	Unit Count (#)	29 (Area A)
Peaking Factor (M)	Harmon Formula		40 (Area B)
Minimum	4.50	Area (ha)	2.26
Maximum	2.00		3.26

4.3.1. EXISTING

The nearest existing sanitary infrastructure to the subject property is a 525 mm sewer on Millcroft Park Drive which drains to the existing 675mm diameter sewer on Corporate Drive. Flows are then conveyed further south across the QEW to the South Service Road and Harvester Road sewers west to Walker's Line. Flows are ultimately conveyed southwest to the Skyway Wastewater Treatment Plant via an 1800mm diameter trunk sewer.

4.3.2. PROPOSED

Based on a review of the available as-constructed information (refer to **Appendix D**), the subject lands can be serviced by gravity sewers. Wastewater infrastructure will be designed in accordance with the latest Region of Halton standards and specifications.

The proposed sanitary sewers that will service the subject lands will discharge to the existing 525mm sanitary sewer on Millcroft Park Drive. Installation of a new sanitary manhole will be required on the existing sanitary sewer on Millcroft Park Drive at the intersection with the North Road (Area B). The sanitary sewer servicing the South Road (Area A) units will connect directly to the existing manhole south of the intersection with Hadfield Court.

Sanitary sewer design sheets have been prepared and used to size proposed sanitary sewers for the proposed development. The population is estimated using the population equivalent densities per the Halton Region standards. Population Estimates and Sanitary Design Sheets can be found in **Appendix E.** See **Table 4-5** for a summary of the sanitary servicing for Areas A & B.

Table 4-5 Sanitary Servicing Summary - Area A & Area B

Area	Sanitary Area (ha)	# of Units	Population Density (ppu) / Population		Dry Weather Flow (L/s @ 275 L/c/d)	Infiltration / Inflow (L/s @ 0.286 L/s/ha)	Peaking Factor	Total Sanitary Flow (L/s)	Ex. MH
A	2.26	29	3.52	103	0.37	0.65	4.24	2.0	EX. MH 23
B	3.26	40	3.52	141	0.45	0.93	4.20	2.8	MH. 5A

Based on the above criteria and the InfoSewer project report (**Appendix E**), it has been determined that there is ample capacity to service the subject lands (over 100 L/s reserve capacity at these locations).

*Refer to **Appendix D** for the as-constructed information with regards to the existing sanitary system. Refer to **Figure SAN-AB** for the sanitary servicing plan for Area A and Area B and **Appendix E** for the Sanitary Sewer Design Sheets and InfoSewer capacity analysis report.*

4.4. WATER SERVICING

The water servicing criteria and flow demands for the proposed development is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 4-6**.

Table 4-6 Water Servicing Criteria and Flow Demands

Water Design Factors		Water Demand (L/s)[1]	
Average Daily Demand	275 L/capita/day	Average Day Demand	1.75
Maximum Daily Demand Peaking Factor	2.25	Maximum Day Demand	3.94
Peak Hourly Demand Peaking Factor	4.00	Peak Hour Demand	7.01

[1]Refer to the Preliminary Water Distribution Modelling Report by MES in **Appendix F** for water demands as calculated per equivalent population

4.4.1. EXISTING

The subject site is located within Pressure Zone B3 in Halton Region's water distribution system. This is based on the service elevation range for the area. It should be noted that the Region is planning to alter the pressure districts in certain areas of Halton, however, Zone B3 is not affected in this change.

Existing water distribution infrastructure in the vicinity of the site includes:

- 300 mm watermain on Millcroft Park Drive (west)
- 150 mm watermain on Parklane Crescent (north)
- 200 mm watermain on Hadfield Court (between two proposed development blocks)

4.4.2. PROPOSED

MES was retained to carry out a detailed hydraulic analysis of the proposed development's impact on both existing and proposed infrastructure. The analysis includes design years 2016 and 2031. The analysis was based on Region of Halton Water and Wastewater Linear Design Manual, October 2019. The watermain to service the proposed development can achieve these hydraulic requirements by prescribing to the design criteria as summarized in **Table 4-7** and further discussed below.

Table 4-7 Modelled Service Pressures and Fire Flows

	Modelled Scenarios	
	2016	2031
Minimum Service Pressure	416 kPa (60.4 psi)	379 kPa (54.9 psi)
Maximum Service Pressure	579 kPa (84.0 psi)	601 kPa (87.2 psi)
Maximum Day + Fire Flow	164 to 337 L/s @ 20 psi	167 to 330 L/s @ 20 psi

- The service pressures from the watermain are expected to range between 60.4 psi to 84.0 psi (416 kPa to 579kPa) under 2016 conditions and between 54.9 psi to 87.2 psi (379 kPa to 601 kPa) under 2031 conditions.
- Pressures will exceed the OBC pressure criteria of 80 psi (550 kPa), pressure reduction will be required. Pressures will be checked following watermain construction to confirm which units require individual pressure reducing valves.
- The available fire flow meets or exceeds the preliminary fire flow demands utilized for this assessment at the minimum pressure of 140 kPa based on the proposed watermain supply and

assumptions made within this report but should be confirmed when additional information becomes available.

- As the development design is at the initial stages, the findings and recommendations in this report are considered preliminary. As more information is available on the watermain and building layout, all building assumptions and the fire flow demands must be reviewed and updated to confirm that the water supply is adequate.

A network of municipal watermains is proposed throughout the subject lands. The proposed site is expected to be serviced by 200 mm watermains on the north (Area B) and south (Area A) sections of the development that will be looped together, with both ends connecting to the 300 mm watermain on Millcroft Park Drive. These sizes will be confirmed at detailed design when building assumptions and fire flow demands will be reviewed and updated.

The subject mains were designed to achieve service criteria during Average Day, Peak Hour, and fire flow during Maximum Day demand.

Based on the above criteria it has been determined that there is sufficient capacity to service the subject lands.

Refer to the Preliminary Water Distribution Modelling Report (MES, March 2020) in ***Appendix F*** *and* ***Drawings WM-A & WM-B*** *the Water Distribution Plan for Area A and Area B.*

4.5. STORM DRAINAGE

4.5.1. PROPOSED MINOR SYSTEM

Storm servicing for the development will conform to City of Burlington standards. Storm sewer infrastructure will be designed to capture the 100-year storm event in underground storage vaults or superpipes before discharging to Appleby Creek.

The runoff coefficients were based on the proposed land use and the City standard runoff coefficients. The 100-year flows from the subject lands were calculated using the increased runoff coefficients (1.25 x $C_{5\text{-year}}$) as per the City requirements.

The proposed storm sewers within the subject lands will be designed to intercept the minor and some of the major system flows as shown on **Drawings STM-A and STM-B**.

The existing Parklane Crescent minor drainage system, which currently outlets via the Hadfield Court storm sewers, will be redirected to a 1050mm-100-year bypass pipe to be located within the servicing corridor between the CNR tracks and the development areas. This diversion allows the controlled flows from Area B to be connected to the Hadfield Court system with no increase to the existing flows in the Hadfield Court sewers. An existing RLCB that collects flows from the majority of Area B and outlets via Hadfield is not proposed to be utilized for minor system flows however, it is recommend that the structure remain as an emergency overflow for rear lots in Area B and Hadfield Court. At detailed design, the available capacity at this outlet can be utilized, provided that the post-development flows are limited to existing rates or lower.

A hydraulic grade line analysis was undertaken using PCSWMM on the proposed bypass pipe and outfall to Appleby Creek, including the existing storm sewer on Hadfield Court that is to receive flows from Area B. The results of the PCSWMM analysis show that the hydraulic grade line in the existing Hadfield Court storm sewer is expected to be lowered compared to that shown on the 1985 design drawings. Additionally, the HGL delivered to Exiting MH 11A at Parklane is lowered by ~ 1.0 m compared to the 1985 design. The proposed bypass storm sewer generally flows part full except when in the influence of Appleby Creek. Refer to **Appendix G** for annotated plan and profile drawings (**HGL-1, HGL-2**) show the new anticipated HGL elevations at key locations.

Groundwater flows, as intercepted by foundation weeping tiles, will be pumped to the surface and be collected by the minor storm system. RJB's Foundation Weeping Tiles & Groundwater Quality Evaluation, referenced in **Appendix B**, indicates groundwater flows for Areas A and B of 4.8 x 10^{-5} m^3/s and 6.6 x 10^{-5} m^3/s, respectively. The expected groundwater flows are a negligible portion of the 100-year post-development flows and have not been considered in storm sewer sizing. Note the sump pumps will also protect basements from the high HGL in the minor system storage.

Refer to **Figures STM-A & STM-B** *for the Storm Drainage Plans,* ***Appendix G*** *for the Storm Sewer Design Sheets, and the PCSWMM model for Area A and Area B.*

4.5.2. PROPOSED MAJOR SYSTEM

100-year flows will be captured into the minor system for control in Areas A and B.

In the event that the minor system or storage facilities overflow, emergency overland flow routes have been provided as follows:

1. Parkland Crescent will drain as originally designed to the CNR corridor where 100-year flows will be collected in the bypass pipe in a new DICB/inlet structure sized to capture the 100-year flows.
2. Area B - A new DICB/high-capacity inlet will be placed within the corridor adjacent to the CNR. The DICB will be connected to the bypass pipe and will intercept emergency flows only from Area B. Emergency flows from Area B have not been counted in the sizing of the bypass pipe.
3. Area B - An existing RYCB that serves rear yards along Hadfield Court will be preserved to act as an emergency overflow for rear yards in Area B.
4. Hadfield Court will drain as originally designed with overland flows exiting the subject lands via a continuation of an existing overland flow route through Area A.
5. Area A - The overland flow route from Hadfield Court and Area A lands continues through the site directly to Appleby Creek.

At detailed design, suitable inlet structures will be designed where 100-year capture is contemplated within the servicing corridor.

*Refer to Drawings **STM-A, STM-B, GR-A1, GR-A2 and GR-B** for locations of major system drainage measures and 100-year capture locations. Refer to **Appendix G** for overland flow capacity calculations.*

4.5.3. CHANNEL REALIGNMENT / RECONFIGURED IRRIGATION POND

Under existing conditions, the golf club utilizes the existing online pond to supply irrigation water for golf course requirements. Millcroft Golf Club previously engaged RJB to renew the PTTW application with the MECP prior to initiating the planning applications. Through that process, the MECP required the golf course to explore opportunities to reduce reliance on Appleby Creek to protect baseflow for aquatic life. RJB has initiated a Permit process with CH, pursuant to Ontario Regulation 162/06, to create a by-pass pond that would re-direct creek flows under certain storm events to the relocated Mill Pond. This by-pass pond would have allowed the reinstatement of the low flow channel in the area currently occupied by the online Mill Pond. Although this would have reduced some reliance on the creek, the pond's by-pass structure would have still resulted in creek flows above a low flow threshold being used to supplement irrigation water supplies. The CH Permit process was not completed as the MECP issued a PTTW allowing the pond to remain as an online feature, with the condition that the proponent complete the work for the by-pass structure.

The development of Areas A and B is not contingent on taking the pond offline. However, taking the pond offline presents a unique set of opportunities that benefit Appleby Creek, including:

- Protect and enhance the baseflow and aquatic resources of Appleby Creek;
- Water quality improvements (reduced sediment loadings and thermal impacts);
- Provision for the requisite meander belt established as set out by the EER (15.0 m); and
- Continued use of existing development area (Parklane Crescent) stormwater for irrigation.

Continued coordination with RJB and CH will be necessary to finalize the water management strategy related to using the proposed Mill Pond to supplement irrigation water supplies. The operation of the reconfigured irrigation pond & Mill Pond pump house will continue to be managed by Millcroft Greens Corporation, while ownership of the irrigation pond lands will be conveyed to the City at the time of registration. In order for the reconfigured irrigation pond to contribute to the Millcroft Greens golf course irrigation demand, draw-down events will occur periodically whenever the pond's operating level exceeds the proposed horizontal draw pipe invert elevation 134.60m as extended from the existing Mill Pond wet well. This draw pipe's invert elevation is set above the pond bottom elevation of 134.00m to minimize sediment resuspension and act as the primary pond outfall. Further, this horizontal draw pipe extension from the Mill Pond existing pump house's wet well will traverse below the proposed low flow channel to enter the reconfigured irrigation pond. In order to avoid disturbing the watercourse, the extended horizontal draw pipe can be constructed concurrently with the watercourse realignment and pond relocation.

A minimum 2.9m clay liner thickness, in conjunction with underliner drainage, may be recommended as determined by DS Consulting Limited per **Appendix A**. However, during the construction of the pond, the minimum clay liner thickness will be re-evaluated based on the observed surface conditions.

In the absence of draw-down events or evaporation processes preventing the pond operating level from exceeding 136.65m, a proposed emergency spillway is located near the Upper Middle Road culvert along the top of pond berm as set at an invert elevation of 136.65m. This emergency spillway is proposed to safely convey Parklane Crescent 100-year flows exiting the single inlet pipe & headwall as extended from MHEX11A calculated in **Appendix H**.

Proposed upgrades to the existing Mill Pond pump house will include but are not limited to ensuring that no municipally sourced water can mix with the irrigation pond volume. This improves current practices whereby municipal water is mixed with existing surface water supplies to top up the existing online pond. The location of the proposed extension of the horizontal draw pipe is made accessible for maintenance purposes by a proposed 4.0m access road aligned along the pond's top of berm. Note that this same

access road is part of the trail network that allows pedestrians to traverse between Areas A & B and Upper Middle Road.

As Hadfield Court's existing drainage area now bypasses the reconfigured irrigation pond along with proposed Area A & B developments, Millcroft Greens Corporation's operation of the reconfigured irrigation pond will include supplementing golf course irrigation demands by municipally sourced water from services available along Country Club Drive. The proposed Mill Pond pump house upgrades and other irrigation demand details by RJB will be submitted separately. Note that supplementing irrigation demand will no longer include using a by-pass component wherein the irrigation pond is connected to Appleby Creek's low flow channel.

Specific to Area A, modifications to the Regional Storm flood plain are proposed to create additional development area and remove the existing weir structure upstream of Upper Middle Road to reduce overall water levels. As part of the proposed Area A development, an outfall pipe & headwall will connect to an extension of the proposed low flow channel at an invert elevation of 136.04m. Note this extension of the proposed low flow channel confluences with the existing low flow channel at an approximate invert elevation of 135.91m. Additionally, a stormwater outfall will be required to connect the existing 900mm SWM outfall from Country Club Drive to the realigned channel at an invert elevation of 135.88m. The natural channel realignment works will include the applicable outfall designs as well as any other natural channel features as determined by a fluvial geomorphologist.

The proposed in-channel works, as extended to the limit of grading as illustrated in **Figure 1 - Constraint Mapping**, provides the necessary grading to construct the right-of-way in Area A. The vertical difference between the proposed floodplain bottom and right-of-way centerline is illustrated on **Figure 2 - Sections**. As confirmed by CH on January 28th, 2022, the existing retaining wall does not constitute a new top of bank and does not create a confined valley system. Further to references in **Section 4.2.1**, Block 99 contains a proposed 1.2m high retaining wall and abutting 1.0m shelf that provides maintenance access outside of the proposed channel corridor. As such, the retaining wall's maintenance avoids the need to access the wall for maintenance purposes via the 7.5m development setback within Block 91. Given the limited height of the wall, its direct accessibility from the ROW in Area A for maintenance purposes and the additional 1.0m beyond the base of the wall, there is sufficient ability to repair the wall from the development side should repairs ever be needed in the future.

The proposed floodplain alterations are outside the CH staked riparian wetland limits and in keeping with negligible changes (approx. 0.25%) between the pre & post-development contributing drainage areas to that wetland as illustrated in **Figure 3 – Riparian Wetland Catchment Area**. Additionally, efforts to keep the development outside of CH's regulated area have been attributed to an increase in riparian storage volume during infrequent storm events from pre- to post-development conditions as further discussed in **Section 4.5.5**. The proposed development would occur in parallel to or following the channel and pond reconfiguration works as a condition of subdivision plan approval. Conservation Halton's Policies and Guidelines for the Administration of Ontario Regulation 162/06 and Land Use Planning Policy Document anticipates a process whereby alterations flood plains alterations to create building lots can be considered. Specifically, Policy 2.19.1 states:

Major flood plain alterations (including placement of fill to create, or enlarge, a building lot) and major watercourse alterations (including enclosures and diversions from one watershed to another) are generally not permitted. Such alterations may be considered where justification is provided through a subwatershed study, an Environmental Assessment or similar comprehensive study and are subject to conformity with municipal planning documents. The applicable study or assessment must be current (generally within 5 years) and must be supported by Conservation Halton.

The term 'comprehensive study' as used in Policy 2.19.1 is defined in CH's policy document as follows:

Means a detailed study that fully assesses existing conditions and the potential impacts of proposed development with respect to, at a minimum, hydrology, hydraulics, hydrogeology, fluvial processes, erosion, slope stability and the natural environment (including, but not limited to, ecological functions).

The contents of this FSR report, hydrogeology study, and EER are intended to fulfill the requirements outlined above in the 'comprehensive study' definition. In addition to the technical studies, a separate Benefit Analysis Report (JLA, 2022) has been submitted under separate cover that summarizes the anticipated benefits as a result of the proposed development. Further, any in-channel works or works that are within the CH regulated area will be subject to applicable CH permit applications which will be appended by design drawing details as submitted under separate cover including but not limited to:

- Area A proposed outfall pipe design
- Floodplain alterations:
  - Grading to reconfigure the Regional storm floodplain
  - Outfall connections from existing Country Club Drive & the proposed Area A right-of-way
  - Low flow channel re-creation upstream of Upper Middle Road culvert (through the area that is currently an online pond)
  - Reconfigured irrigation pond works & trail connection to Upper Middle Road

*Refer to **Appendix B** for the Hydrogeology Report by RJB and the EER by Beacon provided under separate cover as part of the ZBA submission.*

4.5.4. EXISTING & PROPOSED FLOOD ELEVATIONS

The existing HEC-RAS floodplain model was obtained from CH and updated with more accurate topographic information for the reach to create an existing condition floodplain as indicated in **Drawing FP-1**. A detailed survey was used for the terrain within the subject lands. The Urbantech existing model results are reasonably consistent with the original CH model upstream of Upper Middle Road. Downstream of Upper Middle Road, the Urbantech existing water levels were generally lower, likely due to updated channel works / topographic information in this reach. This decrease in water levels downstream did not have any bearing on the upstream water levels on the north side of Upper Middle Road, which are nearly equal for the two models (Section 281).

The existing model was updated to reflect the proposed removal of the online pond and the reconfiguration of the channel. Proposed floodplain limits are depicted in **Drawing FP-2**. A summary of the existing and proposed flood elevations is provided in **Table 4-8** below. The proposed modifications occur in the highlighted sections (i.e., along Area A between Millcroft Park Drive and Upper Middle Road).

Table 4-8 Existing and Proposed Flood Elevations

Section Numbers	Regional Storm			
	CH Model	Urbantech Existing Condition	Urbantech Proposed Condition	Difference (Proposed-Existing)
	W.L.	W.L.	W.L.	
301	141.42	141.33	141.33	0
300	141.43	141.32	141.32	0
299	141.3	141.26	141.26	0
298	141.08	140.82	140.82	0
297	140.52	140.69	140.5	-0.19

Section Numbers	Regional Storm			
	CH Model	Urbantech Existing Condition	Urbantech Proposed Condition	Difference (Proposed-Existing)
	W.L.	W.L.	W.L.	
296	140.2	140.58	140.3	-0.28
295.5	Millcroft Park Drive			
295	139.9	140.15	139.33	-0.82
294	139.93	140.20	139.42	-0.78
293	139.79	139.84	139.37	-0.47
292	139.42	139.60	139.23	-0.37
291	139.22	139.15	138.92	-0.23
290	138.96	138.94	138.73	-0.21
289	138.89	138.83	138.57	-0.26
288	138.38	138.32	138.04	-0.28
287	138.21	138.12	138.13	0.01
286	138.23	138.13	138.14	0.01
285	138.24	138.14	138.1	-0.04
284	138.24	138.13	138.09	-0.04
283	138.21	138.10	138.05	-0.05
282	138.22	138.00	137.94	-0.06
281	137.99	137.98	137.97	-0.01
279.48	Upper Middle Road			
278	137.59	137.53	137.45	-0.08
277	137.71	137.26	137.26	0
276	137.7	137.21	137.21	0
275	137.69	137.25	137.25	0
274	137.69	137.28	137.28	0
273	137.68	137.27	137.27	0

The results in the preceding table and **Drawings FP-1 / FP-2** indicate that the reconfiguration of the irrigation pond and channel under the proposed scenario result in a decrease in flood elevations on adjacent/existing lots along the corridor, as well as upstream of Millcroft Park Drive. There is a considerable decrease in the Regional Storm water levels on the existing lots along the subject lands as well as no increase in Appleby Creek water levels downstream. Several negligible (~0.01m) increases occur in sections that are far removed from existing lots. Based on Wood's Flood Vulnerability Prioritization and Mitigation Study (2017), the study identified a total of three (3) flood vulnerable areas (#AP11-13) as identified on Sheet AP-05 referenced in **Appendix M**. AP13 is located immediately upstream of Millcroft Park Drive, between cross-sections 301 & 295.5. No increases to the Regional Storm water surface elevation are expected to occur in this location based on Urbantech's existing and proposed model comparison. AP12 is located immediately upstream of Upper Middle Road, between cross-sections 285 & 281. This area experiences, minor decreases to the water surface elevation under proposed conditions. Lastly, AP11 is located downstream of the proposed development (upstream of the Ironstone Drive crossing and downstream of the Corporate Drive crossing) between cross-sections 260 & 245. Note, Urbantech's model reflects CH's HEC-2 (or current floodplain mapping) as extended

immediately upstream of Appleby Line crossing for purposes of downstream floodplain analysis as referenced in **Appendix H**. As such, there will be no negative impacts to the Regional water levels upstream or downstream of the development.

For the purpose of HGL analysis for Areas A and B, the 100-year tailwater elevations in the model are noted below:

- Section 286 (Existing Hadfield Court outfall / Proposed Area A outfall)
  - 137.46m (existing)
  - 137.51m (proposed)

- Section 282 (downstream of Mill Pond / Proposed Parklane Crescent outfall)
  - 137.36m (existing)
  - 137.34m (proposed)

The review of the previous FSR submission noted by the agencies that the Upper Middle Road culvert size was inconsistent with the survey, which indicated twin 1.84 m x 1.84 m openings. Based on field verification and the CH model/culvert inventory report, the culverts were confirmed to be 4.7 m wide, and the measured height was approximately 1.92m. The depth of the stone within the bottom of the culvert appeared to be reduced (less than 0.1 m) compared to the original CH estimates (0.3 m). CH also suggested that the culvert should be modelled as a bridge due to the suggestion that the internal wall between the two openings would be better represented as a pier.

Urbantech has continued to model the crossing as a twin box culvert for the following reasons:

- The HEC-RAS hydraulic reference manual does not consider this small internal wall area, nor the areas around the outer edges of the culverts, to be ineffective / contributing to significant losses.
- The culverts are twin boxes, rather than a span with piers separating the flows. It is assumed that a bridge modelling approach would not be recommended for twin circular culverts.
- The culverts are closed bottom and a suitable roughness (0.035) has been selected.
- The use of culverts avoids the need for creating a “false” flat bottom for the sections within the bridge

4.5.5. RIPARIAN STORAGE ANALYSIS

The HEC-RAS model was used to undertake a riparian storage analysis for the reach to assess how the proposed development plan will impact the flood storage for the corridor. The 2-year through Regional Storm was assessed and the results are summarized in **Table 4-9A** below. The model geometry files were adjusted to remove weirs, culverts, and berms, including the irrigation pond berm for the riparian scenarios.

Table 4-9A Summary of Riparian Storage Volumes

	Pre-development conditions (Riparian)		Proposed without pond* (Riparian)		Difference	
Section Numbers	Total (XS 240 to XS 345)	Site (XS 281 to XS 295)	Total (XS 240 to XS 345)	Site (XS 281 to XS 295)	Total (XS 240 to XS 345)	Site (XS 281 to XS 295)
Return Period	Volume (x1000 m³)					
Regional	142.46	29.32	144.80	31.86	2.34	2.54
100-year	86.17	19.09	87.59	20.73	1.42	1.64
50-year	78.86	17.68	80.12	19.15	1.26	1.47

25-year	71.08	15.95	71.93	17.01	0.85	1.06
10-year	59.07	13.28	59.30	13.72	0.23	0.44
5-year	51.58	11.42	51.49	11.53	-0.09	0.11
2-year	36.99	7.67	36.96	7.75	-0.03	0.08

*This scenario demonstrates that the proposed channel realignment / cut-fill does not rely on the offline irrigation pond berm

As can be seen from the above results, the proposed development concept does not have a negative impact on riparian storage volumes for the reach proposed to be modified. The proposed works increase the storage on site. There are minor reductions in the overall riparian storage in the 2-year and 5-year storms – this is likely attributed to improvements/reductions in water levels within the subject lands causing reduction in water levels upstream, which reduces the flood storage slightly. There is a general increase in floodplain storage resulting from the proposed grading for the larger storms, which provides an overall benefit.

A summary of the total flood storage (i.e., with structures in place) was also completed (**Table 4-10B**). While this comparison shows a net reduction in overall flood storage in the system (and a slight increase through the subject lands), these results must be considered with the proposed reduction in water levels attributed to the improvements within Area A (not unlike how a culvert improvement would reduce water levels and corresponding flood plain volumes). These results should be seen as a positive improvement as there is less volume (and area) flooded under proposed conditions.

Table 4-10B Summary of Flood Storage Volumes

	Pre-development conditions		Proposed		Difference	
Section Numbers	Total (XS 240 to XS 345)	Site (XS 281 to XS 295)	Total (XS 240 to XS 345)	Site (XS 281 to XS 295)	Total (XS 240 to XS 345)	Site (XS 281 to XS 295)
Return Period	Volume (x1000 m³)					
Regional	187.90	30.67	175.90	31.36	-12.00	0.69
100-year	110.44	17.32	91.16	17.51	-19.28	0.19
50-year	98.43	15.84	81.83	15.77	-16.60	-0.07
25-year	80.43	14.02	72.69	13.74	-7.74	-0.28
10-year	61.72	11.39	58.37	10.27	-3.35	-1.12
5-year	53.28	9.92	49.79	7.96	-3.49	-1.96
2-year	38.41	7.50	34.98	4.56	-3.43	-2.94

*Refer to **Appendix I** for the Hydraulic Modelling and Results. Refer to **Drawing FP-1** for the Existing Floodplain Mapping and **Drawing FP-2** for the Proposed Floodplain Mapping.*

4.6. STORMWATER MANAGEMENT

4.6.1. DESIGN

The existing Millcroft Community was developed in the mid-1980s to design standards and criteria of the time. New works have been designed to conform to current City, Conservation and MECP requirements. The proposed stormwater management system and drainage areas are depicted on **Drawings STM-A & STM-B**.

4.6.2. QUALITY CONTROL

Area A will discharge directly into Appleby Creek and Area B is proposed to tie into existing development which ultimately discharges to Appleby Creek though Area A. An oil & grit separator (OG/S) is proposed downstream of the Area A and Area B storage tanks/pipes. CB shields will be provided to augment the water quality and provide a treatment train approach in conjunction with the OG/S. A biofiltration LID measure can be implemented in the “island” at the south end of Area A to operate in conjunction with the water quality benefits provided by the OG/S & CB shields as illustrated on **Drawings LID-A1**, **LID-A2** & **LID-B1**. Note, this biofiltration LID measure would not be required to meet any SWM or site-wide water balance infiltration targets but rather provided as a best management practice. Lastly, roof leaders will be discharged to pervious areas to enhance water balance/erosion protection as per standard practice, as discussed in further detail in **Section 4.6.4**.

Based on the minimum practical orifice size of 50mm, only a small measure of extended detention can be achieved for the 25mm event. It is recommended that ED requirements be waived due to the small catchment sizes.

Refer to **Drawings STM-A & STM-B** for details of the stormwater management plan for Areas A and B. Preliminary oil/grit separator sizing is included in **Appendix G**.

4.6.3. QUANTITY CONTROL

100-year flows from Area A will be controlled to pre-development levels. 100-year flows from Area B are proposed to be controlled to the 5-year flow originating from Parklane Crescent as flows from Parklane will be diverted to a new storm sewer. The proposed controlled flows from Area B are less than the 100-year predevelopment flow from Area B. Predevelopment flow rates are based on runoff coefficients

(generally 0.3 to 0.4) for the subject lands that were established when the original community was developed. The Pre-development areas are shown on **EX-AB**.

Table 4-10 summarizes 5- and 100-year predevelopment flows for Areas A and B and proposed target flows rates for the post development conditions for Areas A and B.

Table 4-10 - Pre-Development/Target Flows

Area ID	5-Year (m^3/s)	100-year (m^3/s)	Target (m^3/s)
Area A	0.253	0.509	0.509
Area B	0.354	0.712	0.546

To meet target flow rates, on-site detention in the form of super pipes and underground storage vaults will be utilized. Storage will be accomplished through the use of orifice plates placed in control manholes at the downstream end of the storage systems.

Area B will require ~ 870 m^3 of storage in the 100-year event which will be accomplished by a 450 mm orifice. Conceptual placements of underground storage vaults within Area B are shown on **STM-B.**

Area A will require ~ 815 m^3 of storage in the 100-year event. Storage within Area A will be split into an east (338m^3) and west (477m^3) system. The required orifice sizes in Area A are 225 mm (east) and 270 mm (west). Conceptual placements of underground storage vaults within Area A are shown on **STM-A.** A portion of Area A (i.e. rear yards abutting the irrigation pond) is proposed to be released uncontrolled directly to the Appleby Creek watercourse corridor. Over control of flows within the controlled portions of Area A account for uncontrolled flows that are to be released.

As a single orifice device is being proposed for quantity control, it is recommended that only the 100-year target be required to be met for the site development. The proposed developments represent ~2.5% of the watershed area and any minor exceedances in 2–50-year events will be immeasurable in the watershed context.

Stormwater management calculations can be found in **Appendix H**.

4.6.4. WATER BALANCE

RJB prepared revised detailed water balance calculations (March 2022) in a separate memo to the hydrogeology report (November 2020) as informed by land use plan **Drawing LU-AB**. Both the memo and report are provided in **Appendix B**.

Development of the Subject Property will reduce infiltration to the shallow groundwater flow system. The groundwater balance calculations suggest that, without mitigation, a potential reduction of 28% of the average annual groundwater infiltration could occur as per the memo (RJB, 2022) in **Appendix B**.

In order to promote infiltration while mitigating against high groundwater conditions, the proposed SWM strategy recommends directing a portion of available roof areas to rear yards (disconnection of roof leaders) and an increased topsoil depth of 300 mm. The revised water balance calculations suggest that implementing increased topsoil depth and downspout disconnection to direct roof runoff to pervious areas across the proposed development will effectively maintain pre-development infiltration volumes.

*Refer to **Appendix B** for the Hydrogeological Report Investigation by RJB (November 2020) and Revised Water Balance Calculations by RJB (March 2022).*

4.7. IMPLEMENTATION

Area A will be developed in conjunction with or following the reconfiguration of the creek and removal of the online pond as a condition of subdivision plan approval.

Area B relies on Area A based on the storm servicing configuration to outlet controlled flows into Appleby Creek. However, Area B can be developed independently of delivering serviced lots within Area A by re-routing Area B controlled flows away from Hadfield Court. Instead, Area B controlled flows would discharge to an appropriately sized storm pipe with existing Parklane Crescent flows into the irrigation pond's single inlet headwall. As such, Area B relies on Area A to complete irrigation pond reconfiguration works to outlet Area B controlled flows.

The limit of excavation for connections to external services is depicted for Areas A & B in the Construction Management and Mobility Plan as illustrated on **Drawings CMMP-A & CMMP-B**.

4.8. NOISE AND VIBRATION FEASIBILITY STUDY

HGC completed a Noise and Vibration Feasibility Study for the proposed development. The study results indicate that the proposed residential development, inclusive of Areas A and B, is feasible. Future road and rail traffic sound levels will exceed MECP and CN guidelines, but feasible means exist to reduce the impact to within acceptable limits. The study presents several recommendations to consider during the detailed design of the site including acoustic barrier requirements along the limits of development nearest to CNR lands and adjacent to Upper Middle ROW as illustrated on Figure 3a in **Appendix K**. Note, the proposed acoustic barrier will tie into the existing acoustic barrier located along CNR's railway.

*Refer to **Appendix K** for the Noise and Vibration Feasibility Study completed by HGC (February 2022).*

4.9. SUMMARY

The proposed development can be adequately serviced through a combination of existing and proposed municipal infrastructure. In summary:

- Sanitary Servicing will be accomplished through the construction of local sanitary sewers which will connect to the existing 525 mm diameter municipal sanitary sewer within Millcroft Park Drive.
- Water servicing for domestic potable and fire protection will be through connections to the existing 300 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.
- Storm drainage will include the construction of local storm sewers and infrastructure designed to capture, convey, control all flows up to the 100-year storm to Appleby Creek and maintain external flow conveyance. The Hadfield Court drainage will drain to Appleby Creek. The Parklane Crescent drainage will be diverted from Hadfield Court and conveyed directly to the reconfigured irrigation pond via a new storm sewer in the servicing corridor north of the CNR.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks to maximize reuse of soils within the property and minimize the use of retaining walls.
- Stormwater quantity control will be provided via superpipes and underground storage vaults. A hydraulic grade line analysis and comparison of existing and proposed drainage areas has demonstrated that there will be no increase in flows or water levels on the existing properties.
- Quality control will be provided through the use of oil/grit separators and CB Shields. A biofiltration LID measure can be implemented in the “island” at the south end of Area A.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

5 AREA C

LOCATION: North side of Country Club Drive and east of Berwick Drive.

AREA: 1.46 ha

CURRENT LAND USE:
Hole No. 1



5.1. EXISTING CONDITIONS

5.1.1. LAND USE & TOPOGRAPHY

The site is currently being used as Hole 1 for the Millcroft Golf Club. The golf course will remain operational, and the Hole will be re-designed to accommodate future development. Currently, the site is covered with greens/fairways/and bunkers, low-lying vegetation and trees, and golf cart trails.

R-PE completed topographical surveys of the subject lands in September 2019.

The site generally falls from northeast to the southwest with a maximum grade change of approximately 0.50 m.

***Figure EX-C** illustrates the existing site features, topography, and drainage patterns.*

5.1.2. SOIL CONDITIONS

Millcroft Greens Corporation retained DSCL to undertake the preliminary geotechnical investigation for the proposed development.

The geotechnical report indicates the following subsurface conditions:

- Topsoil – 150 mm to 180 mm thick surficial layer of topsoil
- Fill material – ranging from 0.2 m to 1.1 m below grade consisting mainly of clayey silt, trace rootlets and trace topsoil.
- Upper soils – Clayey silt to sandy silt till
  Lower soils – Clayey silt till
- Bedrock encountered at depths ranging from 3.3 mbgs to 6.1 mbgs

DSCL's preliminary investigation indicated that the soil stratigraphy of the subject property consists of fill underlain by native deposits of clayey silt, sandy silt till, clayey silt till and a shale complex overlying shale bedrock. This report also establishes that no groundwater was found in the boreholes during drilling or upon completion of drilling. The stabilized groundwater levels observed in the monitoring wells installed in the boreholes range from 1.4m (BH19-102) to 4.5m (BH19-101).

Highly weathered shale bedrock of the Queenston Formation was encountered below depths ranging from 3.3m to 6.1m, however was not proven through core drilling. Methane gas is anticipated in the bedrock, and appropriate care and monitoring is essential in all confined bedrock excavations. For the purposes of this investigation, it is assumed that any remediation work which may be required to deal with potential contamination will be completed prior to earthworks and servicing.

The complete study in **Appendix A** provides a detailed discussion of geotechnical site conditions. The report further discusses recommendations pertaining to site alteration procedures as well as the construction of roads, sewers, foundations, and drainage systems based on the subsoil conditions.

5.1.3. GEOTECHNICAL RECOMMENDATIONS

The geotechnical report recommended the following minimum pavement makeup for all internal site roads based on the assumed residential minor local or local traffic usage conditions:

- 40 mm HL3 Asphaltic Concrete
- 50 mm HL8 Asphaltic Concrete
- 150 mm Granular 'A'
- 350 mm Granular 'B'

The Area C proposed design grade at road surface will be at Elevation 143.9 to 144.9m. Assuming basement depth of 3m, the basement floors will be at about Elev. 141 to 142m. Bedrock elevation in the boreholes varies from 139.3 to 141.7m as per the geotechnical report prepared by DS Consultants Ltd in **Appendix A**.

Area C basements floor will generally be constructed in glacial till deposits. However, in any areas where basements are to be constructed below the bedrock surface, methane gas monitoring should be carried out during excavation. If methane gas is encountered during excavation, gas venting systems may be required depending upon the gas concentration readings.

*Refer to **Appendix A** for the Geotechnical Report for Areas A through D as completed by DSCL (December 2020).*

5.1.4. HYDROGEOLOGY RECOMMENDATIONS

A hydrogeological report investigation was completed in November 2020 by RJB and has been included in **Appendix B**.

The field and laboratory work included the installation of three monitoring wells across the subject property in November 2018. Groundwater levels across Area C have been monitored monthly from January 2019 through to November 2021 and will continue to be monitored quarterly thereafter.

A review of the groundwater level data suggests the following:

- Groundwater levels in monitoring wells were observed have varied about 136m above sea level (masl) with seasonal fluctuations ranging from 0.5m to 4.0m range.
- Shallow groundwater levels typically exhibit seasonal fluctuations with the highest elevations generally observed in the spring and the lowest in the summer months. To date, seasonal high levels were observed in January 2020 as a result of the mild winter conditions.

*Refer to **Appendix B** for the Hydrogeological Report Investigation completed by RJB (November 2020).*

A conventional foundation weeping tile drain system and sump pumps for the proposed detached lots within Area C will address seasonal high groundwater elevations and hydraulic gradeline. Based on the Foundation Weeping Tiles and Groundwater Quality Evaluation by RJB provided in **Appendix B**, groundwater flows are estimated to be 2.3 x 10^{-5} m^3/s, which is 7 orders of magnitude less than the 100-year flow that that will be captured and conveyed by the minor system. These flows will not impact the allowable release rate or storage volumes. Groundwater samples were collected and submitted for analysis to compare to the City's storm sewer discharge criteria. The results of the groundwater analysis in comparison to the City's storm sewer discharge criteria and discussion are presented by RJB in **Appendix B**. For discussion on the estimated groundwater flow impacts to the storm sewer system, refer to **Section 5.5.1**.

*Refer to **Appendix B** for the Foundation Weeping Tiles & Groundwater Quality Evaluation completed by RJB (February 2022).*

5.1.5. EXISTING MINOR SYSTEM DRAINAGE

There are 1800mm and 300mm storm sewers on Berwick Drive and Country Club Drive, respectively. A 900mm storm pipe on the east side of the development (running between backyards of a housing development and the Millcroft Golf Club lands) connects to the storm pipe in Country Club Drive. A RYCB at the northwest corner of the site collects overland flow from backyards and houses along Berwick Drive. The RYCB connects to a 300mm storm pipe (running along the side of the northmost house) and outlets to the storm sewer on Berwick Drive.

A RYCB at the southwest corner of the site collects overland flow from the backyards of houses on Berwick Drive and potentially from the golf course. The RYCB connects to a 200mm storm pipe (running along the backyard of the southernmost house) and outlets to the storm sewer on Country Club Drive. A DICB on the northeast corner the site collects overland flow from backyards and the Millcroft Golf Course. The DICB connects to the 900mm storm sewer running behind the houses and outlets to the storm sewer on Country Club Drive. Two double catchbasins are located along the Millcroft Golf Club roadway crossing Country Club Drive to connect two sections of the course as illustrated in **Figure EX-C**.

The majority of Area C ultimately drains to the existing1200mm on Arbourfield Street. This large diameter pipe will serve as the outlet for Area C.

5.1.6. EXISTING MAJOR SYSTEM DRAINAGE

The majority of Area C generally drains from the interior of the site to CBs at the exterior limits, ultimately making its way to the existing storm sewer located within the Country Club Drive ROW. A small plot of land drains to a RYCB connected to the existing storm sewer within the Berwick Drive ROW. The average slope across Area C is approximately 1.0%.

Area C has been divided into four (4) existing drainage subcatchments and is associated with four (4) external existing drainage areas as depicted in **Figure EX-C**, summarized in **Table 5-1** and further discussed below.

Table 5-1 Existing Drainage Areas - Area C

Area C			External to Area C		
Area ID	Area (ha)	RC (-)	Area ID	Area (ha)	RC (-)
S01-C	0.37	0.30	E01-C	2.70 (1.95 + 0.75)	0.33
S02-C	0.80	0.30	E02-C	0.43	0.40
S03-C	0.26	0.30	E03-C	0.41	0.40
S04-C	0.04	0.30	E04-C	0.19	0.40
Total	1.46	0.30	Total	3.73	0.35

- Catchment S-01C currently sheet drains southeast overland to an existing double inlet catchbasin (DICB) as Outlet #1C, ultimately connected to the storm sewer within the Country Club Drive ROW.
- Catchment S-02C currently sheet drains southeast overland to an existing RYCB as Outlet #2C, ultimately connected to the storm sewer within the Country Club Drive ROW.
- Catchment S-03C currently sheet drains southwest overland to an existing RYCB as Outlet #3C, ultimately connected to the storm sewer within the Country Club Drive ROW.
- Catchment S-04C currently sheet drains north overland to an existing RYCB as Outlet #4C, ultimately connected to the storm sewer within the Berwick Drive ROW.

The total existing drainage from Area C is 1.70 ha with an average runoff coefficient of 0.30. There are four (4) external drainage subcatchments that are conveyed through Area C.

- Catchment E01-C consists of existing golf course and a small portion of road; this currently sheet drains southwest overland to catchment S01-C, ultimately draining to the existing DICB as Outlet #1C, connected to the storm sewer within the Country Club Drive ROW.
- Catchment E02-C consists of existing rear lot residential from 2175 Country Club Drive, draining northwest overland to catchment S01-C, ultimately draining to the existing DICB as Outlet #1C, connected to the storm sewer within the Country Club Drive ROW.
- Catchment E03-C consists of existing rear lot residential along Berwick Drive, draining south overland to catchment S04-C, ultimately draining to the rear lot double catchbasin (RLDCB) as Outlet #4C, connected to the storm sewer within the Berwick Drive ROW.
- Catchment E04-C consists of existing rear lot residential along Berwick Drive, draining south overland to catchment S03-C, ultimately draining to the RLDCB as Outlet #3C, connected to the storm sewer within the Country Club Drive ROW.

The total external drainage being conveyed through Area C is 3.73 ha with an average runoff coefficient of 0.35.

*Please refer to the existing site drainage for Area C as depicted in **Figure EX-C** and calculations in **Appendix H**.*

5.2. DEVELOPMENT CONCEPT

5.2.1. LAND USE & RIGHT-OF-WAYS

As shown conceptually in **Figure SK-74** & calculated in the **Draft Plan of Subdivision**, by Gerrard Design & Wellings Planning Consultants Inc. respectively, the proposed Area C development consists of the following land use components:

- Detached Homes: Lots 70-83

- Parkette: Block 103
- Cart Path & Walkway Easement: Block 104
- Open Space: Blocks 105 & 106
- ROW: Block 109

The proposed development will be advanced through the draft plan of subdivision and subsequent condominium approval processes.

Area C will consist of 14 residential-detached lots and Block 109 including a 11.2 m ROW with 8.1 m of pavement width (or 8.7m wide; as measured from face of curbs) as illustrated on road cross-section **SEC-C1**. The 11.2m ROW includes but is not limited to a 2.1m sidewalk to accommodate cart-path access, 6.0m two-way travel lanes and a 2.4m parking lane. Generally, residential-detached lot depths are approximately 28.0 m to 40.0m along with widths of 15.24 m to 18.30 m.

The non-developable area will consist of a parkette as Block 103, walkway easement as Block 104 and Open Spaces as Blocks 105 & 106.

The above-mentioned ROW within Area C has been developed in keeping with the master plan vision to support the proposed surface treatment and will be further refined in consultation with the required approval agencies & utility companies. As such, the cross-sections are conceptual and are to be discussed further with the City of Burlington. Note, the ROW cross-section was developed in consultation with RTG Systems Inc with respect to utilities.

*Refer to **Drawing GP-C** for the Area C general plan.*

Note, many of the lots include a 6.0 m buffer within the proposed residential lots adjacent to the existing residential development east and west of Area C. This buffer will be landscaped to act as a privacy feature between existing and proposed residential lots and is illustrated in cross-section view by NAK Design Strategies as part of the Conceptual Open Space package submitted under separate cover with this submission. Existing trees within the 6.0m buffer be preserved wherever possible as illustrated in Arborist Report Figure TP-3 by Beacon included in **Appendix L**. For details on ownership and condition ratings, refer to Arborist Report Figure TP-3.

*Refer to **Appendix L** for the Arborist Report for Area C.*

5.2.2. GRADING

The proposed conceptual grading for the development will be designed in accordance with City of Burlington standards. Grading is generally governed by the existing boundary conditions. Site grading has also been designed to maintain adequate cover over proposed services. No external grading works are proposed.

A preliminary grading concept plan has been prepared for the subject lands based on the following engineering constraints:

- Storm outlet elevations
- Major system drainage paths
- Provision of minimum cover over services
- Proposed road patterns and land use
- Elevations along boundary roads and property lines
- Application of the City of Burlington standards

The grading plan, **Drawing GR-C**, is consistent with the City standards. In general, grading of the proposed plan of subdivision adjacent to the surrounding development and roads matches the existing grades at the property line, as appropriate. Additionally, significant effort has been exercised to ensure that the grades of the proposed lands transitions gradually from northeast to southwest and that proposed grades minimize impacts to existing trees within the 6.0 m buffer adjacent to the existing properties. Note, trees for preservation and associated protection fencing by Beacon is included on **Drawing GR-C** as per Drawing TP-3 included in **Appendix L**. The Preliminary Landscape Design for Area C, as prepared by NAK Design Strategies included under separate cover in this submission, is refleced on **Drawing GR-C**.

*Refer to **Drawing GR-C** for the Area C grading plan.*

5.3. SANITARY SERVICING

The sanitary servicing criteria for Area C is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 5-2**.

Table 5-2 Sanitary Servicing Criteria - Area C

Parameter	Value	Parameter	Value
Average Domestic Flow	275 L/capita/day	Population (Single Family)	3.52 person per unit
Infiltration / Inflow	0.286 L/s/ha	Unit Count (#)	14
Peaking Factor (M)	Harmon Formula	Area (ha)	1.46
Minimum	4.50		
Maximum	2.00		

5.3.1. EXISTING

There is currently no sanitary servicing on Country Club Drive however there are 450 mm and 200 mm sanitary pipes on Berwick Drive (east) and Arbourfield Drive (west), respectively. Construction of a sanitary pipe on Country Club Drive will be required to connect the site with the adjacent sanitary pipes.

5.3.2. PROPOSED

Based on a review of the available as-constructed information (refer to **Appendix D**), the subject lands can be serviced by gravity sewers. Wastewater infrastructure will be designed in accordance with the latest Region of Halton standards and specifications.

The proposed sanitary sewers that will service the subject lands will discharge to the existing 200 mm sanitary sewer on Arbourfield Drive. Installation of a new sanitary manhole will be required within the Country Club Drive ROW to accommodate connection into the existing sewer. Sanitary sewer design sheets have been prepared and used to size proposed sanitary sewers for the proposed development. The population is estimated using the population equivalent densities per the Halton Region standards.

Population Estimates and Sanitary Design Sheets can be found in **Appendix E**. See **Table 5-3** below for a summary of the sanitary servicing for Area C.

Table 5-3 Sanitary Servicing Summary - Area C

Area	Sanitary Area (ha)	# of Units	Population Equivalent Density (ppu)	Population	Dry Weather Flow (L/s @ 275 L/c/d)	Infiltration / Inflow (L/s @ 0.286 L/s/ha)	Peaking Factor	Total Sanitary Flow (L/s)	Connecting Manhole
C	1.46	14	3.52	50	0.16	0.42	4.31	1.1	EX.MH 10

Based on the above criteria and the InfoSewer capacity analysis, it has been determined that there is ample capacity to service the subject lands. The downstream sewers have reserve capacity for approximately 18 L/s and are less than 25% full.

*Refer to **Appendix D** for the as-constructed information with regards to the existing sanitary system. Refer to **Figure SAN-C** for the sanitary servicing plan for Area C and **Appendix E** for the Sanitary Sewer Design Sheets and InfoSewer capacity analysis report.*

5.4. WATER SERVICING

The water servicing criteria and flow demands for the proposed development is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 5-4**.

Table 5-4 Water Servicing and Flow Demands

Water Design Factors		Water Demand (L/s)[1]	
Average Daily Demand	275 L/capita/day	Average Day Demand	1.75
Maximum Daily Demand Peaking Factor	2.25	Maximum Day Demand	3.94
Peak Hourly Demand Peaking Factor	4.00	Peak Hour Demand	7.01

[1]Refer to the Preliminary Water Distribution Modelling Report by MES in **Appendix F** for water demands as calculated per equivalent population

5.4.1. EXISTING

The subject site is located within Pressure Zone B3 in Halton Region's water distribution system. This is based on the service elevation range for the area. It should be noted that the region is planning to alter the pressure districts in Halton, however, Zone B3 is not affected in this change. Existing water distribution infrastructure in the vicinity of the site includes:

- 200 mm watermain on Country Club Drive.

5.4.2. PROPOSED

MES was retained to carry out a detailed hydraulic analysis of the proposed development's impact on both existing and proposed infrastructure. The analysis includes design years 2016 and 2031. The analysis was based on Region of Halton Water and Wastewater Linear Design Manual, October 2019. The watermain to service the proposed development can achieve these hydraulic requirements by prescribing to the design criteria as summarized in **Table 5-5** and further discussed below.

Table 5-5 Modelled Service Pressure and Fire Flows

	Modelled Scenarios	
	2016	2031
Minimum Service Pressure	416 kPa (60.4 psi)	379 kPa (54.9 psi)
Maximum Service Pressure	579 kPa (84.0 psi)	601 kPa (87.2 psi)
Maximum Day + Fire Flow	164 to 337 L/s @ 20 psi	167 to 330 L/s @ 20 psi

- The service pressures from the watermain are expected to range between 60.4 psi to 84.0 psi (416 kPa to 579kPa) under 2016 conditions and between 54.9 psi to 87.2 psi (379 kPa to 601 kPa) under 2031 conditions.
- Pressures will exceed the OBC pressure criteria of 80 psi (550 kPa) and pressure reduction may be required. Pressures will be checked following watermain construction to confirm which units require individual pressure reducing valves.
- The available fire flow meets or exceeds the preliminary fire flow demands utilized for this assessment at the minimum pressure of 140 kPa based on the proposed watermain supply and assumptions made within this report but should be confirmed when additional information becomes available.
- As the development design is at the initial stages, the findings and recommendations in this report are considered preliminary. As more information is available on the watermain and building layout,

all building assumptions and fire flow demands must be reviewed and updated to confirm adequate water supply.

A municipal watermain is proposed throughout the subject lands. The proposed site is expected to be serviced by 200 mm watermain connecting to the existing 200 mm watermain on Country Club Drive. These sizes will be confirmed at detailed design when building assumptions and fire flow demands will be reviewed and updated. The subject mains were designed to achieve service criteria during Average Day, Peak Hour, and fire flow during Maximum Day demand. Based on the above criteria it has been determined that there is sufficient capacity to service the subject lands.

Refer to the Preliminary Water Distribution Modelling Report (MES, March 2020) in ***Appendix F*** *and* ***Figure WM-C*** *the Water Distribution Plan for Area C.*

5.5. STORM SERVICING

5.5.1. PROPOSED MINOR SYSTEM

Storm servicing for the development will conform to City of Burlington standards. Storm sewers will be designed to convey minor system flows resulting from the 5-year storm event for ultimate discharge to Appleby Creek.

The runoff coefficients were based on the proposed land use and the City standard runoff coefficients. The 100-year flows from the subject lands were calculated using the increased runoff coefficients (1.25 x $C_{5\text{-year}}$) as per the City requirements.

The proposed storm sewers within the subject lands will be designed to intercept the minor and major system (100-year) flows as shown on **Figure STM-C**. Flows will be directed to an underground storage tank in the parkette area for control to pre-development 5-year target flows (based on 1999 IDF parameters). Note, the groundwater flows, as intercepted by the foundation weeping tile drain system will be pumped to the surface and be collected by the minor storm system. RJB's Foundation Weeping Tiles & Groundwater Quality Evaluation, referenced in **Appendix B**, indicates groundwater flows for Area C of 2.3 x 10^{-5} m^3/s. The expected groundwater flows are a negligible portion of the 100-year post-development flows and have not been considered in storm sewer sizing. Note, the sump pumps will also protect basements from the high HGL in the minor system storage.

*Refer to **Figure STM-C** for the Storm Drainage Plan and **Appendix G** for the Storm Sewer Design Sheets & PCSWMM model for Area C.*

The external/existing drainage areas to the north (golf course and pipeline corridor) will continue to drain into public infrastructure at the existing outlets (rear yard CBs on existing lots). There will be no net increase in flows at these locations. The site grading and implementation of cutoff swales will ensure that these flows are directed to the appropriate existing outlets and away from the internal plan of subdivision rear yard catchbasins.

5.5.2. PROPOSED MAJOR SYSTEM

All major system flows from Area C will be captured and controlled prior to discharge into the existing storm sewer system within the Country Club Drive ROW as shown on **Figure STM-C**.

The number and size of catchbasins will be determined to ensure the 100-year flow will be entirely captured at detailed design. In the event of complete blockage of inlets, Area C will drain onto Country Club Lane.

*Refer to **Appendix G** for the Overland Flow Route & ROW Calculations for Area C.*

5.6. STORMWATER MANAGEMENT

The existing Millcroft Community was developed in the mid-1980s to design standards and criteria of the time. New works are to conform to current City, Conservation and MECP requirements. The proposed stormwater management system and drainage areas are depicted on **Drawing STM-C**.

5.6.1. QUALITY CONTROL

The proposed development will mimic the existing development and will drain into the existing storm sewer system, which ultimately discharges to Appleby Creek. An oil & grit separator (OG/S) is proposed downstream of the storage tank prior to discharge into the existing sewers on Country Club Drive. CB shields will be provided to augment the water quality and provide a treatment train approach in conjunction with the OG/S. As per standard practice, roof leaders will be discharged to pervious areas to enhance water balance / erosion protection.

Based on the minimum practical orifice size of 50mm, only a small measure of extended detention can be achieved for the 25mm event. It is recommended that ED requirements be waived due to the small catchment sizes and in consideration that Area C drains to a municipal storm sewer.

Preliminary oil/grit separator sizing is included in **Appendix G**.

5.6.2. QUANTITY CONTROL

New development areas will consist of split drainage on the lots. Rear-yard drainage will be intercepted in the rear-yard swales and rear yard catchbasins (RYCBs). Rooftops and front lot drainage will be conveyed into the storm sewer system and post-development flows will be captured and restricted to pre-development levels. The pre-development release rate (5-year pre-development flow based on 1999 IDF parameters) is 96 L/s, which is based on the available capacity remaining at the connection to the 1200mm storm sewer at Arbourfield Road (refer to **Appendix H** for capacity calculations). The post-development (controlled) area was modelled using PCSWMM to evaluate the system's storage and associated hydraulic gradeline. The modelled storage volume and orifice size was iterated to achieve the target flow, and the HGL has been confirmed to be below the road. Sump pumps will be required on all lots, subject to refinements at detailed design.

A 125 mm diameter orifice tube is required to meet the pre-development flows. In conjunction with the minor system sewer (1200mm), a storage tank will provide flow attenuation. The proposed tank will be placed in the ROW's cul-de-sac and provide 350m^3 of storage. The 1200mm pipe provides approximately 151m^3 of storage (not including structures). The total modelled storage of 501m^3 is sufficient to ensure the target flows in the existing sewer system are not exceeded. Calculations for the storage and orifice can be found in **Appendix H**. The proposed SWM drainage areas and the storm servicing strategy are included in **Figure STM-C**.

A summary of the flows for Area C can be found in **Appendix H.**

Note, the proposed orifice-controlled flows at Outlet #2C over-control for the portion of proposed uncontrolled flows that exceed the existing uncontrolled flows at Outlet #4C.

*Refer to **Appendix H** for the SWM calculations. Refer to **Figure STM-C** for the storm servicing strategy.*

5.6.3. WATER BALANCE

RJB prepared revised detailed water balance calculations (March 2022) in a separate memo to the hydrogeology report (November 2020) as informed by land use plan **Drawing LU-C**. Both the memo and report are provided in **Appendix B**.

Development of the Subject Property will reduce infiltration to the shallow groundwater flow system. The groundwater balance calculations suggest that, without mitigation, a potential reduction of 28% of the average annual groundwater infiltration could occur as per the memo (RJB, 2022) in **Appendix B**.

In order to promote infiltration while mitigating against high groundwater conditions, the proposed SWM strategy recommends directing a portion of available roof areas to rear yards (disconnection of roof leaders) and an increased topsoil depth of 300 mm. The revised water balance calculations suggest that implementing increased topsoil depth and downspout disconnection to direct roof runoff to pervious areas across the proposed development will effectively maintain pre-development infiltration volumes.

*Refer to **Appendix B** for the Hydrogeological Report Investigation by RJB (November 2020) and Revised Water Balance Calculations by RJB (March 2022).*

5.7. IMPLEMENTATION

The limit of excavation for connections to external services is depicted for Area C in the Construction Management and Mobility Plan as **Figure CMMP-C**.

5.8. NOISE AND VIBRATION FEASIBILITY STUDY

HGC completed a Noise and Vibration Feasibility Study for the proposed development, and the results of the study indicate that the proposed residential development, inclusive of Area C, is feasible.

*Refer to **Appendix K** for the Noise and Vibration Feasibility Study completed by HGC (February 2022).*

5.9. SUMMARY

The proposed development can be adequately serviced through a combination of existing and proposed municipal infrastructure. In summary:

- Sanitary Servicing will be accomplished through the construction of local sanitary sewers which will connect to the existing 200 mm diameter municipal sanitary sewer within Arbourfield Drive.
- Water servicing for domestic potable and fire protection will be through connections to the existing 200 mm diameter watermain within Country Club Drive and the construction of local watermains.
- Storm drainage will include the construction of local storm sewers and infrastructure designed to capture, convey, control all flows up to the 100-year storm to existing sewers in Arbourfield Drive.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks to maximize reuse of soils within the property and avoid the need for retaining walls.
- Stormwater quantity control will be provided via pipe and an underground tank storage. Proposed flows will be controlled to the pre-development release rate via 501 m^3 of combined storage (350m^3 tank + 151m^3 of 1200mm storm sewer) pipe and a 125 mm diameter orifice tube. Major system flows from Area C will be captured in sewers and directed to the existing storm network.
- Quality control will be provided through the use of oil/grit separators at the downstream end of Area C's control manhole, as well as CB Shields internally.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

6 AREA D

LOCATION: East side of Millcroft Park Drive, south of Latimer Crescent and north of Turnberry Road.

AREA: 0.76 ha

CURRENT LAND USE:
Hole No. 16



6.1. EXISTING CONDITIONS

6.1.1. LAND USE & TOPOGRAPHY

The site is currently used as Hole 16 for the Millcroft Golf Club. The golf course will remain operational, and the Hole will be re-designed to accommodate future development. Currently, the site is covered with greens/fairways/and bunkers, low-lying vegetation and trees, and golf cart trails.

R-PE completed topographical surveys of the subject lands in September 2019.

The site generally falls from northeast to the southwest with a maximum grade change of approximately 2.0 m.

Figure EX-D *illustrates the existing site features, topography, and drainage patterns.*

6.1.2. SOIL CONDITIONS

Millcroft Greens Corporation retained DSCL to undertake the preliminary geotechnical investigation for the proposed development.

The geotechnical report indicates the following subsurface conditions:

- Topsoil – 50 mm to 150 mm thick surficial layer of topsoil
- Fill material – ranging from 0.8 m to 1.1 m below grade consisting mainly of clayey silt, trace rootlets and trace topsoil.
- Upper soils – Clayey silt to sandy silt till
- Lower soils – Sandy silt till
- Bedrock encountered at depths ranging from 3.7 mbgs to 4.6 mbgs

DSCL's preliminary investigation indicated that the soil stratigraphy of the subject property consists of fill underlain by native deposits of clayey silt, sandy silt till, clayey silt till and a shale complex overlying shale bedrock. This report also establishes that no groundwater was found in the boreholes during drilling or upon completion of drilling. The stabilized groundwater levels observed in the monitoring wells installed in the boreholes range from 0.5m (BH19-201) to 4.5m (BH19-202).

Highly weathered shale bedrock of the Queenston Formation was encountered below depths ranging from 3.7m to 4.6m, however was not proven through core drilling. Methane gas is anticipated in the bedrock, and appropriate care and monitoring is essential in all confined bedrock excavations. For the purposes of this investigation, it is assumed that any remediation work which may be required to deal with potential contamination will be completed prior to earthworks and servicing.

The complete study in **Appendix A** provides a detailed discussion of geotechnical site conditions. The report further discusses recommendations pertaining to site alteration procedures as well as the construction of roads, sewers, foundations, and drainage systems based on the subsoil conditions.

6.1.3. GEOTECHNICAL RECOMMENDATIONS

The geotechnical report recommended the following minimum pavement makeup for all internal site roads based on the assumed residential minor local or local traffic usage conditions:

- 40 mm HL3 Asphaltic Concrete
- 50 mm HL8 Asphaltic Concrete
- 150 mm Granular 'A'
- 350 mm Granular 'B'

The Area D proposed design grade at road surface will be at Elevation 147.0 to 147.7m. Assuming basement depth of 3m, the basement floors will be at about Elev. 144 to 144.7m. Bedrock elevation in the boreholes varies from 141.7 to 142.8m as per the geotechnical report prepared by DS Consultants Ltd in **Appendix A**.

Area D basements floor will generally be constructed in glacial till deposits. However, in any areas where basements are to be constructed below the bedrock surface, methane gas monitoring should be carried out during excavation. If methane gas is encountered during excavation, gas venting systems may be required depending upon the gas concentration readings.

*Refer to **Appendix A** for the Geotechnical Report for Areas A through D as completed by DSCL (December 2020).*

6.1.4. HYDROGEOLOGY RECOMMENDATIONS

A hydrogeological report investigation was completed in November 2020 by RJB and has been included in **Appendix B**.

The field and laboratory work included the installation of three monitoring wells across the subject property in November 2018. Groundwater levels across Area D have been measured monthly from January 2019 through to November 2021 and will continue to be monitored quarterly thereafter.

- Groundwater levels in monitoring wells were observed to have varied about 137m above sea level (masl), with seasonal fluctuations ranging from 0.5m to 4.0m range.

- Shallow groundwater levels typically exhibit seasonal fluctuations, with the highest elevations generally observed in the spring and the lowest in the summer months. To date, seasonal high levels were observed in January 2020 due to the mild winter conditions.

*Refer to **Appendix B** for the Hydrogeological Report Investigation completed by RJB (November 2020).*

A conventional foundation weeping tile drain system and sump pumps for the proposed detached lots within Area D will address seasonal high groundwater elevations and hydraulic gradeline. Based on the Foundation Weeping Tiles and Groundwater Quality Evaluation by RJB provided in **Appendix B**, groundwater flows are estimated to be 1.1×10^{-5} m^3/s, which is seven (7) orders of magnitude less than the 100-year flows that will be captured and conveyed by the minor system. These flows will not impact the allowable release rate or storage volumes. Groundwater samples were collected and submitted for analysis to compare to the City's storm sewer discharge criteria. The results of the groundwater analysis in comparison to the City's storm sewer discharge criteria and discussion are presented by RJB in **Appendix B**. For discussion on the estimated groundwater flow impacts to the storm sewer system, refer to **Section 6.5.1**.

*Refer to **Appendix B** for the Foundation Weeping Tiles & Groundwater Quality Evaluation completed by RJB (February 2022).*

6.1.5. EXISTING MINOR SYSTEM DRAINAGE

As shown on **Figure EX-D**, a DICB on the subject site connects to a 375mm storm pipe that discharges to the 1050 mm storm sewer on Millcroft Park Drive which ultimately discharges to Appleby Creek.

6.1.6. EXISTING MAJOR SYSTEM DRAINAGE

The majority of Area D generally drains from the interior of the site to DICB at the exterior limit, ultimately making its way to the existing storm sewer located within the Millcroft Park Drive ROW. Area D consists of one (1) existing drainage subcatchment. The existing site drainage is depicted in **Figure EX-D**, summarized in **Table 6-1** and further discussed below.

Table 6-1 Existing Drainage Areas - Area D

Area D			External to Area D		
Area ID	Area (ha)	RC (-)	Area ID	Area (ha)	RC (-)
S01-D	0.76	0.30	E01-D	0.55	0.30
			E02-D	0.27	0.40
Total	0.76	0.30	Total	0.82	0.33

- Catchment S01-D currently sheet drains southwest overland to an existing DICB as Outlet #1D, ultimately connected to the storm sewer within the Millcroft Park Drive ROW.

The total existing drainage from Area D is 0.76 ha, with an average runoff coefficient of 0.30. There are two (2) external drainage sub-catchments that are conveyed through Area D.

- Catchment E01-D consists of the existing golf course, and currently, sheet drains southwest overland to an existing DICB as Outlet #1D, ultimately connected to the storm sewer within the Millcroft Park Drive ROW.
- Catchment E02-D consists of existing rear lot residential along Latimer Crescent, and currently, sheet drains southwest overland to catchment E01-D, ultimately draining to an existing DICB as Outlet #1D, connected to the storm sewer within the Millcroft Park Drive ROW.

The total external drainage conveyed through Area D is 0.82 ha, with an average runoff coefficient of 0.33.

*Please refer to the existing site drainage for Area D as depicted in **Figure EX-D** and calculations in **Appendix H**.*

6.2. DEVELOPMENT CONCEPT

6.2.1. LAND USE & RIGHT-OF-WAYS

As shown conceptually in **Figure SK-74** & calculated in the **Draft Plan of Subdivision**, by Gerrard Design & Wellings Planning Consultants Inc. respectively, the proposed Area D development consists of the following land use components:

- Detached Homes: Lots 84-90
- Open Space: Blocks 107 & 108
- ROW: Block 110

The proposed developments will be advanced through the draft plan of subdivision and subsequent condominium approval processes.

Area D will consist of 7 residential lots, and Block 110 includes a 9.1 m ROW with 8.1 m pavement width (8.7m; as measured from the face of curbs) as illustrated in road cross-section **SEC-D1**. The 9.1m ROW includes but is not limited to 6.0m two-way travel lanes and a 2.4m parking lane. Also, a 1.5m trail abuts the interface between the TCPL 7.0m setback (Block 107) & 9.1 m ROW (Block 110). Residential-detached lot depths range from 34.0 m to 40.0 m with a width of approximately 15.24 m. Two custom lots are proposed as well. The non-developable area will consist of open space as Blocks 107 & 108.

The above-mentioned ROW within Area D's Block 111 has been developed in keeping with the master plan vision to support the proposed surface treatment and will be further refined in consultation with the required approval agencies & utility companies. As such, the cross-sections are conceptual and are to be discussed further with the City of Burlington. Note, the ROW cross-section was developed in consultation with RTG Systems Inc with respect to utilities.

*Refer to **Drawing GP-D** for the Area D general plan.*

Note, many of the lots include a 6.0 m buffer within the proposed residential lots adjacent to the existing residential development south of Area D. This buffer will be landscaped to act as a privacy feature between existing and proposed residential lots and is illustrated in cross-section view by NAK Design Strategies as part of the Conceptual Open Space package submitted under separate cover with this submission. Existing trees within the 6.0m buffer be preserved wherever possible as illustrated in Arborist Report Figure TP-4 by Beacon included in **Appendix L**. Note, a tree preservation plan is included in **Drawing GR-D** with respect to City owned street trees only. For details on ownership and condition ratings, refer to Arborist Report Figure TP-4.

*Refer to **Appendix L** for the Arborist Report for Area D.*

6.2.2. GRADING

The proposed conceptual grading for the development will be designed in accordance with City of Burlington standards. Grading is generally governed by the existing boundary conditions. Site grading has also been designed to ensure that adequate cover over proposed services is maintained. No external grading works are proposed.

A preliminary grading concept plan has been prepared for the subject lands based on the following engineering constraints:

- Storm outlet elevations
- Major system drainage paths
- Provision of minimum cover over services
- Proposed road patterns and land use
- Elevations along boundary roads, property lines and the CNR slopes
- Application of the City of Burlington standards

The grading plan, **Drawing GR-D**, is consistent with the City standards. In general, grading of the proposed plan of subdivision adjacent to the surrounding development and roads matches the existing grades as appropriate. Additionally, significant effort has been exercised to ensure that the grades of the proposed lands transitions gradually from northeast to southwest and that proposed grades minimize impacts to existing trees within the 6.0 m buffer adjacent to existing properties. Note, trees for preservation and associated protection fencing by Beacon is included on **Drawing GR-D** as per Drawing

TP-4 included in **Appendix L**. The Preliminary Landscape Design for Area D, as prepared by NAK Design Strategies included under separate cover in this submission, is refleced on **Drawing GR-D**.

*Refer to **Drawing GR-D** for the grading plan for Area D.*

6.3. SANITARY SERVICING

The sanitary servicing criteria for Area D is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 6-2**.

Table 6-2 Sanitary Servicing Criteria - Area D

Parameter	Value	Parameter	Value
Average Domestic Flow	275 L/capita/day	Population (Single Family)	3.52 person per unit
Infiltration / Inflow	0.286 L/s/ha	Unit Count (#)	7
Peaking Factor (M)	Harmon Formula	Area (ha)	0.75
Minimum	4.50		
Maximum	2.00		

6.3.1. EXISTING

There is currently a 525mm sanitary sewer on Millcroft Park Drive.

6.3.2. PROPOSED

Based on a review of the available as-constructed information (refer to **Appendix D**), the subject lands can be serviced by gravity sewers.

Wastewater infrastructure will be designed in accordance with the latest Region of Halton standards and specifications.

The proposed sanitary sewers that will service the subject lands will discharge to the existing 525 mm sanitary sewer on Millcroft Park Drive.

Sanitary sewer design sheets have been prepared and used to size proposed sanitary sewers for the proposed development. The population is estimated using the population equivalent densities per the Halton Region standards.

Population Estimates and Sanitary Design Sheets can be found in **Appendix E**. See **Table 6-3** below for a summary of the sanitary servicing for Area D.

Table 6-3 Sanitary Servicing Summary - Area D

Area	Sanitary Area (ha)	# of Units	Population Equivalent Density (ppu) Population		Dry Weather Flow (L/s @ 275 L/c/d)	Infiltration / Inflow (L/s @ 0.286 L/s/ha)	Peaking Factor	Total Sanitary Flow (L/s)	Connecting Manhole
D	0.76	7	3.52	25	0.08	0.21	4.37	0.56	EX.MH 36

Based on the above criteria and the InfoSewer capacity analysis, it has been determined that there is ample capacity to service the subject lands. There is over 100 L/s of reserve capacity in the sanitary sewer to accommodate this flow.

*Refer to **Appendix D** for the as-constructed information with regards to the existing sanitary system. Refer to **Figure SAN-D** for the sanitary servicing plan for Area D and **Appendix E** for the Sanitary Sewer Design Sheets and InfoSewer capacity analysis.*

6.4. WATER SERVICING

The water servicing criteria and flow demands for the proposed development is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 6-4**.

Table 6-4 Water Servicing and Flow Demands

Water Design Factors		Water Demand (L/s)[1]	
Average Daily Demand	275 L/capita/day	Average Day Demand	1.75
Maximum Daily Demand Peaking Factor	2.25	Maximum Day Demand	3.94
Peak Hourly Demand Peaking Factor	4.00	Peak Hour Demand	7.01

[1]Refer to the Hydraulic Analysis Report by MES in **Appendix F** for water demands as calculated per equivalent population

6.4.1. EXISTING

The subject site is located within Pressure Zone B3 in Halton Region's water distribution system. This is based on the service elevation range for the area. It should be noted that the Region is planning to alter the pressure districts in Halton, however, Zone B3 is not affected in this change. Existing water distribution infrastructure in the vicinity of the site includes:

- 300 mm watermain on Millcroft Park Drive

6.4.2. PROPOSED

MES was retained to carry out a detailed hydraulic analysis of the proposed development's impact on both existing and proposed infrastructure. The analysis includes design years 2016 and 2031. The analysis was based on Region of Halton Water and Wastewater Linear Design Manual, October 2019. The watermain to service the proposed development can achieve these hydraulic requirements by prescribing to the design criteria as summarized in **Table 6-5** and further discussed below.

Table 6-5 Modelled Service Pressure and Fire Flows

	Modelled Scenarios	
	2016	2031
Minimum Service Pressure	416 kPa (60.4 psi)	379 kPa (54.9 psi)
Maximum Service Pressure	579 kPa (84.0 psi)	601 kPa (87.2 psi)
Maximum Day + Fire Flow	164 to 337 L/s @ 20 psi	167 to 330 L/s @ 20 psi

- The service pressures from the watermain are expected to range between 60.4 psi to 84.0 psi (416 kPa to 579kPa) under 2016 conditions and between 54.9 psi to 87.2 psi (379 kPa to 601 kPa) under 2031 conditions.

- Pressures will exceed the OBC pressure criteria of 80 psi (550 kPa) and pressure reduction may be required. Pressures will be checked following watermain construction to confirm which units require individual pressure reducing valves.

- The available fire flow meets or exceeds the preliminary fire flow demands utilized for this assessment at the minimum pressure of 140 kPa based on the proposed watermain supply and assumptions made within this report but should be confirmed when additional information becomes available.

- As the development design is at the initial stages, the findings and recommendations in this report are considered preliminary. As more information is available on the watermain and building layout, all building assumptions and the fire flow demands must be reviewed and updated to confirm that the water supply is adequate.

A municipal watermain is proposed throughout the subject lands. The proposed site is expected to be serviced by a 200 mm watermain connecting to the 300 mm watermain on Millcroft Park Drive. These sizes will be confirmed at detailed design when building assumptions and fire flow demands will be reviewed and updated. The subject mains were designed to achieve service criteria during Average Day, Peak Hour, and fire flow during Maximum Day demand. Based on the above criteria it has been determined that there is sufficient capacity to service the subject lands.

*Refer to the Preliminary Water Distribution Modelling Report (MES, March 2020) in **Appendix F** and **Figure WM-D** for the Water Distribution Plan for Area D.*

6.5. STORM SERVICING

6.5.1. PROPOSED MINOR SYSTEM

Storm servicing for the development will conform to City of Burlington standards. Storm infrastructure will be designed to capture, convey, and control the 100-year post-development flow to the allowable minor system flows at Millcroft Park Drive resulting from the 5-year storm event. The minor system ultimately discharges to Appleby Creek.

The runoff coefficients were based on the proposed land use and the City standard runoff coefficients. The 100-year flows from the subject lands were calculated using the increased runoff coefficients (1.25 x C5-year) as per the City requirements.

The proposed storm sewers within the subject lands will be designed to intercept all flows up to the 100-year storm as shown on **Figure STM-D**. at detailed design, suitable inlets will be designed and located on the site to ensure that the 100-year flows are entirely captured. Note, the groundwater flows, as intercepted by the foundation weeping tile drain system will be pumped to the surface and be collected by the minor storm system. RJB's Foundation Weeping Tiles & Groundwater Quality Evaluation, referenced in **Appendix B**, indicates groundwater flows for Area D are 1.1 x 10^{-5} m^3/s. The expected groundwater flows are a negligible portion of the 100-year post-development flows and have not been considered in storm sewer sizing. Note, the sump pumps will also protect basements from the high HGL in the minor system storage.

*Refer to **Figure STM-D** for the Storm Drainage Plan and **Appendix G** for the Storm Sewer Design Sheets & PCSWMM model for Area D.*

6.5.2. PROPOSED MAJOR SYSTEM

The major system flows from Area D will be captured into the pipe and directed to Millcroft Park Drive The major system drainage patterns are shown on **Figure STM-D**. In the event of total CB blockage, emergency flow will drain to Millcroft Park Drive.

6.6. STORMWATER MANAGEMENT

The existing Millcroft Community was developed in the mid-1980s to design standards and criteria of the time. New works are to conform to current City, Conservation and MECP requirements. The proposed stormwater management system and drainage areas are depicted on **Drawing STM-D.**

6.6.1. QUALITY CONTROL

The proposed development will mimic the existing development and will drain into the existing storm sewer system, ultimately discharging to Appleby Creek. An oil & grit separator (OG/S) is proposed downstream of the storage tank to treat runoff prior to discharging into the existing sewers on Millcroft Park Drive. Note that CB shields will provide a water quality treatment train approach in conjunction with the OG/S.

Based on the minimum practical orifice size of 50mm, only a small measure of extended detention can be achieved for the 25mm event. It is recommended that ED requirements be waived due to the small catchment sizes and in consideration that Area C drains to a municipal storm sewer.

Preliminary oil/grit separator sizing is included in **Appendix G**.

6.6.2. QUANTITY CONTROL

The general approach applied to new development involves control of the post-development flow rates to pre-development levels.

New development areas will consist of split drainage on the lots. Rear-yard drainage will be intercepted in the rear-yard swales and rear yard catchbasins (RYCBs). Rooftops and front lot drainage will be conveyed into the storm sewer system, and post-development flows will be captured and restricted to pre-development levels. The pre-development release rate (5-year predevelopment flow based on 1999 IDF parameters is 56 L/s) is based on the available capacity remaining at the connection to Millcroft Park Drive.

A 120 mm diameter orifice tube is required to meet the pre-development flows. In conjunction with the minor system sewer superpipe (1050 mm), a storage tank will provide flow attenuation. The proposed tank will be placed in the ROW's cul-de-sac and will provide 133m^3. The 1050 mm pipe provides approximately 90m^3 of storage (not including structures). The total modelled storage of 223m^3 is sufficient to ensure the target flow is not exceeded. Calculations for the storage and orifice can be found in **Appendix H**. The proposed SWM drainage areas and the storm servicing strategy are included in **Figure STM-D**.

A summary of the flows for Area D can be found in **Appendix H.**

Note, the proposed orifice-controlled flows over-control for the portion of proposed uncontrolled flows that exceed the existing uncontrolled flows from the external drainage area north of Area D.

*Refer to **Appendix H** for the SWM calculations. Refer to **Figure STM-D** for the storm servicing strategy.*

A summary of the flows for Area D can be found in **Appendix H.**

The proposed SWM drainage areas and the storm servicing strategy is shown on **Figure STM-D**.

*Refer to **Figure PR-D** for the stormwater management plan for Area D. Refer to **Appendix H** for the SWM calculations. Refer to **Figure STM-D** for the storm servicing strategy.*

6.6.3. WATER BALANCE

RJB prepared revised detailed water balance calculations (March 2022) in a separate memo to the hydrogeology report (November 2020) as informed by land use plan **Drawing LU-D**. Both the memo and report are provided in **Appendix B**.

Development of the Subject Property will reduce infiltration to the shallow groundwater flow system. The groundwater balance calculations suggest that, without mitigation, a potential reduction of 28% of the average annual groundwater infiltration could occur as per the memo (RJB, 2022) in **Appendix B**.

In order to promote infiltration while mitigating against high groundwater conditions, the proposed SWM strategy recommends directing a portion of available roof areas to rear yards (disconnection of roof leaders) and an increased topsoil depth of 300 mm. The revised water balance calculations suggest that implementing increased topsoil depth and downspout disconnection to direct roof runoff to pervious areas across the proposed development will effectively maintain pre-development infiltration volumes.

*Refer to **Appendix B** for the Hydrogeological Report Investigation by RJB (November 2020) and Revised Water Balance Calculations by RJB (March 2022).*

6.7. IMPLEMENTATION

The limit of excavation for connections to external services is depicted for Area D in the Construction Management and Mobility Plan as **Figure CMMP-D**.

6.8. NOISE AND VIBRATION FEASIBILITY STUDY

A Noise and Vibration Feasibility Study was completed by HGC for the proposed development and the results of the study indicate that the proposed residential development, inclusive of Area D, is feasible.

*Refer to **Appendix K** for the Noise and Vibration Feasibility Study completed by HGC (February 2022).*

6.9. SUMMARY

The proposed development can be adequately serviced through a combination of existing and proposed municipal infrastructure. In summary:

- Sanitary Servicing will be accomplished through the construction of local sanitary sewers, which will connect to the existing 525 mm diameter municipal sanitary sewer within Millcroft Park Drive.
- Water servicing for domestic potable and fire protection will be through connections to the existing 300 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.
- Storm drainage will include the construction of local storm sewers and infrastructure designed to capture, convey, control all flows up to the 100-year storm to existing sewers in Millcroft Park Drive.
- Grading will be in accordance with City of Burlington requirements and minimize on-site earthworks to maximize the reuse of soils within the property and avoid the need for retaining walls.
- Stormwater quantity control will be provided via pipe and an underground tank storage. Proposed flows will be controlled to the pre-development release rate via 223 m^3 of combined storage (133m^3 tank + 90m^3 of 1050mm storm sewer) pipe and a 120 mm diameter orifice tube. Major system flows from Area D will be captured in sewers and directed to the existing storm network.
- Quality control will be provided through the use of oil/grit separators at the downstream end of Area D's control manhole, as well as CB Shields internally.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

7 CONCLUSION

It can be noted that each of the four (4) development areas (three study areas below: Areas A & B, Area C and Area D) can be adequately serviced through a combination of existing and proposed municipal infrastructure. In summary:

Areas A & B:

- Sanitary servicing will be accomplished through the construction of local sanitary sewers which will connect to the existing 525 mm diameter municipal sanitary sewer within Millcroft Park Drive.
- Water servicing for domestic potable and fire protection will be through connections to the existing 300 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.
- Storm drainage will include the construction of local storm sewers and infrastructure designed to capture, convey, control all flows up to the 100-year storm to Appleby Creek and maintain external flow conveyance. The Hadfield Court drainage will drain to Appleby Creek. The Parklane Crescent drainage will be diverted from Hadfield Court and conveyed directly to the reconfigured irrigation pond via a new storm sewer in the servicing corridor north of the CNR.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks to maximize reuse of soils within the property and minimize the use of retaining walls.
- Stormwater quantity control will be provided via superpipes and underground storage vaults. A hydraulic grade line analysis and comparison of existing and proposed drainage areas has demonstrated that there will be no increase in flows or water levels on the existing properties.
- Quality control will be provided through the use of oil/grit separators and CB Shields. A biofiltration LID measure can be implemented in the “island” at the south end of Area A.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

Area C:

- Sanitary servicing will be accomplished through the construction of local sanitary sewers which will connect to the existing 200 mm diameter municipal sanitary sewer within Arbourfield Drive.
- Water servicing for domestic potable and fire protection will be through connections to the existing 200 mm diameter watermain within Country Club Drive and the construction of local watermains.
- Storm drainage will include the construction of local storm sewers and infrastructure designed to capture, convey, control all flows up to the 100-year storm to existing sewers in Arbourfield Drive.
- Grading will be in accordance with City of Burlington requirements and minimize on-site earthworks to maximize the reuse of soils within the property and avoid the need for retaining walls.
- Stormwater quantity control will be provided via pipe and an underground tank storage. Proposed flows will be controlled to the pre-development release rate via 501 m^3 of combined storage (350m^3 tank + 151m3 of 1200mm storm sewer) pipe and a 125 mm diameter orifice tube. Major system flows from Area C will be captured in sewers and directed to the existing storm network.

- Quality control will be provided through the use of oil/grit separators at the downstream end of Area C's control manhole, as well as CB Shields internally.

- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

Area D:

- Sanitary servicing will be accomplished through the construction of local sanitary sewers which will connect to the existing 525 mm diameter municipal sanitary sewer within Millcroft Park Drive.

- Water servicing for domestic potable and fire protection will be through connections to the existing 300 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.

- Storm drainage will include the construction of local storm sewers and infrastructure designed to capture, convey, control all flows up to the 100-year storm to existing sewers in Millcroft Park Drive.

- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks to maximize reuse of soils within the property and avoid the need for retaining walls.

- Stormwater quantity control will be provided via pipe and an underground tank storage. Proposed flows will be controlled to the pre-development release rate via 223 m^3 of combined storage (133m^3 tank + 90m^3 of 1050mm storm sewer) pipe and a 120 mm diameter orifice tube. Major system flows from Area D will be captured in sewers and directed to the existing storm network.

- Quality control will be provided through the use of oil/grit separators at the downstream end of Area D's control manhole, as well as CB Shields internally.

- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

Should you have any questions regarding the information provided in this report, please do not hesitate to contact the undersigned.

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FIGURES & DRAWINGS

APPENDIX A

GEOTECHNICAL FEASIBILITY STUDY (DSCL)

APPENDIX B

HYDROGEOLOGICAL REPORT
(RJB)

APPENDIX C

FOUNDATION WEEPING TILES & GROUNDWATER QUALITY EVALUATION (RJB)

APPENDIX D

AS-CONSTRUCTED DRAWINGS
(City of Burlington)

APPENDIX E

SANITARY SEWER DESIGN SHEETS

APPENDIX F

PRELIMINARY WATER DISTRIBUTION MODELLING REPORT (MES)

APPENDIX G

STM CALCULATIONS

Storm Sewer Design Sheets
OGS Sizing
Right-of-Way (ROW) Calculations

APPENDIX H

SWM DESIGN CALCULATIONS

Existing Drainage Area Breakdown
PCSWMM Modelling

APPENDIX I

CHANNEL RE-ALIGNMENT & HYDRAULICS

APPENDIX J

PERMIT TO TAKE WATER APPLICATION (RJB)

APPENDIX K

NOISE STUDY
(HGC)

APPENDIX L

ARBORIST REPORT
(BEACON)

APPENDIX M

WOOD REPORT EXCERPT
(formerly AMEC)