URBANTECH®

FUNCTIONAL SERVICING REPORT

MILLCROFT GOLF CLUB

CITY OF BURLINGTON

REGION OF HALTON

PREPARED FOR
MILLCROFT GREENS CORPORATION

Urbantech File No.: 17-526

1ST SUBMISSION – DECEMBER 2020

Urbantech® Consulting, A Division of Leighton-Zec Ltd. | 3760 14th Avenue, Suite 301 · Markham · ON · L3R 3T7 | 905.946.9461
urbantech.com

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 1

2 INTRODUCTION 2

2.1. BACKGROUND 2
2.2. SUBJECT PROPERTY 2

3 DESIGN CRITERIA AND OBJECTIVES 4

3.1. SANITARY DESIGN CRITRIA 4
3.2. WATER DESIGN CRITERIA 4
3.3. STORM DESIGN CRITERIA 5
3.4. STORMWATER MANAGEMENT DESIGN CRITERIA 7
3.5. GRADING DESIGN OBJECTIVES 7

4 AREAS A & B 9

4.1. EXISTING CONDITIONS 9
4.1.1. LAND USE & TOPOGRAPHY 9
4.1.2. SOIL CONDITIONS 9
4.1.3. GEOTECHNICAL RECOMMENDATIONS 10
4.1.4. HYDROGEOLOGY RECOMMENDATIONS 10
4.1.5. EXISTING MINOR SYSTEM DRAINAGE 11
4.1.6. EXISITING MAJOR SYSTEM DRAINAGE – AREA A 11
4.1.7. EXISTING MILL POND 12
4.1.8. EXISITING MAJOR SYSTEM DRAINAGE – AREA B 13
4.2. DEVELOPMENT CONCEPT 14
4.2.1. LAND USE 14
4.2.2. GRADING 14
4.2.3. RIGHT-OF-WAYS 15
4.3. SANITARY SERVICING 16
4.3.1. EXISTING 16
4.3.2. PROPOSED 16
4.4. WATER SERVICING 18
4.4.1. EXISTING 18
4.4.2. PROPOSED 18
4.5. STORM DRAINAGE 19
4.5.1. PROPOSED MINOR SYSTEM 19
4.5.2. PROPOSED MAJOR SYSTEM 19
4.5.3. CHANNEL REALIGNMENT / POND RETROFIT 20
4.5.4. EXISTING & PROPOSED FLOOD ELEVATIONS 21
4.5.5. RIPARIAN STORAGE ANALYSIS 22
4.6. STORMWATER MANAGEMENT 24
4.6.1. DESIGN 24
4.6.2. QUALITY CONTROL 24
4.6.3. QUANTITY CONTROL 25
4.6.4. WATER BALANCE 26

4.7. IMPLEMENTATION 27
4.8. NOISE AND VIBRATION FEASIBILITY STUDY 27
4.9. SUMMARY 28

5 AREA C 29

5.1. EXISTING CONDITIONS 29
5.1.1. LAND USE & TOPOGRAPHY 29
5.1.2. SOIL CONDITIONS 29
5.1.3. GEOTECHNICAL RECOMMENDATIONS 30
5.1.4. HYDROGEOLOGY RECOMMENDATIONS 30
5.1.5. EXISTING MINOR SYSTEM DRAINAGE 31
5.1.6. EXISTING MAJOR SYSTEM DRAINAGE 31
5.2. DEVELOPMENT CONCEPT 32
5.2.1. LAND USE 32
5.2.2. GRADING 32
5.2.3. RIGHT-OF-WAYS 33
5.3. SANITARY SERVICING 34
5.3.1. EXISTING 34
5.3.2. PROPOSED 34
5.4. WATER SERVICING 36
5.4.1. EXISTING 36
5.4.2. PROPOSED 36
5.5. STORM SERVICING 38
5.5.1. PROPOSED MINOR SYSTEM 38
5.5.2. PROPOSED MAJOR SYSTEM 38
5.6. STORMWATER MANAGEMENT 39
5.6.1. DESIGN 39
5.6.2. QUALITY CONTROL 39
5.6.3. QUANTITY CONTROL 39
5.6.4. WATER BALANCE 40
5.7. IMPLEMENTATION 41
5.8. NOISE AND VIBRATION FEASIBILITY STUDY 41
5.9. SUMMARY 42

6 AREA D 43

6.1. EXISTING CONDITIONS 43
6.1.1. LAND USE & TOPOGRAPHY 43
6.1.2. SOIL CONDITIONS 43
6.1.3. GEOTECHNICAL RECOMMENDATIONS 44
6.1.4. HYDROGEOLOGY RECOMMENDATIONS 44
6.1.5. EXISTING MINOR SYSTEM DRAINAGE 46
6.1.6. EXISTING MAJOR SYSTEM DRAINAGE 46
6.2. DEVELOPMENT CONCEPT 46
6.2.1. LAND USE 46
6.2.2. GRADING 47
6.2.3. RIGHT-OF-WAYS 47
6.3. SANITARY SERVICING 49
6.3.1. EXISTING 49
6.3.2. PROPOSED 49
6.4. WATER SERVICING 50
6.4.1. EXISTING 50
6.4.2. PROPOSED 50
6.5. STORM SERVICING 52

6.5.1. PROPOSED MINOR SYSTEM 52
6.5.2. PROPOSED MAJOR SYSTEM 52
6.6. STORMWATER MANAGEMENT 53
6.6.1. DESIGN 53
6.6.2. QUALITY CONTROL 53
6.6.3. QUANTITY CONTROL 53
6.6.4. WATER BALANCE 54
6.7. IMPLEMENTATION 55
6.8. NOISE AND VIBRATION FEASIBILITY STUDY 55
6.9. SUMMARY 56

7 AREA E 57

7.1. EXISTING CONDITIONS 57
7.1.1. LAND USE & TOPOGRPAHY 57
7.1.2. SOIL CONDITIONS 57
7.1.3. GEOTECHNICAL RECOMMENDATIONS 58
7.1.4. HYDROGEOLOGY RECOMMENDATIONS 58
7.1.5. EXISTING MINOR SYSTEM DRAINAGE 60
7.1.6. EXISTING MAJOR SYSTEM DRAINAGE 60
7.2. DEVELOPMENT CONCEPT 61
7.2.1. LAND USE 61
7.2.2. GRADING 61
7.3. SANITARY SERVICING 62
7.3.1. EXISTING 62
7.3.2. PROPOSED 62
7.4. WATER SERVICING 64
7.4.1. EXISTING 64
7.4.2. PROPOSED 64
7.5. STORM SERVICING 66
7.5.1. PROPOSED MINOR SYSTEM 66
7.5.2. PROPOSED MAJOR SYSTEM 66
7.6. STORMWATER MANAGEMENT 67
7.6.1. DESIGN 67
7.6.2. QUALITY CONTROL 67
7.6.3. QUANTITY CONTROL 67
7.6.4. WATER BALANCE 67
7.7. IMPLEMENTATION 69
7.8. NOISE AND VIBRATION FEASIBILITY STUDY 69
7.9. SUMMARY 70

8 EROSION & SEDIMENT CONTROL 71

9 CONCLUSION 72

TABLE OF TABLES

Table 2-1 Study Areas 3
Table 3-1 Residential Dry Weather Flow 4
Table 3-2 Average Daily Consumption Rates for each Land Use 5
Table 3-3 Runoff Coefficients (City of Burlington[1]) 6
Table 3-4 IDF Values for City of Burlington 6
Table 4-1 Existing Drainage Areas - Area A 11
Table 4-2 Average Daily Turf Irrigation Requirements 12
Table 4-3 Existing Drainage Areas - Area B 13
Table 4-4 Sanitary Servicing Criteria - Areas A and B 16
Table 4-5 Sanitary Servicing Summary - Area A & Area B 16
Table 4-6 Water Servicing Criteria and Flow Demands 18
Table 4-7 Modelled Service Pressures and Fire Flows 18
Table 4-8 Existing and Proposed Flood Elevations 21
Table 4-9 Summary of Riparian Storage Volumes 22
Table 4-10 Proposed Reconfigured Offline Pond Characteristics 24
Table 4-11 SWM Summary for Area A & Area B 25
Table 4-12 Offline Mill Pond Peak Flow Reduction: Existing vs. Proposed Conditions 26
Table 5-1 Existing Drainage Areas - Area C 31
Table 5-2 Sanitary Servicing Criteria - Area C 34
Table 5-3 Sanitary Servicing Summary - Area C 34
Table 5-4 Water Servicing and Flow Demands 36
Table 5-5 Modelled Service Pressure and Fire Flows 36
Table 5-6 Stormwater Management Summary for Area C 40
Table 6-1 Existing Drainage Areas - Area D 46
Table 6-2 Sanitary Servicing Criteria - Area D 49
Table 6-3 Sanitary Servicing Summary - Area D 49
Table 6-4 Water Servicing and Flow Demands 50
Table 6-5 Modelled Service Pressure and Fire Flows 50
Table 6-6 Stormwater Management Summary for Area D 53
Table 7-1 Existing Drainage Areas - Area E 60
Table 7-2 Sanitary Servicing Criteria - Area E 62
Table 7-3 Sanitary Servicing Summary - Area E 62
Table 7-4 Water Servicing and Flow Demands 64
Table 7-5 Modelled Service Pressure and Fire Flows 64
Table 7-6 Stormwater Management Summary for Area E 67

APPENDICES

Appendix A – Geotechnical Report
Appendix B – Hydrogeology Report & Foundation Drain Analysis Memo
Appendix C – Tuck, Shoreacres, Appleby And Sheldon Creeks Watershed Study Letter
Appendix D – Sanitary Sewer As-Constructed Drawings
Appendix E – Sanitary Sewer Design Sheets
Appendix F – Water Assessment Report
Appendix G – STM Sewer Design Sheets & R.O.W. Calculations
Appendix H – SWM Calculations
Appendix I – Channel Re-alignment & Hydraulics
Appendix J – PTTW Application
Appendix K – Noise Study
Appendix L – Arborist Report
Appendix M – Area E Crane Review Letter

FIGURES

ROW-1– 10.3m Condo Road Cross-section (without transformer)
ROW-1-TX - 10.3m Condo Road Cross-section (with transformer)
ROW-2– 10.8m Condo Road Cross-section (without transformer)
ROW-2-TX - 10.8m Condo Road Cross-section (with transformer)

EX-AB – Existing Drainage Plan (Areas A & B)
PR-AB – Proposed Drainage Plan (Areas A & B)
SAN-AB – Sanitary Servicing Plan (Areas A & B)
WM-AB – Water Distribution Plan (Areas A & B)
STM-AB – Storm Drainage Plan (Areas A & B)
LID-A1 – Conceptual LID Plan (Area A)
LID-A2 – Conceptual LID Plan (Area A)
LID-B – Conceptual LID Plan (Area B)
CMMP-AB – Construction Management & Mobility Plan (Areas A & B)

EX-C – Existing Drainage Plan (Area C)
PR-C – Proposed Drainage Plan (Area C)
SAN-C – Sanitary Servicing Plan (Area C)
WM-C – Water Distribution Plan (Area C)
STM-C – Storm Drainage Plan (Area C)
LID-C – Conceptual LID Plan (Area C)
CMMP-C – Construction Management & Mobility Plan (Area C)

EX-D – Existing Drainage Plan (Area D)
PR-D – Proposed Drainage Plan (Area D)
SAN-D – Sanitary Servicing Plan (Area D)
WM-D – Water Distribution Plan (Area D)
STM-D – Storm Drainage Plan (Area D)
LID-D – Conceptual LID Plan (Area D)
CMMP-D – Construction Management & Mobility Plan (Area D)

EX-E – Existing Drainage Plan (Area E)
PR-E – Proposed Drainage Plan (Area E)
SAN-E – Sanitary Servicing Plan (Area E)
WM-E – Water Distribution Plan (Area E)
STM-E – Storm Drainage Plan (Area E)
LID-E – Conceptual LID Plan (Area E)
CMMP-E – Construction Management & Mobility Plan (Area E)
(as appended to Shoring Plans, Sections and Details as Drawings SHO-SH3 prepared by Terra Engineering & Structural Consulting Inc.)

Urbantech® Consulting, A Division of Leighton-Zec Ltd. | 3760 14th Avenue, Suite 301 • Markham • ON • L3R 3T7 | 905.946.9461
urbantech.com

DRAWINGS

GP-AB – General Plan (Areas A & B)
GR-AB – Grading Plan (Areas A & B)
FP-1 – Existing Floodplain (Areas A & B)
FP-2 – Proposed Floodplain (Areas A & B)

GP-C – General Plan (Area C)
GR-C– Grading Plan (Area C)

GP-D – General Plan (Area D)
GR-D – Grading Plan (Area D)

GP-E – General Plan (Area E)
GR-E – Grading Plan (Area E)

Urbantech® Consulting, A Division of Leighton-Zec Ltd. | 3760 14th Avenue, Suite 301 • Markham • ON • L3R 3T7 | 905.946.9461
urbantech.com

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 the five (5) development Areas under consideration (Areas A, B, C, D and E) hereafter referred to as the subject property.

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. Area E will require extension of a sanitary sewer along Dundas Street for connection to the existing Millcroft Park Drive sewer. The additional discharge flow from the developments will not have a negative net impact on the existing system.

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. Area E will require extension of a watermain along Dundas Street for connection to the existing Millcroft Park Drive watermain. The results from the hydrant flow tests reveal the existing water infrastructure can support the proposed developments.

1.3. STORM SERVICING AND STORMWATER MANAGEMENT

The storm sewer network has been sized in compliance with the City's guidelines. Areas discharging to existing storm sewers (Areas B, C and D) will provide stormwater management or flow diversion measures to ensure there is no increase / no impact to existing downstream sewers. Note, Areas C & D are both designed with quantity controls to target pre-development flow rates whereas discharge from Area B is diverted to avoid downstream existing residential impacts. Areas discharging directly to the adjacent watercourses or ponds (Areas A, E, and a portion of D) do not require stormwater management in accordance with the approved subdivision design. However, the reconfigured & offline Mill Pond's (to be taken offline as part of the PTTW process) contributing area discharge, inclusive of Areas A & B as well as existing residential areas, will be quantity controlled and effectively reduced to below pre-development flows rates directed to the existing online Mill Pond. As such, Areas A through D's post development flows reflect reductions to the pre-development peak flows since Areas C & D's minor system capacity is overcontrolled and Areas A & B's discharge is attenuated via the reconfigured Mill Pond's active storage component. Further, Area E and a portion of Area D are both proposed to discharge uncontrolled without quantity control to the respective receiving tributary. However, Area D is effectively quantity controlled based on the 100-year overcontrol offered by the super-pipe storage capacity and orifice control flows. Lastly, the proposed storm sewers internal to all Areas have been sized accordingly.

1.4. GRADING

The proposed grading meets the City's requirements. Grades will be maintained along the property line and a 6.0m buffer will be provided between existing and future lots. The grading design has avoided the use of retaining walls and minimizes earthworks where possible.

2 INTRODUCTION

2.1. BACKGROUND

This report supports various planning applications for the proposed residential development Areas (A-D) located at 2155 Country Club Drive & residential development Area E located at 4274 Dundas Street, 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 Zoning By-Law Amendment (ZBA) submission requirements and addresses City of Burlington preliminary 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)
- Hydrogeological Assessment – R.J Burnside & Associates Ltd. (RJB, November 2020)
- Foundation Drain Assessment - R.J Burnside & Associates Ltd. (RJB, December 2020)
- Water Distribution Analysis – Municipal Engineering Solutions (MES, March 2020)
- Noise and Vibration Feasibility Study – Howe Gastmeier Chapnik Limited (HGC, November 2020)

Note, the Environmental Evaluation Report (EER) by Beacon Environmental (Beacon, November 2020) are provided under separate cover as part of the ZBA 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 site plan, 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 (February 2016)
- 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 4 distinct study areas for the overall FSR and total approximately 12.36 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-53** 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	9.49 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	0.75 ha	Hole 16	East side of Millcroft Park Drive, south of Latimer Crescent and north of Turnberry Road
Area E	0.66 ha	Maintenance Yard	South side of Dundas Street West between Westlock Common and Millcroft Park Drive

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 CRITRIA

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. Except for Area E, the proposed development will consist of Single-Family 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). This value has been conservatively applied to sanitary flow estimates.

- 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

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

Storm sewers are to be designed using the criteria outlined 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 3-4** below:

Table 3-4 IDF Values for City of Burlington

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 Concentration (min) =	10	10	10	10	10	10
I =	Intensity (mm/hr)	78.39	101.31	116.31	135.38	149.30	163.18

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.*"

Taking this into account, the Tuck, Shoreacres, Appleby and Sheldon Creeks Watershed Study (SWS) prepared by the City of Burlington in June 1985 takes precedence over the City of Burlington's and MECP's design guidelines and standard practices. The approved watershed study guiding the subdivision design recommends that quantity and erosion controls are not required. At the time this study was completed quality control was not a requirement. The attached August 1995 stormwater management letter in **Appendix C** describes the stormwater management requirements.

In accordance with the SWS, no runoff quantity controls, erosion controls or quality controls are required for the watercourses adjacent to or downstream of the proposed development Areas. However, per pre-consultation meetings with the City of Burlington, this FSR report outlines a water quality benefit strategy for the proposed development Areas.

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.

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, 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 neighbouring properties.*

Urbantech® Consulting, A Division of Leighton-Zec Ltd. | 3760 14th Avenue, Suite 301 · Markham · ON · L3R 3T7 | 905.946.9461
urbantech.com

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: 9.49 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 south west 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 north east to the south west 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 1.1m (BH18-2) to 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'

*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 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 since December 2018 through to June 2020 and will continue to be monitored a quarterly frequency thereafter.

A review of the groundwater level data suggests the following:

- Groundwater levels in monitoring wells were observed to range between a high of 0.7 mbgs to dry conditions (i.e. 4.7 mbgs)
- 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).*

Foundation drain collectors, for the proposed detached lots within Areas A & B, will address address seasonal high groundwater elevations with respect to the interface along the detached lots underside of footing elevation and outlet to the proposed minor storm sewer system as detailed in the Foundation Drain Analysis (FDA) by RJB provided in **Appendix B**. 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 in the FDA memorandum prepared by RJB. For discussion on the estimated groundwater flow impacts to the storm sewer system, refer to **Section 4.5.1**.

*Refer to **Appendix B** for the Foundation Drain Analysis completed by RJB (December 2020).*

4.1.5. EXISTING MINOR SYSTEM DRAINAGE

There is an existing 675mm storm sewer that traverses an easement from Parklane Crescent through eastern limit of Area B to a 750mm storms 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. 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. EXISITING MAJOR SYSTEM DRAINAGE – AREA A

The site consists of two separate Areas (Area A and Area B), which are divided by an existing development (Hadfield Court). The site generally drains north to south, ultimately making its way to the existing pond at the south west corner of Area A.

Existing drainage from Area A generally drains from the north east to south west with an average slope of approximately 1.0%. Area A has been divided into five (5) existing drainage subcatchments and associated with five (5) external existing drainage subcatchments 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 subcatchments 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 being 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 Mill Pond is also used by the existing golf course for irrigation. The current irrigation distribution network irrigates all greens, tee, fairways, and designated portions of rough. 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 Renewal Application completed by RJB, May 2016.*

4.1.8. EXISITING MAJOR SYSTEM DRAINAGE – AREA B

The existing drainage from Area B generally drains from the north east to south west 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 being 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

As shown on **Figure SK-53A&B**, Area A will be comprised of developable area and non-developable area. The developable area will consist of 33 residential lots with a 10.3 m right-of-way width and 8.5 m pavement width as measured to face of curbs. Residential lot depths average approximately 28.0 m and range from approximately 26.85 m to 32.76 m along with an average width of approximately 15.24 m. The non-developable area consists of the existing Appleby Creek, the proposed low flow channel re-instatement that is required as part of the Ministry of Environment and Conservation (MECP) Permit to Take Water (PTTW) process, existing online pond (to be taken offline as part of the PTTW process) and CN Rail Berm (located along the east side of the site, parallel to the railway).

As shown on **Figure SK-53A&B,** Area B will be comprised of developable area and non-developable area. The developable area will consist of 42 residential lots with a 10.3 m right-of-way width and 8.5 m pavement width as measured to face of curb. Residential lot depths average approximately 34.7 m and range from approximately 26.96 m to 34.0 m with an average width of approximately 15.24 m. The non-developable area consists of the CN Rail Berm (located along the east side of the site, parallel to the railway).

Note, a 6.0 m treed buffer is proposed between the proposed residential development and the existing residential development along Areas A and B. This buffer will act as a privacy feature between existing and proposed residential lots interface 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-1 by Beacon included in **Appendix L**. Note, a tree preservation plan is included in **Drawing GR-AB** with respect to City owned street trees only. For details on ownership and condition ratings, refer to Arborist Report Figure 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
- Application of the City of Burlington standards

The grading plan, **Drawing GR-AB**, 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 north east to south west and that proposed grades minimize impacts to existing trees within the 6m buffer along the existing properties. Note, trees for preservation and associated protection fencing by Beacon is included on **Drawing GR-AB** as per Drawing TP-1 included in **Appendix L**.

*Refer to **Drawing GR-AB** for the grading plan for Area A and Area B.*

4.2.3. RIGHT-OF-WAYS

As shown on Figure SK-53A&B (produced by Gerrard Design for the Areas A & B Concept Plan as included under separate cover), the proposed Area A and Area B developments consist of private condominium/site plan blocks, including:

- Low density residential blocks
- Park blocks / Open space
- Private ROWs

The proposed development will be advanced through the condominium approval process. The low-density residential site plan block will consist of the private ROW areas, open space blocks, and services. Preliminary right-of-way cross-sections have been prepared as **Figure ROW-1** and **Figure ROW-1-TX**. These cross-sections have been developed to support the proposed surface treatment of the various right-of-ways. The cross-sections will be further refined in consultation with the required approval agencies and utility companies, and in keeping with the developing master plan vision. The cross-sections are conceptual and are to be discussed further with the City of Burlington.

*Refer to Figure SK-53A&B (produced by Gerrard Design) for the Areas A and B Concept Plan. Refer to **Figure ROW-1** & **Figure ROW-1-TX** (in consultation with RTG Systems Inc.) for preliminary right-of-way cross-sections / renderings for Areas A and B.*

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 (#)	33 (Area A)
Peaking Factor (M)	Harmon Formula		42 (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) to accommodate a right-angle connection of the proposed sewer. 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** below 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 Equivalent Density (ppu)	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
A	2.26	33	3.52	0.37	0.65	4.22	2.22	EX.MH 23
B	3.26	42	3.52	0.47	0.93	4.19	2.91	MH. 5A

Based on the above criteria and the InfoSewer project report, it has been determined that there is ample capacity to service the subject lands.

*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.*

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 Hydraulic Analysis 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 Hydraulic Analysis Report (MES, March 2020) in **Appendix F** and **Figure WM-AB** 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 sewers will be designed to convey minor system flows resulting from the 5-year storm event for ultimate discharge to the reconfigured offline pond (to be taken offline as part of the PTTW process) adjacent 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 **Figure STM-AB**. Note, the groundwater flows, as intercepted by the foundation drain collectors, outlets to the minor storm system which negligibly impacts the percent full capacity as demonstrated in the storm design sheets included in **Appendix G**.

*Refer to **Figure STM-AB** for the Storm Drainage Plan and **Appendix G** for the Storm Sewer Design Sheets for Area A and Area B.*

4.5.2. PROPOSED MAJOR SYSTEM

An overland flow route has been designed to convey major system flows from Area A to the reconfigured offline pond (to be taken offline as part of the PTTW process) as shown on **Figure STM-AB** and **Drawing GR-AB**.

All major system flows from Area B will be captured prior to discharge into the reconfigured offline pond as shown on **Figure STM-AB** and **Drawing GR-AB**.

The proposed ROWs within the subject lands have been evaluated and will provide conveyance capacity for the major system flows (evaluated as the 100-year less 5-year storm flows).

*Refer to **Appendix G** for the Overland Flow Route & ROW Calculations for Areas A and B.*

4.5.3. CHANNEL REALIGNMENT / POND RETROFIT

Under existing conditions, the golf club utilizes the existing online pond for all their irrigation requirements. RJB was engaged by Millcroft Golf Club to renew the PTTW application with the MECP. Through that process, the MECP required the golf course to take the Mill Pond offline and remove it from the creek to protect baseflow for aquatic life within Appleby Creek. RJB has initiated a Permit process with CH, pursuant to Ontario Regulation 162/06 to enable this removal of the online pond and re-instatement of a low flow channel for Appleby Creek through this area.

The development of Areas A and B is not contingent on taking the pond offline however, taking the pond offline achieves an overall water quality benefit for Appleby Creek.

Further, the scenario proposed to satisfy the PTTW process presents a unique opportunity to achieve the following:

- Protect and enhance the baseflow and aquatic resources of Appleby Creek; and
- Provide for the required meander belt established in the EER by Beacon (15.0 m); and
- Optimize Pond Functions to include water quality benefits and reduce peak flows from the proposed contributing drainage area; and
- Use stormwater for irrigation

Continued coordination with RJB and CH will be necessary to finalize the proposed Mill Pond in terms of the PTTW and irrigation requirements.

Specific to Area A, modifications to the Regional Storm flood plain are proposed to create additional development area. Conservation Halton's Policies and Guidelines for the Administration of Ontario Regulation 162/06 and Land Use Planning Policy Document (revised 2015) anticipates alterations to flood plains to create building lots. Specifically, Policy 3.19.1 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 3.19.1 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 fulfil the requirements as outlined above in the definition of 'comprehensive study'. However, CH staff have advised that, in addition to the requirements as outlined in the definition of 'comprehensive study' they will also require demonstration that the proposed alteration is not only to facilitate more developable land but that it also provides additional upstream/downstream benefits and that it is in the public interest. While the Study Team does not agree that such a demonstration is required, based on the contents of Policy 3.19.1 or the definition of 'comprehensive study', in order to advance discussions with CH as it relates to a future Permit application pursuant to O. Regulation 162/06, a separate summary report will be submitted, upon receipt

of comments on the FSR, EER and hydrogeology report, that summarizes the anticipated benefits as a result of the proposed development.

*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 information for the reach to create an existing condition floodplain as indicated in **Drawing FP-1**. The model was updated to reflect the proposed removal of the online pond and the reconfiguration of the channel and installation of a future pedestrian bridge located downstream of crossing section 287.5 and upstream of cross-section 287. 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.

Table 4-8 Existing and Proposed Flood Elevations

Section Numbers	Regional Storm				
	CH Model	Urbantech Existing Condition	Urbantech Proposed Condition	Difference [m]	Note
	W.L. [m]	W.L. [m]	W.L. [m]		
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	140.51	-0.18	
296	140.2	140.58	140.32	-0.26	
295.5	Millcroft Park Drive				
295	139.9	140.15	139.47		
294	139.93	140.2	139.39		
293	139.79	139.84	139.19		
292	139.42	139.6	138.93		
291.5	-	-	138.93		Add Section
291	139.22	139.15	138.88		
290	138.96	138.92	138.52		
289	138.89	138.8	138.33		
288	138.38	138.28	138.19		
287.7	-	-	138.14		Add Section
287.5	-		138.12	-	Add Section
287.3	-Proposed Pedestrian Bridge-				New Bridge
287	138.21	138.1	138.1	0	
286	138.23	138.11	138.1	-0.01	
285	138.24	138.12	138.09	-0.03	
284	138.24	138.11	138.08	-0.03	
283	138.21	138.08	138.02	-0.06	
282	138.22	137.97	137.96	-0.01	

Section Numbers	Regional Storm				
	CH Model	Urbantech Existing Condition	Urbantech Proposed Condition	Difference [m]	Note
	W.L. [m]	W.L. [m]	W.L. [m]		
281	137.99	137.89	137.84	-0.05	
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	

As can be seen from the results in the above table and **Drawings FP-1 / FP-2**, the reconfiguration of the pond and channel under the proposed scenario do not result in an increase in flood lines on adjacent/existing lots along the corridor. There is a considerable decrease in the regional water levels on the existing lots along the subject lands as well as no increase in Appleby Creek water levels downstream.

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 flood storage for the corridor will be impacted by the proposed development plan. The 2 year through Regional Storm was assessed and the results are summarized in **Table 4-9** below.

Table 4-9 Summary of Riparian Storage Volumes

	Pre-development conditions (Riparian)		Proposed without pond* (Riparian)		Proposed with offline pond (Riparian)	
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	148.05	26.01	156.18	34.10	169.07	47.03
100-year	85.66	16.12	89.14	19.91	101.79	32.54
50-year	78.03	14.68	80.84	17.76	93.45	30.35
25-year	70.01	12.94	71.98	15.18	84.35	27.53
10-year	57.41	10.23	58.38	11.47	70.10	23.17
5-year	49.74	8.38	50.21	9.12	60.57	19.46
2-year	35.08	4.85	36.34	6.32	38.95	8.88

*this scenario has been provided to demonstrate that the proposed channel realignment / cut-fill does not rely on the offline 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, or for the larger reach extending upstream and downstream of the subject lands. There is a general increase in floodplain storage resulting from the proposed grading which provides an overall net community benefit. An

incremental cut/fill assessment has been completed for each cross-section and is included in **Appendix I**.

*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

As noted previously, the existing Millcroft Community was developed in the mid-1980s and pre-dates the MECP SWM Guidelines (2003). As such, the existing pond does not have any SWM controls except for a small amount of storage/pond volume for quality treatment. The existing pond is online, and the water level is controlled by the elevation of the downstream weir (~136.30m). The pond provides all the irrigation requirements for the existing golf course. Quantity and extended detention (erosion) controls are not being provided by the existing development.

Through the redevelopment of the subject property, there is a unique opportunity to improve the health of Appleby Creek by removing the pond from the stream as well as reconfiguring the pond to provide SWM and irrigation functions within a reconfigured offline facility. Given the multiple uses of the pond, a standard approach to achieving SWM criteria is not practical and will require innovation and flexibility to achieve a net benefit to the receiving creek system. Further, water quality benefits will be provided as part of LID treatment train approach as discussed in **Section 4.6.2** and partial quantity control of post-development flows is offered by the reconfigured offline pond's active storage component as discussed in **Section 4.6.3.**

Refer to **Figure PR-AB** for the stormwater management plan for Areas A and B.

4.6.2. QUALITY CONTROL

As stated in the guiding watershed study, quality control for the subject lands is not required but will be implemented wherever feasible. For instance, the proposed offline & reconfigured Mill Pond downstream of the site will provide an Enhanced Level of water quality protection to ensure removal of 80% of total suspended solids for the proposed development areas inclusive of proposed residential Areas A & B, as well as existing residential on Hadfield Court & Parklane Crescent. The reconfigured and offline Mill Pond provides approximately 9274 m^3 of permanent pool storage between elevations 134.3 m and 136.3 m as shown in the following **Table 4-10**.

Table 4-10 Proposed Reconfigured Offline Pond Characteristics

Operating Level	Elevation (m)	Area (m^2)	Cumulative Volume (m^3)
Pond Bottom	134.3	3721	0
	135.3	4615	4168
Permanent Pool	136.3	5597	9274
Top of Berm	136.8	6083	12194

For the proposed contributing drainage area consisting of both proposed and existing drainage areas as abovementioned inclusive of the pond block (total 18.18 ha at 41% imperviousness), a permeant pool volume of 2109 m^3 is required. Therefore, 80% TSS removal for Areas A and B as well as the existing development can be achieved. This is an improvement over the current situation in which all existing flows discharge directly into the online Mill Pond on Appleby Creek with no quality controls.

Urbantech® Consulting has incorporated input into the design of taking the Mill Pond offline as part of the PTTW process. This includes taking the following recommendations into consideration:

- Providing a meander belt width of 15 m from the re-instated channel; and
- Provide stormwater quality treatment for the existing & proposed upstream drainage area; and
- Use of stormwater for irrigation

As mentioned in **Section 4.5.3**, ongoing coordination with RJB will be necessary to finalize the proposed Mill Pond in terms of PTTW and irrigation requirements.

In addition to providing quality control with end of pipe controls, lot level low-impact development (LID) controls are proposed within Areas A and B to support water quality benefits by a treatment train approach as illustrated in **Figures LID-A1, LID-A2 & LID-B**.

4.6.3. QUANTITY CONTROL

As stated in Section 3.4, quantity control criteria was not required for the historic/existing development and is not required under the current master drainage plan. The general approach applied to new development involves control of the post-development flow rates to pre-development levels for Area B to ensure no impact to the existing development downstream. No quantity control is required for Area A. However, quantity control is provided by the active storage component of the reconfigured offline Mill Pond by reducing post-development peak flows below existing peak flow rates.

New development Areas will consist of split drainage on the lots. Rear-yard drainage will be intercepted in the rear-yard swales and rear lot catchbasins. Roof tops and front lot drainage will be conveyed into the storm sewer system. Post-development flows (5-year) from Area B will be directed to Hadfield Court via the existing 750mm storm sewer. These flows are lower than the current combined total of the existing / external flows approaching Hadfield Court. The external flows from north of Area B will be diverted to the east, in a proposed sewer along the east edge of the subject lands (between the existing development and rail corridor) while the existing 750 mm sewer is removed from Parklane Crescent to Hadfield Court. This strategy ensures that flows discharged to the existing Hadfield Road sewers are lower than existing conditions.

The remaining flows (external minor and major flows and major system flows from Area B) will be conveyed towards Area A via the proposed storm sewer and overland drainage path along the east side of the subject lands.

A summary of the flows for Area A and Area B can be found in **Table 4-11** below:

Table 4-11 SWM Summary for Area A & Area B

	Location	Flow Rates (m^3/s)					
		2-year	5-year	10-year	25-year	50-year	100-year
Existing	Outlet 1A	0.629	0.813	0.934	1.196	1.438	1.638
Proposed (uncontrolled)	Outlet 1A	1.023	1.322	1.518	1.943	2.338	2.662
Existing	Outlet 1B	0.616	0.796	0.913	1.170	1.407	1.602
Proposed (Uncontrolled)	Outlet 1B	0.533	0.689	0.792	1.013	1.219	1.388
Existing	Outlet 2B	0.210	0.272	0.312	0.400	0.481	0.548
Proposed (Uncontrolled)	Outlet 2B	0.065	0.084	0.097	0.124	0.149	0.170

Based on the reconfigured offline Mill Pond design, approximately 3000 m^3 of active storage is available above the permanent pool elevation of 136.3 m to 136.8 m which partially attenuates flows from the reconfigured offline Mill Pond's proposed contributing drainage area as listed in **Table 4-12** and calculated in **Appendix H**.

Table 4-12 Offline Mill Pond Peak Flow Reduction: Existing vs. Proposed Conditions

		Flow Rates (m^3/s)					
		2-year	5-year	10-year	25-year	50-year	100-year
Existing Mill Pond	Inflow*	1.398	2.352	2.861	3.580	4.122	4.624
Proposed Mill Pond Active Storage	Inflow	1.533	2.306	2.789	3.466	4.433	4.950
	Outflow	1.098	1.627	1.965	2.437	2.889	3.229
Peak Flow Reduction (%) [1-Proposed Inflow/ Prop. Outflow]		28.3	29.4	29.5	29.7	34.8	34.8
Peak Flow Reduction (%) [1-Existing Inflow/ Prop. Outflow]		21.5	30.8	31.3	31.9	29.9	30.2

*In cases where irrigation demands do not exist, inflow to equal outflow as existing Mill Pond configuration does not include active storage

As seen in **Table 4-12** per City of Burlington Stormwater Management Design Guidelines, the above analysis is based on a 24-hour SCS Type II design storm which was determined to be the most appropriate following a sensitivity analysis of all acceptable rainfall distributions as modelled in **Appendix H**. The active storage component of the reconfigured offline Mill Pond provides an average 31% peak flow reduction for 2-year to 100-year storm events from post-development flows using the 1st stage of a two-stage weir control outlet structure. The second stage of the outlet structure is an emergency spillway which will be designed at the detailed design stage to safely convey the 100-year flow. Note, the active storage component of the reconfigured and offline Mill Pond provides an average peak flow reduction of 29% as compared to pre-development flows directed to the existing online Mill Pond.

The proposed SWM drainage areas are included on **Figure PR-AB** and the storm servicing strategy is shown on **Figure STM-AB**. The proposed layout of the reconfigured Mill Pond is depicted on **Drawing GR-AB**.

*Refer to **Figure PR-AB** for the stormwater management plan for Area A and B. Refer to **Appendix H** for the SWM calculations. Refer to **Figure STM-AB** for the storm servicing strategy and **Drawing GR-AB** for the reconfigured Mill Pond layout.*

4.6.4. WATER BALANCE

A water balance was prepared by RJB in conjunction with their hydrogeology report. A copy of the report is 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 37% of the average annual groundwater infiltration could occur (refer to RJB Report Table G-3, Appendix G in **Appendix B**).

As such, in order to mitigate any groundwater impacts, the proposed SWM strategy is recommending directing roof areas to rear-yards (disconnection of roof leaders) and an increased topsoil depth of 300 mm. This will result in a potential increase of 1% of the average annual groundwater infiltration.

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

4.7. IMPLEMENTATION

Area A will be developed first in conjunction with the reconfiguration of the creek and removal of the online pond to accommodate the conveyance of storm drainage from the existing and proposed development areas draining to the Appleby Creek.

Area B relies on the completion of services and grading for Area A; hence it will not be developed until Area A is complete.

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

4.8. 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 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 back of select lots closest to the railway & Upper Middle Road as illustrated on Figure 3A in **Appendix K**. The associated grading design impacts will be reviewed at the detailed design stage.

*Refer to **Appendix K** for the Noise and Vibration Feasibility Study completed by HGC (November 2020).*

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 designed to convey the 5-year flow to the proposed reconfigured offline Mill Pond east of Appleby Creek.
- 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.
- Quality control will be provided through the reconfigured pond and lot level LIDs as part of a treatment train approach
- Stormwater quantity control is not required for Area A. However, flows from Area B will be controlled to the pre-development level via a flow splitter. The major system flows from Area B will be captured in sewers and directed to the outlet pipe and pond feature. The proposed reconfigured offline Mill Pond's active storage component will reduce peak flows from the pond's contributing drainage area below existing peak flows.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.
- The development of the site is feasible from a noise and vibration perspective while feasible means exist to reduce impacts to within acceptable limits.

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.

Topographical surveys of the subject lands have been completed by R-PE in September 2019.

The site generally falls from north east to the south west 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

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 – 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'

*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 installation of three monitoring wells across the subject property in November 2019. Groundwater levels across Area C have been monitored monthly from December 2019 through to June 2020 and will continue to be monitored on a quarterly frequency thereafter.

A review of the groundwater level data suggests the following:

- Groundwater levels in monitoring wells were observed to range between a high of 0.2 mbgs in response to precipitation events to a low of 2.4 mbgs.
- 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).*

Foundation drain collectors, for the proposed detached lots within Area C, will address address seasonal high groundwater elevations with respect to the interface along the detached lots underside of footing elevation and outlet to the proposed minor storm sewer system as detailed in the Foundation Drain

Analysis (FDA) by RJB provided in **Appendix B**. 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 in the FDA memorandum prepared by RJB. 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 Drain Analysis completed by RJB (December 2020).*

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**.

5.1.6. EXISTING MAJOR SYSTEM DRAINAGE

The majority of Area C generally drains from the interior of the site to CB's 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	0.60	0.30	E01-C	2.70	0.30
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	1.70	0.30	Total	3.73	0.33

- 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 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 0.33.

*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

Area C will be comprised of developable area and non-developable area. The developable area will consist of 16 residential lots with a 10.8 m right-of-way width and 8.5 m pavement width as measured from face of curbs. Residential lot depths average approximately 34.58 m and range from approximately 24.67 m to 36.81 m. Lot widths range from approximately 10.68 to 34.5 m. The non-developable area will consist of park / green space / landscaped areas.

Note, a 6.0 m treed buffer has been proposed between the proposed and existing residential areas. This buffer will act as a privacy feature between existing and proposed residential lots interface 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 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 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 road and site plan block 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 north east to south west and that proposed grades minimize impacts to existing trees within the 6.0 m buffer along 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**.

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

5.2.3. RIGHT-OF-WAYS

As shown on Figure SK-53C&D (produced by Gerrard Design for the Area C Concept Plan as included under separate cover), the proposed 1.46 ha development consists of a site plan block, including:

- Low density residential blocks
- Park blocks / Open space
- Private ROWs

The proposed development will be advanced through the condominium approval process. The low-density residential site plan block will consist of the private ROW areas, open space blocks, and services. Preliminary right-of-way cross-sections have been prepared as **Figure ROW-2** and **Figure ROW-2-TX**. These cross-sections have been developed to support the proposed surface treatment of the various right-of-ways. The cross-sections will be further refined in consultation with the required approval agencies and utility companies, and in keeping with the developing master plan vision. The cross-sections are conceptual and are to be discussed further with the City of Burlington.

*Refer to Figure SK-53C&D (produced by Gerrard Design) for the Area C Concept Plan as included under separate cover. Refer to **Figure ROW-2** & **Figure ROW-2-TX** (in consultation with RTG Systems Inc.) for preliminary right-of-way cross-sections for Area C.*

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 (#)	16
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)	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	16	3.52	0.16	0.42	4.31	1.13	EX.MH 10

Based on the above criteria and the InfoSewer project report, it has been determined that there is ample capacity to service the subject lands.

*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.*

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 Hydraulic Analysis 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 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 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 Hydraulic Analysis 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 some of the major system flows as shown on **Figure STM-C**. Note, the groundwater flows, as intercepted by the foundation drain collectors, outlets to the minor storm system which negligibly impacts the percent full capacity as demonstrated in the storm design sheets included in **Appendix G**.

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

5.5.2. PROPOSED MAJOR SYSTEM

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

The proposed ROWs within the subject lands have been evaluated and will provide conveyance capacity for the major system flows (evaluated as the 100-year less 5-year storm flows) and catchbasins (CBs) have been sized to capture the 100-year storm event.

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

5.6. STORMWATER MANAGEMENT

5.6.1. DESIGN

As noted previously, the existing Millcroft Community was developed in the mid-1980s and pre-dates the MECP SWM Guidelines (2003). As such, quantity, erosion, and quality controls are not required. However, water quality benefits will be provided as part of LID treatment train approach as discussed in **Section 5.6.2** and quantity control of post-development flows overcontrolled to pre-development flows to be met at existing storm sewer connections as discussed in **Section 5.6.3.**

*Refer to **Figure PR-C** for the stormwater management plan for Area C.*

5.6.2. QUALITY CONTROL

The proposed development will mimic the existing development and will drain into the existing storm sewer system, ultimately discharging to Appleby Creek. Minor flows will be captured, downstream of the orifice control detailed in **Section 5.6.3**, in an oil & grit separator (OG/S) prior to discharging into the existing sewers on Country Club Drive. Note, CB shields will be provided where necessary to provide a water quality treatment train approach in conjunction with the OG/S. In addition, lot level low-impact development (LID) controls are proposed within Area C to support water quality benefits by a treatment train approach as illustrated in **Figure LID-C**.

5.6.3. QUANTITY CONTROL

As stated in **Section 3.4**, quantity control criteria was not required for the historic/existing development and is not be required under the current master drainage plan. 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). Roof tops 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. A 110 mm diameter orifice tube is required to meet the pre-development flows. This produces a required storage of 195 m^3. A super pipe was designed, 1200 mm diameter and 172.00 m in length, to meet the required storage target. This super pipe has a provided storage volume of 195 m^3 and will therefore satisfy the design. Stormwater management calculations can be found in **Appendix H**.

A summary of the flows for Area C can be found in **Table 5-6**.

Table 5-6 Stormwater Management Summary for Area C

	Location	Flow Rates (m^3/s)					
		2-year	5-year	10-year	25-year	50-year	100-year
Existing	Outlet 1C	0.252	0.325	0.374	0.478	0.575	0.655
Proposed (uncontrolled)	Outlet 1C	0.232	0.300	0.345	0.441	0.531	0.604
Existing	Outlet 2C	0.052	0.068	0.078	0.099	0.119	0.136
Proposed (Uncontrolled)	Outlet 2C	0.170	0.220	0.252	0.323	0.388	0.442
Proposed (Controlled)	Outlet 2C	0.075	0.084	0.090	0.103	0.115	0.125
Existing	Outlet 3C	0.033	0.043	0.049	0.063	0.076	0.087
Proposed (Uncontrolled)	Outlet 3C	0.032	0.042	0.048	0.061	0.073	0.084
Existing	Outlet 4C	0.038	0.049	0.057	0.073	0.087	0.099
Proposed (uncontrolled)	Outlet 4C	0.044	0.056	0.063	0.081	0.097	0.111

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

Calculations for the storage and orifice can be found in **Appendix H**. The proposed SWM drainage areas are included on **Figure PR-C** and the storm servicing strategy is shown on **Figure STM-C**.

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

5.6.4. WATER BALANCE

A water balance was prepared by RJB in conjunction with their hydrogeology report. A copy of the report is 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 37% of the average annual groundwater infiltration could occur (refer to RJB Report Table G-3, Appendix G in **Appendix B**). As such, the proposed SWM strategy is recommending directing roof top to rear-yards (disconnection of roof leaders) and an increased topsoil depth of 300 mm to mitigate any groundwater impacts. This will result in a potential increase of 1% of the average annual groundwater infiltration.

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

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

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 C, is feasible.

*Refer to **Appendix K** for the Noise and Vibration Feasibility Study completed by HGC (November 2020).*

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 designed to convey the 5-year flow.
- 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 is not required for Area C. However, flows will be controlled to the pre-development level via a 189 m^3 super pipe and a 100 mm diameter orifice tube. Major system flows from Area C will be captured in sewers and directed to the existing storm network.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.
- The development of the site is feasible from a noise & vibration perspective.

6 AREA D

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

AREA: 0.75 ha

CURRENT LAND USE:
Hole No. 16



6.1. EXISTING CONDITIONS

6.1.1. LAND USE & TOPOGRAPHY

The site is currently being 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.

Topographical surveys of the subject lands have been completed by R-PE in September 2019.

The site generally falls from north east to the south west 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

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 – 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'

*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 installation of three monitoring wells across the subject property in November 2019. Groundwater levels across Area D have been measured monthly since December 2018 through to June 2020 and will continue to be monitored a quarterly frequency thereafter.

- Groundwater levels in monitoring wells were observed to range between a high of 0.1 mbgs to a low of 1.9 mbgs.
- 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).*

Foundation drain collectors, for the proposed detached lots within Area D, will address address seasonal high groundwater elevations with respect to the interface along the detached lots underside of footing elevation and outlet to the proposed minor storm sewer system as detailed in the Foundation Drain Analysis (FDA) by RJB provided in **Appendix B**. 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 in the FDA memorandum prepared by RJB. 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 Drain Analysis completed by RJB (December 2020).*

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 subcatchments that are conveyed through Area D.

- Catchment E01-D consists of 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 being 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

Area D will be comprised of developable area and non-developable area. The developable area will consist of 7 residential lots with a 10.3 m right-of-way width and 8.5 m pavement width as measured from face of curbs. Residential lot depths range from approximately 27.86 m to 40.12 m with an average width of approximately 15.24 m. The non-developable area will consist of park / green space / landscaped areas.

Note, a 6.0 m treed buffer has been proposed between the proposed and existing residential areas. This buffer will act as a privacy feature between existing and proposed residential lots interface 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 road and site plan block 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 north east to south west and that proposed grades minimize impacts to existing trees within the 6.0 m buffer along the 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**.

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

6.2.3. RIGHT-OF-WAYS

As shown on Figure SK-53C&D (produced by Gerrard Design for the Area D Concept Plan as included under separate cover), the proposed 0.75 ha development consists of a site plan block, including:

- Low density residential blocks
- Park blocks / Open space / Golf Course
- Private ROWs

The proposed development will be advanced through the condominium approval process. The low-density residential site plan block will consist of the private ROW areas, open space blocks, and services. Preliminary right-of-way cross-sections have been prepared as **Figure ROW-1** and **Figure ROW-1-TX**. These cross-sections have been developed to support the proposed surface treatment of the various right-of-ways. The cross-sections will be further refined in consultation with the required approval agencies and utility companies, and in keeping with the developing master plan vision. The cross-sections are conceptual and are to be discussed further with the City of Burlington.

Refer to Figure SK-53C&D (produced by Gerrard Design) for the Area D Concept Plan as included under separate cover. Refer to ***Figure ROW-1*** *&* ***Figure ROW-1-TX*** *(in consultation with RTG Systems Inc.) for preliminary right-of-way cross-sections 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)	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	0.75	7	3.52	0.08	0.21	4.37	0.56	EX.MH 36

Based on the above criteria and the InfoSewer project report, it has been determined that there is ample capacity to service the subject lands.

*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.*

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 Hydraulic Analysis 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 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 C5-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 **Figure STM-D**. Note, the groundwater flows, as intercepted by the foundation drain collectors, outlets to the minor storm system which negligibly impacts the percent full capacity as demonstrated in the storm design sheets included in **Appendix G**.

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

6.5.2. PROPOSED MAJOR SYSTEM

The major system flows from Area D will be split into two different discharge areas. One area will be captured prior to discharge into the existing storm sewer system within the Millcroft Park Drive ROW. The second area will discharge north to the existing online pond on West Sheldon Creek via a private road ROW within Area D and an overland flow swale external to Area D. The major system flows are shown on **Figure STM-D**.

The proposed ROWs within the subject lands have been evaluated and will provide conveyance capacity for the major system flows (evaluated as the 100-year less 5-year storm flows) and catchbasins (CBs) have been sized to capture the 100-year storm event.

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

6.6. STORMWATER MANAGEMENT

6.6.1. DESIGN

As noted previously, the existing Millcroft Community was developed in the mid-1980s and pre-dates the MECP SWM Guidelines (2003). As such, quantity, erosion, and quality controls are not required. However, water quality benefits will be provided as part of LID treatment train approach as discussed in **Section 6.6.2** and quantity control of some post-development flows overcontrolled to pre-development flows to be met at existing storm sewer connections as discussed in **Section 6.6.3**.

*Refer to **Figure PR-D** for the stormwater management plan for Area D.*

6.6.2. QUALITY CONTROL

The proposed development will mimic the existing development and will drain into the existing storm sewer system, ultimately discharging to Appleby Creek. Minor flows will be captured, downstream of the orifice control detailed in **Section 6.6.3**, in an oil & grit separator (OG/S) prior to discharging into the existing sewers on Millcroft Park Drive. Note, CB shields will be provided where necessary to provide a water quality treatment train approach in conjunction with the OG/S. In addition, lot level low-impact development (LID) controls are proposed within Area D to support water quality benefits by a treatment train approach as illustrated in **Figure LID-D**.

6.6.3. QUANTITY CONTROL

As stated in **Section 3.4**, quantity control criteria was not required for the historic/existing development and is not be required under the current master drainage plan. The general approach applied to new development involves control of the post-development flow rates to pre-development levels.

Pre-development flows can be met at the existing storm sewer within Millcroft Park Drive through combination of diverting a portion of the flows to the pond north of the site, and controlling the remaining flows discharging to the storm sewer within Millcroft Park Drive. An 80 mm orifice tube was designed to meet the pre-development release rate. This produced a required storage of 58 m^3. A super pipe was designed, 825 mm diameter and 109.1 m in length to meet the storage target. This super pipe has a provided storage of 58 m^3 and will therefore satisfy the design. Stormwater management calculations can be found in **Appendix H**.

A summary of the flows for Area D can be found in **Table 6-6** below:

Table 6-6 Stormwater Management Summary for Area D

	Location	Flow Rates (m^3/s)					
		2-year	5-year	10-year	25-year	50-year	100-year
Existing	Outlet 1D	0.110	0.142	0.163	0.209	0.252	0.286
Proposed (Uncontrolled)	Outlet 1D	0.114	0.148	0.170	0.198	0.218	0.238
Proposed (Controlled)	Outlet 1D	0.091	0.107	0.117	0.120	0.117	0.118

Note, the proposed 100-year controlled release rate at STM Outlet 1D reflects an overcontrol of 0.120 m^3/s for the uncontrolled 100-year flow of 0.118 m^3/s at STM Outlet 2D to the online pond on West Sheldon Creek. As such, the super pipe provides the requisite storage to effectively quantity control for the entirety of Area D.

The proposed SWM drainage areas are included on **Figure PR-D** 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.*

6.6.4. WATER BALANCE

A water balance was prepared by RJB in conjunction with their hydrogeology report. A copy of the report is 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 37% of the average annual groundwater infiltration could occur (refer to RJB Report Table G-3, Appendix G in **Appendix B**). As such, the proposed SWM strategy is recommends directing roof top to rear-yards (disconnection of roof leaders) and an increased topsoil depth of 300 mm to mitigate any groundwater impacts. This will result in a potential increase of 1% of the average annual groundwater infiltration.

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

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 (November 2020).*

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 designed to convey the 5-year flow.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks in order to maximize reuse of soils within the property and avoid the need for retaining walls.
- Stormwater quantity control is not required for Area D. However, flows will be controlled to the pre-development level through combination of sending excess flows to the pond north of the site and by controlling the remaining flows via a 39 m^3 super pipe and a 110 mm diameter orifice tube.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.
- The development of the site is feasible from a noise & vibration perspective.

7 AREA E

LOCATION: South side of Dundas Street, west of Weslock Common / Tim Dobbie Drive and east of Millcroft Park Drive.

AREA: 0.66 ha

CURRENT LAND USE:
Club maintenance & operations grounds/facility



7.1. EXISTING CONDITIONS

7.1.1. LAND USE & TOPOGRPAHY

The site is currently being used as the maintenance and operations grounds / facility for the Millcroft Golf Club. The golf course maintenance building will be relocated to Country Club Drive, beside the Club House, to accommodate the proposed development. Currently, the site is covered with gravel parking area and a maintenance building.

Topographical surveys of the subject lands have been completed by Ltd. **R-PE** in September 2019.

The site generally falls from north-east to the south-west.

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

7.1.2. SOIL CONDITIONS

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 – 100 mm to 125 mm thick surficial layer of topsoil
- Fill material – ranging from 2.3 m to 4.6 m below grade consisting mainly of clayey silt, sandy silt till and silty clay.
- Upper soils – Clayey silt to sandy silt till
- Lower soils – Clayey silt till

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 0.6m (BH19-2) to 2.6m (BH19-1).

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.

7.1.3. 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'

*Refer to **Appendix A** for the Geotechnical Report for Area E as completed by DSCL (December 2020).*

7.1.4. 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 installation of three monitoring wells across the subject property in November 2019. Groundwater levels across Area E have been monitored monthly from December 2019 through to June 2020 and will continue to be monitored on a quarterly frequency thereafter.

A review of the groundwater level data suggests the following:

- Groundwater levels in monitoring wells were observed to range between a high of 0.4 mbgs to a low of 1.3 mbgs.
- 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 foundation drain collector will address seasonal high groundwater with respect to the interface along the underground parkade underside of slab elevation and outlet to be confirmed at the site plan design stage as detailed in the Foundation Drain Analysis (FDA) memorandum by RJB provided in **Appendix B**. Groundwater samples were collected and submitted for analysis to compare to the City's storm and sanitary sewer discharge criteria. The results of the groundwater analysis in comparison to the City's

storm sewer discharge criteria and discussion are presented in the FDA memorandum prepared by RJB. Additional chemical analysis based on low flow sampling techniques to mimic passive drainage conditions that may occur in foundation drainage systems was not available at the time of reporting to confirm any exceedances for sanitary and/or storm discharge criteria. For discussion on the estimated groundwater flow impacts to either the sanitary or storm sewer system, refer to **Section 7.3.2** and **Section 7.5.1** respectively.

*Refer to **Appendix B** for the Foundation Drain Analysis completed by RJB (December 2020).*

7.1.5. EXISTING MINOR SYSTEM DRAINAGE

There is currently no storm sewer infrastructure along Dundas Street. However, existing drainage outlets to a grass swale south of the site, ultimately discharging to Appleby Creek. There is an overland flow route immediately west of the site that conveys emergency flows from Dundas Street to a tributary of Appleby Creek. Refer to **Figure EX-E** for details.

7.1.6. EXISTING MAJOR SYSTEM DRAINAGE

Existing drainage from Area E generally drains from the north east to south west with an average slope of approximately 1.0%. Area E has been divided into three (3) existing drainage subcatchments and associated with one (1) external existing drainage area as depicted in **Figure EX-E**, summarized in **Table 7-1** and further discuss below.

Table 7-1 Existing Drainage Areas - Area E

Area D			External to Area D		
Area ID	Area (ha)	RC (-)	Area ID	Area (ha)	RC (-)
S01-E	0.32	0.65	E01-E	0.83	0.90
S02-E	0.31	0.65			
S03-E	0.03	0.30			
Total	0.66	0.41	Total	0.83	0.90

- Catchment S01-E currently sheet drains overland to a grassed swale south of the site (within the hydro corridor), ultimately discharging to Appleby Creek.
- Catchment S02-E currently sheet drains overland to the hydro corridor, ultimately discharging to Appleby Creek
- Catchment S03-E currently sheet drains overland to the Dundas Street right-of-way, ultimately discharging to West Sheldon Creek.

The total existing drainage from Area E is 0.66 ha with an average runoff coefficient of 0.41.

There is one (1) external drainage subcatchments that are conveyed through Area E.

- Catchment E01-E is a section of Dundas Street ROW that sheet drains to an existing overland flow swale along the west limit of Area E, which flows to the grass swale within the hydro corridor, ultimately discharging to Appleby Creek.

The total external drainage being conveyed through Area E is 0.83 ha with an average runoff coefficient of 0.90.

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

7.2. DEVELOPMENT CONCEPT

7.2.1. LAND USE

Area E will be comprised of a six (6) storey condominium with one (1) level of underground parking as shown in Site Plan dA1.2 within the Proposed Residential Development package by Kirkor Architects and Planners as submitted under separate cover with this submission. Note, the privacy feature between existing residential and proposed condominium interface 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. Trees are preserved wherever possible as illustrated in Arborist Report Figure TP-5 by Beacon included in **Appendix L**. For details on ownership and condition ratings, refer to Arborist Report TP-5.

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

7.2.2. 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 and property lines
- Application of the City of Burlington standards

The grading plan, **Drawing GR-E**, is consistent with the City standards. In general, grading of the site blocks adjacent to the surrounding development, hydro corridor, and Dundas Street matches the existing and ultimate (with respect to Dundas Street) anticipated grades at the property line, as appropriate. Note, trees for preservation and associated protection fencing by Beacon is not applicable for **Drawing GR-E** as per Drawing TP-5 included in **Appendix L**.

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

7.3. SANITARY SERVICING

The sanitary servicing criteria for Area E is in accordance with the Regional Municipality of Halton's Water and Wastewater Linear Design Manual (October 2019) as listed in **Table 7-2**.

Table 7-2 Sanitary Servicing Criteria - Area E

Parameter	Value	Parameter	Value
Average Domestic Flow	275 L/capita/day	Population (Condominium)	3.20 person per unit
Infiltration / Inflow	0.286 L/s/ha	Unit Count (#)	130
Peaking Factor (M)	Harmon Formula	Area (ha)	0.66
Minimum	4.50		
Maximum	2.00		

7.3.1. EXISTING

The nearest existing sanitary infrastructure to the subject property is a 600mm 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 conveyed southwest to the Skyway Wastewater Treatment Plant via an 1800mm diameter trunk sewer.

7.3.2. 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. Installation of a new 250 mm sanitary sewer along Dundas Street will be required to connect to the existing sanitary sewer on Millcroft Park Drive at the intersection of Dundas Street and 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 7-3** below for a summary of the sanitary servicing for Area E.

Table 7-3 Sanitary Servicing Summary - Area E

Area	Sanitary Area (ha)	# of Units	Population Equivalent Density (ppu)	Dry Weather Flow (L/s @ 0.286 L/c/d)	Infiltration / Inflow (L/s @ 0.286 L/s/ha)	Peaking Factor	Total Sanitary Flow (L/s)	Connecting Manhole
E	0.66	130	3.20	1.32	0.19	4.01	5.5	EX. MH. 4A

Based on the above criteria and the InfoSewer project report, it has been determined that there is ample capacity to service the subject lands. Note, the groundwater flows, as intercepted by the foundation drain

collectors, may outlet to the sanitary sewer system as proposed on Dundas Street which impacts the percent full sewer capacity by a 1% increase as demonstrated in the sanitary design sheets included in **Appendix E**.

*Refer to **Figure SAN-E** for the sanitary servicing plan for Area E*

7.4. 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 7-4**.

Table 7-4 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

7.4.1. 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:

- 300mm watermain on Millcroft Park Drive (approximately 300 m east of the site).

7.4.2. 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 7-5** and further discussed below.

Table 7-5 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), pressure reduction may be required.

- 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.

- 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 250mm watermain connecting to the 300mm watermain on Millcroft Park Drive. Note, the installation of a 250mm watermain within the Dundas Street West ROW is required to service Area E. 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 Hydraulic Analysis Report (MES, March 2020) in **Appendix F** and **Figure WM-E** for the Water Distribution Plan for Area E.*

7.5. STORM SERVICING

7.5.1. PROPOSED MINOR SYSTEM

Storm servicing for the development will conform to City of Burlington standards. All 5-year storm event flows will be conveyed overland and intercepted via a double catchment basin manhole within the south-west corner of the parking lot prior to the ultimate discharge to Appleby Creek as shown on **Figure STM-E**.

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. Note, the groundwater flows, as intercepted by the foundation drain collectors, may outlet to the storm outfall structure which negligibly impacts the percent full capacity as demonstrated in the storm design sheets included in **Appendix G**.

Refer to **Figure STM-E** for the Storm Drainage Plan and **Appendix G** for the Storm Sewer Design Sheets for Area E.

7.5.2. PROPOSED MAJOR SYSTEM

The major system flows from Area E will be captured prior to discharge into the grassed swale within the hydro corridor. The major system flows are shown on **Figure STM-E**.

The overland flow swale has been sized to convey / maintain the major system drainage from Dundas Street as calculated in **Appendix G.**

*Refer to **Appendix G** for the Overland Flow Route Calculations for Area E.*

7.6. STORMWATER MANAGEMENT

7.6.1. DESIGN

As noted previously, the existing Millcroft Community was developed in the mid-1980s and pre-dates the MECP SWM Guidelines (2003). As such, erosion and quantity control are not required. However, water quality benefits will be provided as part of LID treatment train approach as discussed in **Section 7.6.2**.

*Refer to **Figure PR-E** for the stormwater management plan for Area E.*

7.6.2. QUALITY CONTROL

The proposed development discharges to a grassed swale within the hydro corridor which ultimately outlets to Appleby Creek. The existing grassed swale located along the western boundary of Area E will be restored as an enhanced grassed swale to provide quality control as well as minor flows will be captured in an oil & grit separator (OG/S) prior to discharging into the hydro corridor. Note, CB shields will be provided where necessary to provide a water quality treatment train approach in conjunction with the OG/S. In addition, lot level low-impact development (LID) controls are proposed within Area E to support water quality benefits by a treatment train approach as illustrated in **Figure LID-E**.

7.6.3. QUANTITY CONTROL

As stated in **Section 3.4**, quantity control criteria was not required for the historic/existing development and is not be required under the current master drainage plan. The general approach applied to new development involves control of the post-development flow rates to pre-development levels.

The majority of minor system flows will be intercepted by a double catchbasin manhole located at the south-west corner of the parking lot and major system flows will be captured by a grassed swale within the hydro corridor for ultimately discharge to Appleby Creek. The remaining minor and major system flows are generally split as discharging uncontrolled as sheet flow to either Dundas Street or the hydro corridor lands for ultimate discharge to Appleby Creek. Stormwater management calculations can be found in **Appendix H**.

A summary of the flows for Area E can be found in **Table 7-6** below:

Table 7-6 Stormwater Management Summary for Area E

	Location	Flow Rates (m^3/s)					
		2-year	5-year	10-year	25-year	50-year	100-year
Existing	Outlet 1E	0.200	0.259	0.297	0.380	0.458	0.521
Proposed (Uncontrolled)	Outlet 1E	0.253	0.327	0.375	0.437	0.481	0.526

*The proposed SWM drainage areas are included on **Figure PR-E** and the storm servicing strategy is shown on **Figure STM-E**.*

7.6.4. WATER BALANCE

A water balance was prepared by RJB in conjunction with their hydrogeology report. A copy of the report is 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 37% of the average annual groundwater infiltration could occur (refer to RJB Report Table G-3, Appendix G in **Appendix B**). As such, the proposed SWM strategy recommends additional topsoil where appropriate to mitigate any groundwater impacts. This will result in a potential increase of 1% of the average annual groundwater infiltration.

7.7. IMPLEMENTATION

Coordination with the Region of Halton is required for the sanitary and watermain service connections. It is anticipated that the agency review of Dundas Street widening project Stage 1 90% detailed design and Stage 2 60% detailed design is imminent.

The limit of excavation for external servicing works within the Dundas Street right-of-way to external services is depicted for Area E in the Construction Management and Mobility Plan as **Figure CMMP-E**. The limits of underground parking, as informed by shoring component details prepared by Terra Engineering & Structural Consulting Inc., are included on Figure CMMP-E as well as associated offset dimensions to the property limit and proposed Dundas Street right-of-way limits. Also, **Figure CMMP-E** includes the crane location and swing radius as prepared by 59 Project Management Inc. and associated construction feasibility discussion as included in **Appendix M**.

*Refer to **Figure CMMP-E** for the Construction Management and Mobility Plan for Area E as appended to Shoring Plans, Sections and Details as Drawings SHO-SH3 prepared by Terra Engineering & Structural Consulting Inc. and the Area E Crane Review Letter completed by 59 Project Management Inc. as included in **Appendix M**.*

7.8. 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 preliminary acoustic mitigation measures on potential outdoor living areas (OLA) for Area E. Acoustic mitigation measures will be provided as necessary upon confirmation of all OLA locations at the site plan design stage.

*Refer to **Appendix K** for the Noise and Vibration Feasibility Study completed by HGC (November 2020).*

7.9. 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 on Dundas Street which will connect to the existing 600 mm diameter municipal sanitary sewer within Millcroft Park Drive.
- Water servicing for domestic potable and fire protection will be through connections along Dundas Street to the existing 250 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.
- Storm drainage from the site will discharge into the adjacent watercourse to the southwest via a grassed swale through the hydro corridor.
- 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.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.
- The development of the site is feasible from a noise & vibration perspective.

8 EROSION & SEDIMENT CONTROL

The erosion and sediment control (E&SC) plan for the site servicing program of the subject lands will be designed, approved, and implemented in conformance with the City of Burlington, CH and MECP recommendations.

Erosion and sediment control will be implemented for all construction activities including topsoil stripping, foundation excavation and stockpiling of materials. During construction, temporary sediment ponds may be required to treat pre-development drainage from stripped areas.

The proposed reconfigured offline Mill Pond at Upper Middle Road will be used to detain sediment laden runoff and reduce peak flows and velocities prior to release into the receiving system. The temporary silt pond will maintain a permanent pool as per the MOE guidelines for temporary sediment control facilities. Forebay areas will be provided to enhance sediment removal.

The following erosion and sediment control measures will be installed and maintained during construction:

- A temporary sediment control fence will be installed prior to grading along the perimeter of the proposed area of disturbance
- A construction plan will be implemented to limit the size of disturbed areas and to minimizing non-essential clearing
- Sediment traps will be provided
- Gravel mud mats will be provided at construction vehicle access points to minimize off-site tracking of sediments
- All temporary erosion and sediment control measures will be routinely inspected and repaired during construction. Temporary controls will not be removed until the areas they serve are restored and stable.

Recognizing that erosion and sediment control is a dynamic process, a detailed set of staging plans / construction sequencing will be required for the various stages of creek work, earthworks, servicing, site plan construction, and stabilization, coupled with the proposed development phasing. Each Area may require unique E&SC measures. The E&SC plans will be prepared by a qualified professional (i.e., CISEC, CPESC or an approved equivalent).

9 CONCLUSION

It can be noted that each of the four (4) study areas (Areas A & B, Area C, Area D & Area E) 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 200 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.
- Storm drainage will include the construction of local storm sewers designed to convey the 5-year flow to the proposed Mill Pond east of Appleby Creek.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks in order to maximize reuse of soils within the property and avoid the need for retaining walls.
- Quality control will be provided through the reconfigured pond.
- Stormwater quantity control is not required for Area A. However, flows from Area B will be directed to Hadfield Court, and the existing external area will be routed around the east of Hadfield Court along with major system flows from Area B. This ensures that the Hadfield Court sewers do not receive excess runoff. Also, quantity control is provided by the active storage component of the reconfigured offline Mill Pond by reducing post-development peak flows below existing peak flow rates.
- 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 designed to convey the 5-year flow.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks in order to maximize reuse of soils within the property and avoid the need for retaining walls.
- Stormwater quantity control is not required for Area C. However, flows will be controlled to the pre-development level via a 189 m^3 super pipe and a 100 mm diameter orifice tube. Major system flows from Area C will be captured in sewers and directed to the existing storm network.
- 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 designed to convey the 5-year flow.
- 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 is not required for Area D. However, flows will be controlled to the pre-development level through combination of sending excess flows to the pond north of the site and by controlling the remaining flows via a 39 m^3 super pipe and a 110 mm diameter orifice tube.
- Erosion and Sediment Control measures will be designed in accordance with City of Burlington, MECP and CH requirements.

Area E:

- Sanitary servicing will be accomplished through the construction of local sanitary sewers which will connect to the existing 600 mm diameter municipal sanitary sewer within Millcroft Park Drive.
- Water servicing for domestic potable and fire protection will be through connections to the existing 250 mm diameter watermain within Millcroft Park Drive and the construction of local watermains.
- Storm drainage from the site will discharge into the adjacent watercourse to the southwest via a grassed swale.
- Grading will be in accordance with City of Burlington requirements and minimize on site earthworks in order to maximize reuse of soils within the property and avoid the need for retaining walls.
- 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.

Report Prepared by:





Andrew Fata, M.Sc.Eng., P. Eng.
Senior Associate, Water Resources

Paul Chiocchio, EIT
Water Resources Coordinator

FIGURES & DRAWINGS

APPENDIX A

GEOTECHNICAL FEASIBILITY STUDY (DSCL)

APPENDIX B

HYDROGEOLOGICAL REPORT
FOUNDATION DRAIN ANALYSIS MEMO
(RJB)

APPENDIX C

TUCK, SHOREACRES, APPLEBY AND SHELDON CREEKS WATERSHED STUDY LETTER (City of Burlington, August 1995)

APPENDIX D

SANITARY SEWER AS-CONSTRUCTED DRAWINGS
(City of Burlington)

APPENDIX E

SANITARY SEWER DESIGN SHEETS

APPENDIX F

WATER ASSESSMEMT REPORT (MES)

APPENDIX G

STORM DESIGN CALCULATIONS

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

APPENDIX H

SWM DESIGN CALCULATIONS

Existing Drainage Area Breakdown
SWM Calculations

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

AREA E CRANE REVIEW
(59 Project Management Inc.)