FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

In Support of a Zoning By-Law Amendment



1600 Kerns Road, City of Burlington

Proposed Retirement Home Development

Report Prepared by:

2453 Auckland Drive

Burlington, Ontario

L7L 7A9

March 26th 2021

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- 18-30-288-00-A – Topographic Survey by J.D. Barnes
- dA1.03 – Architectural Site Plan by Kirkor Architects + Planners
- Existing Drainage Plan – EDP
- Proposed Drainage Plan – PDP
- Proposed Quality Control Plan – PQCP
- Proposed Emergency Overland Drainage Plan - PEODP
- Site Grading Plan – SGP
- Site Servicing Plan – SSP
- Records Drawing - E-6-5 Kerns Road Plan and Profile dated September 1993.

1.0 INTRODUCTION

KWA Site Development Consulting Inc. ("KWA") has been retained by FGL Kerns Inc. to prepare this Functional Servicing & Stormwater Management Report in support of a Zoning By-Law Amendment application for the re-development at 1600 Kerns Road in the City of Burlington, herein referred to as the "Site".

The Site is bounded by Four Seasons Drive to the north, Kerns Road to the east, the existing Tyandaga Mews townhouse development to the south and the Upper Hager Creek to the west. The Site currently contains a 1-storey multi-unit commercial building. Refer to Figure 1 – Site Location Plan.

The total Site area is approximately 0.8 ha (1.98 acres), which includes the Upper Hager Creek regulated area to the west. Refer to legal survey 18-30-288-00-A by J.D. Barnes dated September 2018. A top of bank limit was staked by Conservation Halton (CH) on November 29th, 2019 and shown on the legal survey. The proposed re-development area, excluding the Upper Hager Creek regulated area to the top of bank limit, is 0.63ha.

The development proposal contemplates a four-storey retirement home facility with one underground parking level and associated landscape and amenity areas. The proposed underground parking limit will cover approximately 0.37 ha (58%) of the 0.63 ha re-development area. Refer to architectural Site Plan dA1.03 by Kirkor attached.

2.0 STUDY PARAMETERS

This Functional Servicing and Stormwater Management Report has been prepared in coordination and with regard to the following documents and drawings:

- Architectural Plans prepared by Kirkor.
- Topographic and Legal Survey by J.D. Barnes
- Landscape Design by Adesso Design Inc.
- Hydrogeological and Water Balance Investigation by EXP Services Inc.
- Environmental Impact Assessment by PLAN B Natural Heritage
- Regional Municipality of Halton–Water and Wastewater Linear Design Manual (October 2019 Version 5)
- City of Burlington - Stormwater Management Design Guidelines (May 21, 2020 Edition)
- Upper Hager Creek – Escarpment to Four Seasons Drive – Class Environmental Assessment City of Burlington by Amec Foster Wheeler Environment and Infrastructure November 2015.



KWA SITE DEVELOPMENT CONSULTING INC.
2453 Auckland Drive
Burlington, ON L7L 7A9

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL

FIGURE 1 - SITE LOCATION PLAN

RETIREMENT HOME DEVELOPMENT

1600 KERNS ROAD

BURLINGTON, ONTARIO

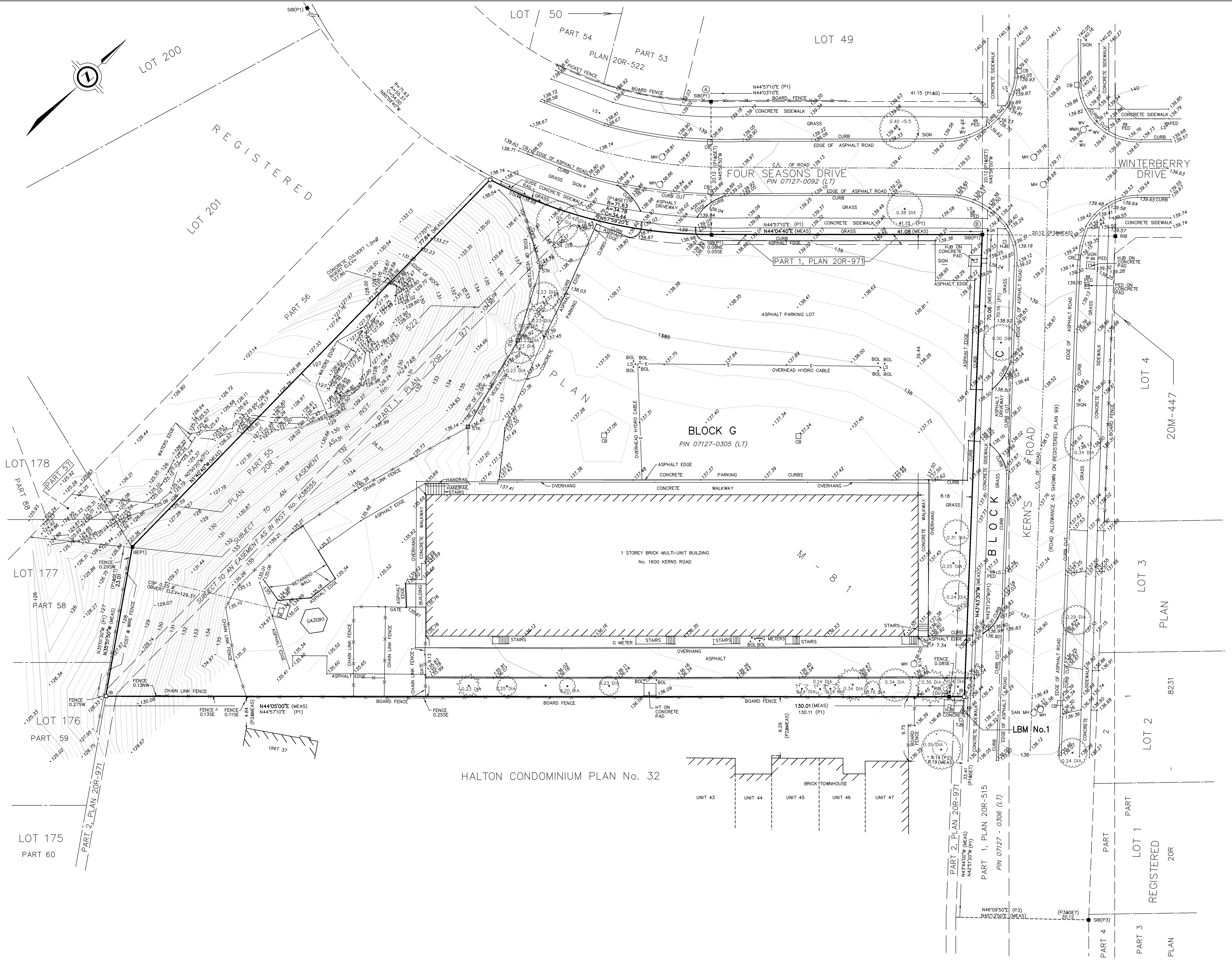
PROJECT No: 20500	DRAWN BY:BC2	CHECKED BY:CDB

NORTH ARROW

METRIC SCALE

N.T.S.

FIG 1



SURVEYOR'S REAL PROPERTY REPORT
SHOWING TOPOGRAPHIC INFORMATION ON

BLOCK G
REGISTERED PLAN M-81
CITY OF BURLINGTON
REGIONAL MUNICIPALITY OF HALTON

SCALE 1 : 250

5 0 5 10 15 metres

J.D. BARNES LIMITED

© COPYRIGHT

METRIC DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

PART 2 - SURVEY REPORT

- DESCRIPTION

BLOCK G, REGISTERED PLAN M-81, CITY OF BURLINGTON

- ADDITIONAL REMARKS

PLAN PREPARED FOR FIELDGATE PROPERTIES

REGISTERED EASEMENTS AND/OR RIGHTS OF WAY

SUBJECT TO AN EASEMENT AS IN INST. No. H24748 OVER PART 55 ON PLAN 20R-522.
SUBJECT TO AN EASEMENT AS IN INST. No. H38555 OVER PART 1 ON PLAN 20R-971.

- BOUNDARY FEATURES

NOTE THE LOCATION OF FENCES AS SHOWN ON THE PLAN

NOTES

BEARINGS ARE UTM GRID, DERIVED FROM (ORP's) OBSERVED REFERENCE POINTS A AND B, BY REAL TIME NETWORK (RTN) OBSERVATIONS, UTM ZONE 17, NAD83 (CSRS)(2010.0).

INTEGRATION DATA

OBSERVED REFERENCE POINTS (ORPs): UTM ZONE 17, NAD83 (CSRS) (2010.0).

COORDINATES TO URBAN ACCURACY PER SECTION 14 (2) OF O.REG 216/10.

POINT ID	EASTING	NORTHING
ORP (A)	594 128.12	4 799 342.19
ORP (B)	594 171.15	4 799 357.70

COORDINATES CANNOT, IN THEMSELVES, BE USED TO RE-ESTABLISH CORNERS OR BOUNDARIES SHOWN ON THIS PLAN.

DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999896.

BEARING COMPARISONS SHOWN FOR UNDERLYING PLANS, REGISTERED PLAN M-81, HALTON CONDOMINIUM PLAN No. 32 AND PLAN 20R-515 ARE WITH ASTRONOMIC BEARINGS.

ALL BUILDING TIES ARE TAKEN TO CONCRETE FOUNDATION.

LEGEND

△	DENOTES BENCHMARK
■	DENOTES SURVEY MONUMENT FOUND
□	DENOTES SURVEY MONUMENT PLANTED
IB	DENOTES IRON BAR
SIB	DENOTES STANDARD IRON BAR
⊠ STK	DENOTES WOOD STAKE PLANTED
P1	DENOTES REGISTERED PLAN M-81
P2	DENOTES HALTON CONDOMINIUM PLAN No. 32
P3	DENOTES PLAN 20R-515
P4	DENOTES BUILDING LOCATION SURVEY BY J. D. BARNES LIMITED, DATED JUNE 23, 1988.
P5	DENOTES BUILDING LOCATION SURVEY BY J. D. BARNES LIMITED, DATED JUNE 23, 1988.
P6	DENOTES PLAN 20R-8231
MEAS	DENOTES MEASURED
CB	DENOTES SINGLE CATCHBASIN
HJB	DENOTES HYDRO JUNCTION BOX
GK	DENOTES GAS KEY
G METER	DENOTES GAS METER
MH	DENOTES MANHOLE
SAN MH	DENOTES SANITARY MANHOLE
WMH	DENOTES WATER MANHOLE
BOL	DENOTES BOLLARD
LS	DENOTES LIGHT STANDARD
TJB	DENOTES TELEPHONE JUNCTION BOX
PED	DENOTES TELEPHONE PEDESTAL
H	DENOTES FIRE HYDRANT
HT	DENOTES HYDRO TRANSFORMER
WV	DENOTES WATER VALVE
— E —	DENOTES OVERHEAD HYDRO CABLE
CSP	DENOTES CORRUGATED STEEL PIPE

ELEVATION NOTE

ELEVATIONS SHOWN ON THIS PLAN ARE DERIVED FROM CITY OF BURLINGTON BENCHMARK No. 1081
ELEVATION = 157.586m
AND
BENCHMARK No. 1082
ELEVATION = 139.47m

LOCAL BENCHMARK No. 1

BENCHMARK No. 503 FOUND ON SOUTHEAST CORNER OF HYDRO PAD ACROSS FROM LOT 2, 20M-447.
ELEVATION = 136.52m

BEFORE DIGGING, UNDERGROUND SERVICES SHOULD BE LOCATED ON SITE BY THE RESPECTIVE AGENCIES.

IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO VERIFY THAT THE LOCAL BENCHMARKS HAVE NOT BEEN ALTERED OR DISTURBED AND THAT THE RELATIVE ELEVATONS AND DESCRIPTIONS AGREE WITH THE INFORMATION SHOWN ON THIS SKETCH.

SURVEYOR'S CERTIFICATE

I CERTIFY THAT:

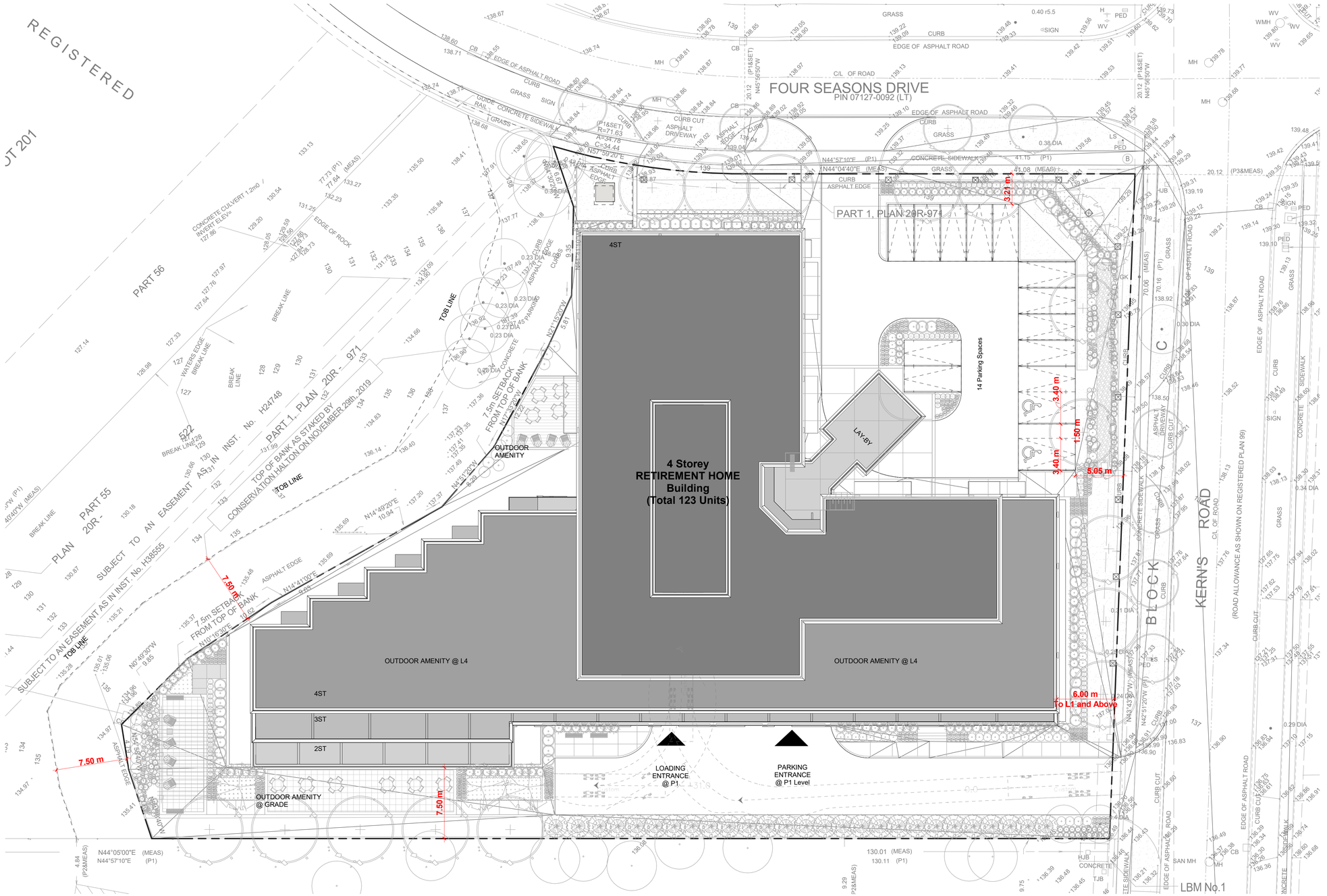
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON SEPTEMBER 13, 2018.

DATE

THOMAS J. SALB
ONTARIO LAND SURVEYOR

J.D.BARNES LIMITED
SURVEYING MAPPING GIS
LAND INFORMATION SPECIALISTS
401 WHEELABRATOR WAY, SUITE A, MILTON, ON L9T 3C1
T: (905) 875-9955 F: (905) 875-9956 www.jdbarnes.com

DRAWN BY:	CHECKED BY:	REFERENCE NO.:
CE		18-30-288-00-A
FILE: G:\18-30-288\00\Drawing\183028800-A.dgn		DATED: SEPTEMBER 14, 2018
		PLOTTED: 24/09/2018



Site Plan
Scale: 1 : 200

1
dA1.03

Plot Date: 3/22/2021 2:10:24 PM File Path: C:\Revit\2019\19062P11-Model-Fieldgate - 1600 Kerns_FGorjdooz.rvt

Contractor Must Check And Verify All Dimensions On The Job.

Do Not Scale The Drawings.

All Drawings, Specifications And Related Documents Are The Copyright Of The Architect And Must Be Returned Upon Request.

Reproduction Of Drawings, Specifications And Related Documents In Part Or Whole Is Forbidden Without The Architects Written Permission.

This Drawing Is Not To Be Used For Construction Until Signed ByThe Architect.

Date:

KIRKOR ARCHITECTS + PLANNERS

20 De Boers Dr. # 400 Toronto ON M3J 0H1
TEL 416 665 6060 kirkorarchitects.com

No.:	Revision:	Date:

DRAFT

07	Review	Mar 22, 2021
06	Review	Mar 17, 2021
05	Review	Mar 09, 2021
04	Coordination	Feb 19, 2021
03	Review	Jan 21, 2021
02	Client's Review	May 27, 2020
01	Client's Review	Oct 07, 2019
No	Issued For:	Date:

Drawing Title:

Site Plan

Project:

FIELDGATE PROPERTIES

1600 KERNS RETIREMENT HOME

1600 KERNS ROAD, BURLINGTON, ON

Scale: 1 : 200

Drawn by: Author

Checked by: Checker

Project No.: 19062

Date: Mar 22,2021

Drawing No.:

dA1.03

3.0 DESIGN POPULATION

The proposed re-development contemplates a total of 127 (1-bedroom) residential units with approximately 23 staff members and 24 visitors per day. The projected design population for the re-development is **174 persons.**

4.0 WATER SUPPLY

The Site is within Region of Halton Pressure Zone B3. The pressure zone is located north of Upper Middle Road in Burlington and includes all lands with ground elevations between 129.7 and 165.9m west of Bronte Creek. Water within this pressure zone is supplied by the Washburn Pumping Station. The Headon Road and Tyandaga Reservoirs provide storage and regulate pressure within this zone.

The existing water infrastructure adjacent to the Site consists of a 200mm PVC watermain within the west boulevard of Kerns Road and a 250mm PVC watermain within the east boulevard of Kerns Road. The 250mm watermain on the east side of Kerns Road is used for water distribution. The Region of Halton own and operate the local water distribution system. The existing commercial building is serviced off the 250mm PVC watermain within the west boulevard of Kerns Road.

There are two existing municipal hydrants adjacent to the Site. One hydrant is located at the intersection of Kerns Road and Four Seasons Drive, at the north-west corner. The second hydrant is located just south of the Site at the entrance to the Tyandaga Mews townhouse development. The existing commercial Site does not have private hydrants.

4.1 SUPPLY DEMANDS

The domestic and fire water demand for the Site was calculated based on the Region of Halton's Water and Wastewater Linear Design Manual Section 2 – Watermain Design. The relevant design criteria are summarized below:

- Average Daily Demand = 0.275 m^3 / capita / day
- Maximum Daily Demand Peaking Factor = 2.25
- Maximum Hourly Demand Peaking Factor (Residential) = 4.00
- Design Water Demand to meet the greater of the following:
    - Maximum Daily Demand Plus Fire Flow
    - Maximum Hourly Demand
- System Requirements:
    - Minimum pressure during max day plus fire flow demand = 140 kPa (20 psi)
    - Normal Operating Pressure in the system = 350 kPa to 550 kPa (50 to 80 psi) and not less than 275 kPa (40 psi)
    - Maximum pressure in the distribution system should not exceed 700 kPa (100 psi)
- Standard fire service connection and domestic water service connection installations based on Halton Region Standard Drawing RH 409.010. For fire service connections, the minimum pipe size shall be 200mm diameter.

The domestic and fire water demand for the subject site was calculated based on the Region of Halton's demand criteria and Fire Underwriters Survey calculation. The detailed water demand calculations are summarized in Table 1 below.

Table 1: Water Supply Demand

Building	Population	Average Daily Demand [1.] (L/s)	Max. Daily Peaking Factor	Max. Hourly Peaking Factor (Res.)	Max Daily Demand [2.] (L/s)	Peak Hourly Demand [3.] (L/s)	Fire Demand [4.] (L/s)
Building A	174	0.55	2.25	4.00	**1.24**	**2.21**	**193**

1. Average Daily Demand = 275 L/cap/day x population
2. Max Daily Demand = Average Daily Demand x Max Daily Peaking Factor
3. Max Hourly Demand = Average Daily Demand x Max. Hourly Peaking Factor
4. Fire Demand estimated based on the Fire Underwriter Survey Calculation found in Appendix A

The design flows applied to the proposed watermain connections are as follows:

- Maximum Daily Demand + Fire Flow = **194.24 L/s**
- Maximum Hourly Demand = **2.21 L/s**

4.2 PROPOSED CONNECTION AND LAYOUT

A new fire and domestic water service will connect to the existing 250mm watermain on the east side of Kerns Road. A 250mm fire service and 100mm domestic service, as per Region standard RH 409.010, will be brought to the buildings mechanical room located at the south east corner of the Site. A fire hydrant will be provided within the fire access route to ensure a hydrant is located 45m from the buildings siamese connection. The on-site hydrant will be located above the underground parking structure therefore the water service to the hydrant will be serviced internally.

The Region of Halton water model (updated June 2020) was reviewed to confirm available fire flows and pressures under the 2021 and 2031 scenarios. Refer to water model schematic shown in Appendix A. At the existing Site connection node, WFT92380, the available fire flows and pressures are as follows:

Table 2 – 2021 and 2031 Fire Flows and Domestic Pressures at Node WFT92380

Node ID	2021 Fire Flow Available (L/s)	2021 Pressure			2031 Fire Flow Available (L/s)	2031 Pressure		
		Max Day (psi)	Avg Day (psi)	Peak Hour (psi)		Max Day (psi)	Avg Day (psi)	Peak Hour (psi)
WFT92380	136.9	86	82	82	170.6	88	85	81

It is assumed that a fire suppression sprinkler system, designed to NFPA 13 standards, will be provided for the proposed 4-storey retirement building. At this stage, the FUS building construction variables such as "Fire Resistant" and "Non-Combustible" construction are unknown. The more conservative "Non-Combustible" factor was used in the above FUS calculation.

To provide the adequate fire flows in this area, further detail regarding building material construction and collaboration with the architect, mechanical consultant, and fire suppression engineer will be required to confirm the fire pipe service size requirements and fire demands for the re-development.

A hydrant flow test is not available at this time. Confirmation on available pressures and capacity within the system will be further analysed and modelled when the hydrant flow test results have been completed in the spring of 2021.

Refer to Figure 2 – Proposed Site Servicing Figure for location and sizes of proposed water services.

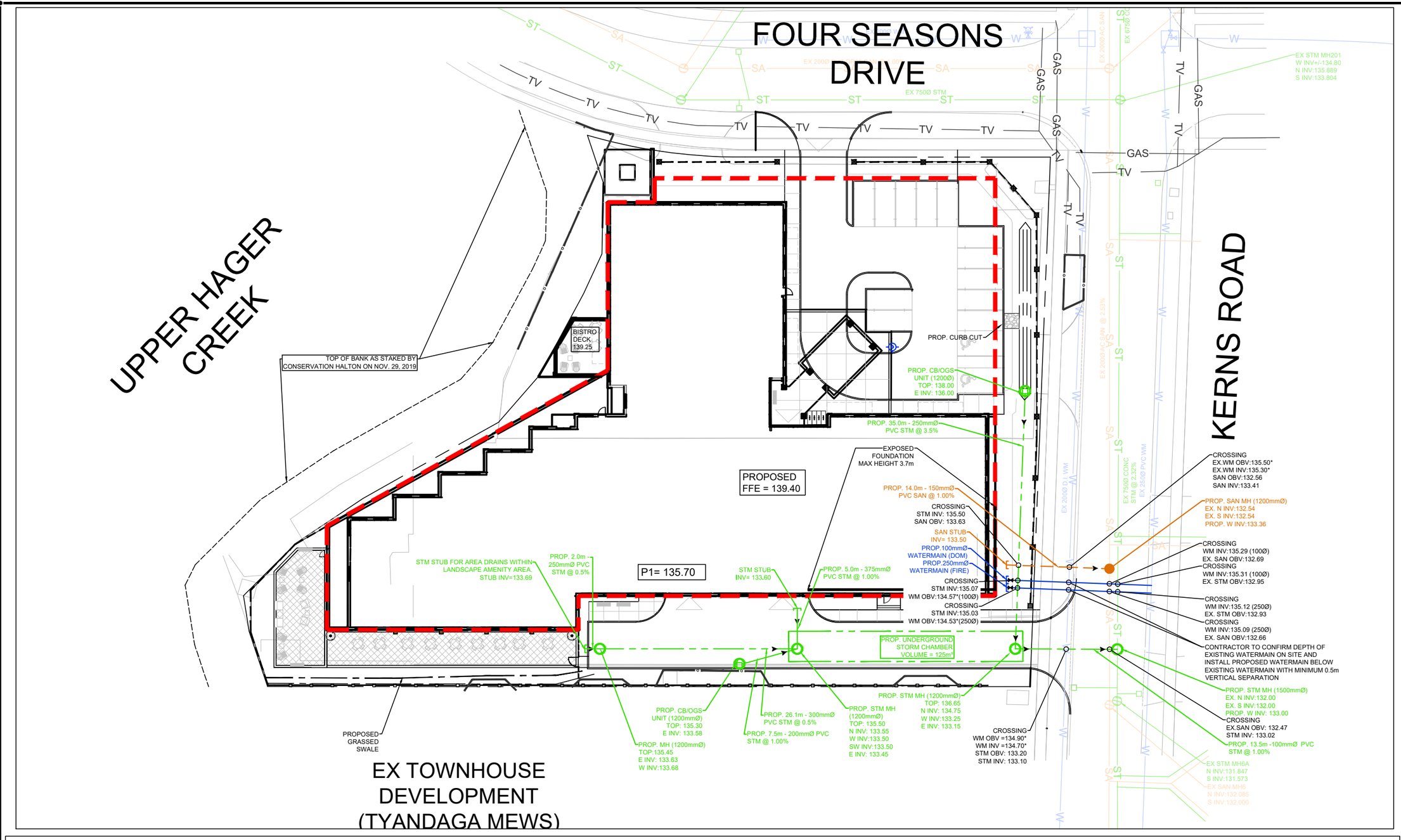
5.0 SANITARY DRAINAGE

The existing sanitary sewer adjacent to the Site consists of a 200mm PVC sanitary sewer on Four Seasons Road draining east and a 200mm PVC sanitary sewer on Kerns Road draining south. The Region of Halton own and operate the sanitary sewer system. The existing commercial Site is serviced off the 200mm PVC sanitary sewer on Kerns Road.

5.1 SANITARY DESIGN FLOW

The sanitary design flows for the Site have been calculated using the Region of Halton's Water and Wastewater Linear Design Manual – Section 3.2. The relevant design criteria are summarized below:

- Unit Sewage Flow = 0.275 m^3 / capita / day
- Equivalent Population Density (Light Commercial) = 90 persons/hectare
- Proposed Population Density = 174 persons



LEGEND

- PROPOSED UNDERGROUND PARKING LIMIT
- EXISTING STORM SEWER
- PROPOSED STORM SEWER
- EXISTING SANITARY SEWER
- PROPOSED SANITARY SEWER
- EXISTING WATERMAIN
- PROPOSED WATERMAIN
- PROPOSED VALVE
- PROPOSED HYDRANT
- EXISTING STORM MANHOLE
- EXISTING CATCHBASIN
- PROPOSED STORM MANHOLE
- PROPOSED CATCHBASIN MANHOLE
- EXISTING SANITARY MANHOLE
- PROPOSED SANITARY MANHOLE
- EXISTING VALVE
- EXISTING HYDRANT

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL

KWA SITE DEVELOPMENT CONSULTING INC.
2430 Fairview St. Unit 7
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE 0 10 1:500 Metres

FIGURE 2 - PROPOSED SITE SERVICING PLAN
RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO
PROJECT No: 20300 | DRAWN BY: BC | CHECKED BY: BC

FIG 2

- Infiltration Flow = 0.286 L / ha / s
- Peaking Factor = Calculated using the Harmon Formula

5.2 Existing Sanitary Flow

The Site's current use is a 1-storey multi-unit commercial building which drains to the 200mm sanitary sewer on Kerns Road.

Using a total site area of 0.63 ha, equivalent population density of 57 persons, calculated Harmon's Peaking factor of 4.3, and infiltration flow rate of 0.286 L/s/ha, the pre-development sanitary sewer flow is:

$Q_{san\ ex}$ = (275 Lpcd x 57 persons x $4.3_{peaking\ factor}$ /86400) + (0.286 L/s/ha x 0.63 ha) = **0.95 L/s**

5.3 Proposed Sanitary Flow

Using a proposed total population density of 174 persons, calculated Modified Harmon's Peaking factor of 4.2, design flow rate of 275 L/capita/day and infiltration flow rate of 0.286 L/s/ha, the post development sanitary sewer flow is:

$Q_{san\ post}$ = (275 Lpcd x 174 persons x $4.2_{peaking\ factor}$ / 86400) + (0.286 L/s/ha x 0.63 ha) = **2.49 L/s**

The proposed peak sanitary sewer flow from this Site will increase by **1.54 L/s** (2.49 L/s – 0.95 L/s) over existing conditions.

Refer to Appendix B for detailed design sanitary calculations.

5.4 Proposed Connection and Layout

Based on the post development design flow, a 150 mm sanitary service connected to the existing 200mm PVC sanitary sewer on Kerns Road will be required to support the proposed development. A new 150 mm service connection at 1.00% slope will be brought to the underground parking limit by gravity and connect to the existing sanitary sewer at elevation 133.36.

The capacity of the proposed 150mm sanitary service at 1.00% is **15.2 L/s**. The existing capacity of the 200mm sanitary sewer at 2.53% on Kerns Road is **52.2 L/s**. The 2.49 L/s theoretical sanitary flow from the proposed redevelopment equates to 4.8% of the capacity of the existing 200mm sanitary sewer on Kerns Road.

Refer to Figure 2 – Proposed Site Servicing Figure for location and sizes of proposed sanitary service.

6.0 STORMWATER MANAGEMENT

6.1 PRE-DEVELOPMENT DRAINAGE CONDITIONS

The existing Site drains from northeast (elevation 139.25) to south west (elevation 135.50). Four Seasons Drive drains west towards the Upper Hager Creek with a centreline road slope of 1.30%. Kerns Road drains south with a centreline road slope of 4.00%.

Upon review of the topographic survey and existing records drawings of Kerns Road, the existing site is divided into two drainage areas. The existing commercial parking area and rainwater roof leaders draining the northern half of the commercial building, Area 101, are collected by two catchbasins within the existing parking area. Based on records drawings of Kerns Road, drawing number E-6-5 Kerns Road plan and profile as-constructed dated September 1993, a 300mm (12") storm service connection is shown servicing the Site and outletting to the existing 750mm storm sewer on Kerns Road. Its is assumed that Area 101 outlets to the existing 750mm storm sewer draining south on Kerns Road.

Area 102 which includes the southern building roof area and rear access driveway drain west to the Upper Hager Creek. An existing catchbasin at the south west corner of the site outlets directly to the Upper Hager Creek.

Refer to Existing Drainage Plan drawing.

Table 3 – Existing Drainage Area, Composite Runoff Coefficient and Drainage Direction

Area ID	Catchment Area (ha)	Composite Runoff Coefficient	Drainage Direction
101	0.37	0.90	To Kerns Road
102	0.26	0.72	To Upper Hager Creek

Based on City of Burlington Stormwater Design Guidelines, a 0.95 runoff coefficient was used for existing roof and asphalt surfaces. A 0.25 runoff coefficient was used for existing landscape surfaces. During large storm events, the existing sites emergency overland spill point is all directed west to the Upper Hager Creek.

6.2 EXISTING PRE-DEVELOPMENT PEAK FLOWS

Using the Rational Method, City of Burlington 2020 IDF data for the 5-year storm event, composite runoff coefficients defined above, and a time of concentration of 10 minutes, the following table summarizes the existing 5-year pre-development peak flows for the two drainage areas.



LEGEND

102 / A=0.26 / C=0.72	CATCHMENT AREA ID
	CATCHMENT AREA(ha)
	RUNOFF COEFFICIENT
137.80	EXISTING CONTOUR
135.78	EXISTING ELEVATION
	EXISTING OVERLAND FLOW ARROW
	EXISTING DRAINAGE AREA DIVIDE
	EXISTING LANDSCAPE AREA
	EXISTING CATCHBASIN
ST ST	APPROXIMATE LOCATION OF EXISTING STORM SERVICE CONNECTION

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL

LICENSED PROFESSIONAL ENGINEER
C. DEL BUONO
100137127
03/26/21
PROVINCE OF ONTARIO

KWA SITE DEVELOPMENT CONSULTING INC.
2485 Lancaster Drive
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE
0 9 Metres
1:300

EXISTING DRAINAGE PLAN
RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO
PROJECT No.20500 | DRAWN BY:BC2 | CHECKED BY:BC

EDP

Table 4 – Existing 5-year Pre-Development Peak Flow Rates

Area ID	Catchment Area (ha)	Runoff Coefficient	Drainage Direction	5-Year Peak Flow Rate (L/s)
101	0.37	0.90	To Kerns Road	**94**
102	0.26	0.72	To Upper Hager Creek	**53**

6.3 DESIGN CRITERIA

The stormwater management servicing strategy proposed for the development is based on the City of Burlington's Stormwater Management Design Guidelines (May 2020) and the Capital Works Pre-Consultation Meeting minutes (updated October 2020). The following SWM criteria was provided by the City's Stormwater Engineer:

Quantity Control

- Control the 100-year post development storm to the 5-year pre-development storm connection to the municipal storm system.
- If a new outlet to Upper Hager Creek is proposed, then post to pre quantity control.
- Any permanent dewatering of the underground parking structure will need to be accommodated in the storm system, and quantified (estimated) including chemical analysis of the groundwater to confirm if it meets municipal discharge criteria.
- For re-development sites: a maximum runoff coefficient of C=0.5 may be applied
- A minimum orifice diameter of 75mm is allowable for flow restrictors. The City of Burlington requires the use of orifice tubes as opposed to orifice plates, to prevent removal in the future. Orifice tubes to be located on the downstream side of the control structure.
- No surface ponding in a 5-year storm and no surface ponding allowed within accessible parking spaces.
- Rooftop storage will not count towards the stormwater storage requirements.

Enhanced Quality Control

- Achieving 80% Total Suspended Solids Removal. A treatment train approach is recommended incorporating Low Impact Development techniques. OGS units will only be credited at 50%.

Water Balance

- Retain / infiltrate the first 5mm on site.

6.4 Allowable Release Rate

Based on the City of Burlington Quantity Control criteria, for re-development sites a maximum runoff coefficient of C=0.5 may be applied. The following table summarizes the allowable release rates based on the pre-development drainage areas:

Table 5 – Allowable Release Rate with Maximum Runoff Coefficient (C=0.5) for Re-development Sites

Area ID	Catchment Area (ha)	Runoff Coefficient	Drainage Direction	5-Year Release Rate (L/s)
101	0.37	0.50	To Kerns Road	**52**
102	0.26	0.50	To Upper Hager Creek	**37**

Comparing the release rates shown in the tables from Section 6.2 and 6.4, based on the City of Burlington criteria the re-development will reduce the 5-year release rate by 30% to 40% compared to its 5- year release rate outletting today.

6.5 Post-Development Drainage Conditions

In the post development condition, approximately 0.50 ha will drain to underground storm sewers that outlet to Kerns Road. The remaining site area, 0.13 ha of landscaped area will drain to Kerns Road and the Upper Hager Creek. Refer to below summary table:

Table 6– Proposed Drainage Areas, Runoff Coefficient and Drainage Direction

Area ID	Areas	Catchment Area (ha)	Runoff Coefficient	Drainage Direction
201A	NE Parking Area Rooftop South Driveway West Landscape amenity area	0.48	0.89	To Kerns Road
201B	East Landscape Areas	0.02	0.25	To Kerns Road
202	North, South, West Landscape Area	0.13	0.38	To Upper Hager Creek

Refer to Proposed Drainage Plan drawing.

6.6 RUNOFF QUANTITY CONTROL – 100 YEAR POST TO 5 YEAR PRE (C=0.5)

Using the drainage areas defined in Table 6, post-development 100-year safety factor of 1.25 applied to the runoff coefficient, and using a time of concentration of 10 minutes, the 100-year uncontrolled post development peak flows are as follows:

Area 201A and 202B – To Kerns Road:

$I_{100yr} = 1281.34 \times (t+5)^{-0.761} = 1281.34 \times (10+5)^{-0.761} = 163.18$ mm/hr

$Q_{100yr} = A \times R \times I_{100yr} \times 360 = 0.50 \times 1.0 \times 163.18/ 360 = 0.23$ m3/s = **227 L/s**

Area 202 – To Upper Hager Creek:

$I_{100yr} = 1281.34 \times (t+5)^{-0.761} = 1281.34 \times (10+5)^{-0.761} = 163.18$ mm/hr

$Q_{100yr} = A \times R \times I_{100yr} \times 360 = 0.11 \times 0.48 \times 163.18/ 360 = 0.02$ m3/s = **28 L/s**

For Area 201A and 201B, the post development uncontrolled flow of 227 L/s is greater than the 5-year allowable release rate of 52 L/s, therefore stormwater attenuation and storage will be required.

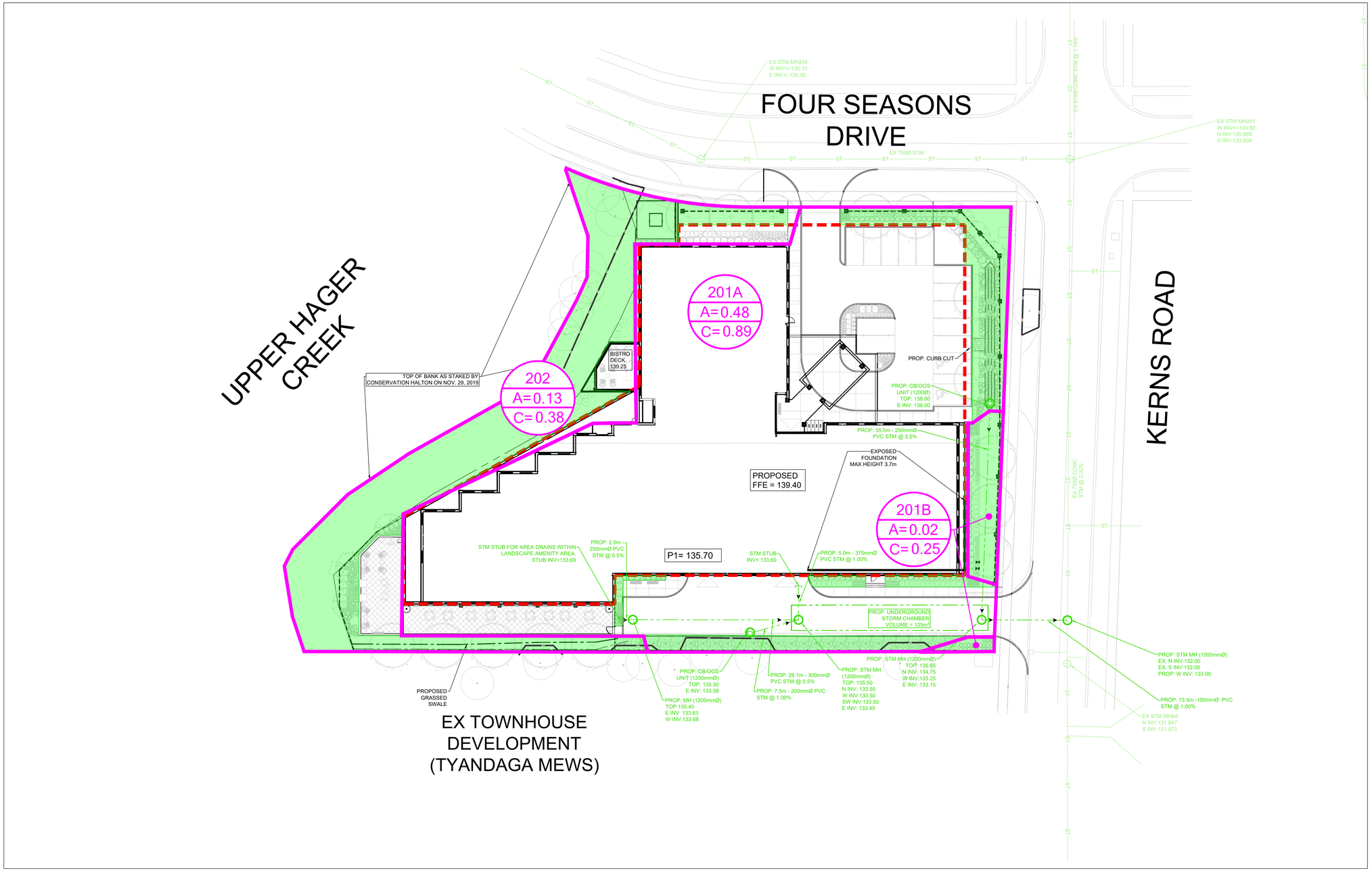
For Area 202, the post development uncontrolled flow of 28 L/s is less than the allowable 5-year allowable release rate of 37 L/s, therefore no stormwater attenuation and storage will be required for this area draining uncontrolled to Upper Hager Creek. It should be noted that the area draining to Upper Hager Creek is predominantly the landscape area within the 7.5m setback area. No new piped outlet to the Upper Hager Creek is proposed.

6.6.1 SHORT TERM AND LONG-TERM DEWATERING

From the Hydrogeological and Water Balance Investigation prepared by EXP, short term and long-term dewatering flow rates were assessed in Section 6 and Section 7 of the EXP report. Refer to excerpts from the EXP hydrogeological investigation found in Appendix C. The short term and long-term flow rates estimated by EXP are as follows:

- Short Term Construction Dewatering Rate = 250 m^3/day = 2.9 L/s
- Long Term (Foundation Drain) Dewater Rate = 40 m^3/day = 0.46 L/s

The greater dewatering rate, short term construction dewatering rate of 2.9 L/s, has been added to Area 201A release rate and included in the below quantity control summary storage requirements.



LEGEND

201A / A=0.48 / C=0.89	CATCHMENT AREA ID / CATCHMENT AREA(ha) / RUNOFF COEFFICIENT
ST ST	EXISTING STORM SEWER
	PROPOSED STORM SEWER
	EXISTING STORM MANHOLE
	EXISTING CATCHBASIN
	PROPOSED OVERLAND FLOW ARROW
	PROPOSED DRAINAGE AREA DIVIDE
	PROPOSED LANDSCAPE AREA
	PROPOSED UNDERGROUND PARKING LIMIT
	PROPOSED STORM MANHOLE
	PROPOSED CATCHBASIN MANHOLE

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL

LICENSED PROFESSIONAL ENGINEER
C. DEL BUONO
100137127
03/26/21
PROVINCE OF ONTARIO

KWA SITE DEVELOPMENT CONSULTING INC.
2450 Lancaster Drive
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE
0 9 Metres
1:300

PROPOSED DRAINAGE PLAN
RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO
PROJECT No.20500 | DRAWN BY:BC2 | CHECKED BY:BC

PDP

As noted in the EXP report, an Environment Activity and Sector Registry (EASR) permit is required for short term dewatering. Based on EXPs long term dewatering estimate of 40 m^3/day, a Category 3 Permit to Take Water (PTTW) would not be required as the estimate it is less than the 50 m^3/day.

6.6.2 Quantity Control

To attenuate flows, a stormwater storage tank is proposed at the south east corner of the site to achieve the remaining storage volumes to adhere to the allowable release rate. A 100mm orifice tube connecting to the 750mm storm sewer on Kerns Road is proposed downstream of the storage tank to attenuate flows to the pre-development design flow for areas 202A and 202B.

Table 7: Area 202A and 202B - Summary of 100-Year Release Rate and Storage Requirements

Area ID	Area (ha)	Runoff Coefficient	t_c (min)	Storage Available (m3)	Storage Required (m3)	Release Rate (m3/s)	Description	Orifice Size (mm)	Orifice Release Rate (m3/s)
201A	0.48	1.00	10	125	124	0.046	Controlled to Kerns Road	100	0.043
201B	0.02	0.31	10		0	0.002	Uncontrolled to Kerns Road		
	0.50			125	124	0.048			
NOTES:									
5 Year Allowable Release Rate to Kerns Road = 52 L/s									
201A Release Rate = 42.5 L/s + 2.9 L/s (short term dewatering rate)									

Based on the above the peak release rate from the stormwater tank to Kerns Road is calculated to **48 L/s** for the 100-year storm event, including the 2.9 L/s short term dewatering rate, which is less than the allowable **52 L/s** pre-development 5-year release rate.

A summary of the release rates in the 100-year event for both drainage areas are shown below.

Table 8 – 100-Year Release Rates and Storage Provided

Area ID	Catchment Area (ha)	100-Year Runoff Coefficient	Drainage Direction	Allowable Release Rate (L/s)	100-year Release Rate (L/s)	Storage Provided (m3)
201A, 201B	0.50	1.00	To Kerns Road	52	48	125
202	0.13	0.48	To Upper Hager Creek	37	28	0

Refer to Appendix C for detailed Quantity Control calculations.

6.7 Post Development Quality Control

6.7.1 Chemical Analysis

Groundwater Quality and chemical analysis was analyzed in the Hydrogeological and Water Balance Investigation prepared by EXP. Refer to Section 3.4 of the EXP report. Excerpts from this section is found in Appendix D for reference. EXP concluded the following:

- "When comparing the chemistry of the collected groundwater samples to the Halton Storm, Sanitary and Combined Sewer Discharge Criteria (Table 1 and Table 2) there were no parameter exceedances to be reported."
- "For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law Limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented during construction dewatering activities to discharge to the applicable sewer system."
- "For the long-term dewatering discharge to the storm or sanitary sewer system (post-development phase) and based on the water quality test results, the water is suitable to discharge without a treatment system."

6.7.2 Low Impact Development Techniques

EXP's Hydrogeological and Water Balance Investigation incorporated groundwater water level monitoring, hydraulic conductivity testing of the subsurface soils and infiltration rate testing results at the sites storm sewer outlet location. Refer to section 3.0 of EXP's report and excerpts found in Appendix D.

One (1) infiltration rate test result was completed at the proposed storm sewer outlet location to Kerns Road. The subsurface material encountered was a Clayey Silt. EXP estimated a design infiltration rate of 5mm/hr be used based on the test result and safety factor applied. Based on the City of Burlington's Stormwater Design Guideline (Section 6.1.4.5) the minimum soil infiltration rate for LID is 15mm/hr.

Groundwater elevations were measured in January/February 2021 and ranged from 2.1m to 4.8m below existing elevations. As noted in EXP's report, seasonal fluctuations of this groundwater elevation are expected, especially during the spring months. We expect the groundwater elevation in the spring months to be higher within these areas. Generally, LID require 1m separation from the seasonal high groundwater elevation.

The proposed underground parking structure encompasses the majority of the site, approximately 0.37 ha, negating the potential for infiltrating stormwater in most areas of the Site.

Based on the information provided to date, rainwater harvesting for potential stormwater re-use may be a suitable LID measure that accommodates the Site characteristics and will be further analyzed during detailed design.

6.7.3 Quality Control

The quality control objective for the Site is to provide an enhanced level of protection which is defined as the long-term average removal of 80% of the total suspended solids (TSS). As per City of Burlington design criteria, Oil Grit Separator Units will only be credited at 50% TSS removal and a treatment train approach is recommended.

Runoff from the rooftop, landscape amenity and landscape areas are considered clean stormwater and do not require treatment. The remaining hard surface areas that will be exposed to vehicular traffic will need to be treated. The proposed hard surface areas are summarized as follows.

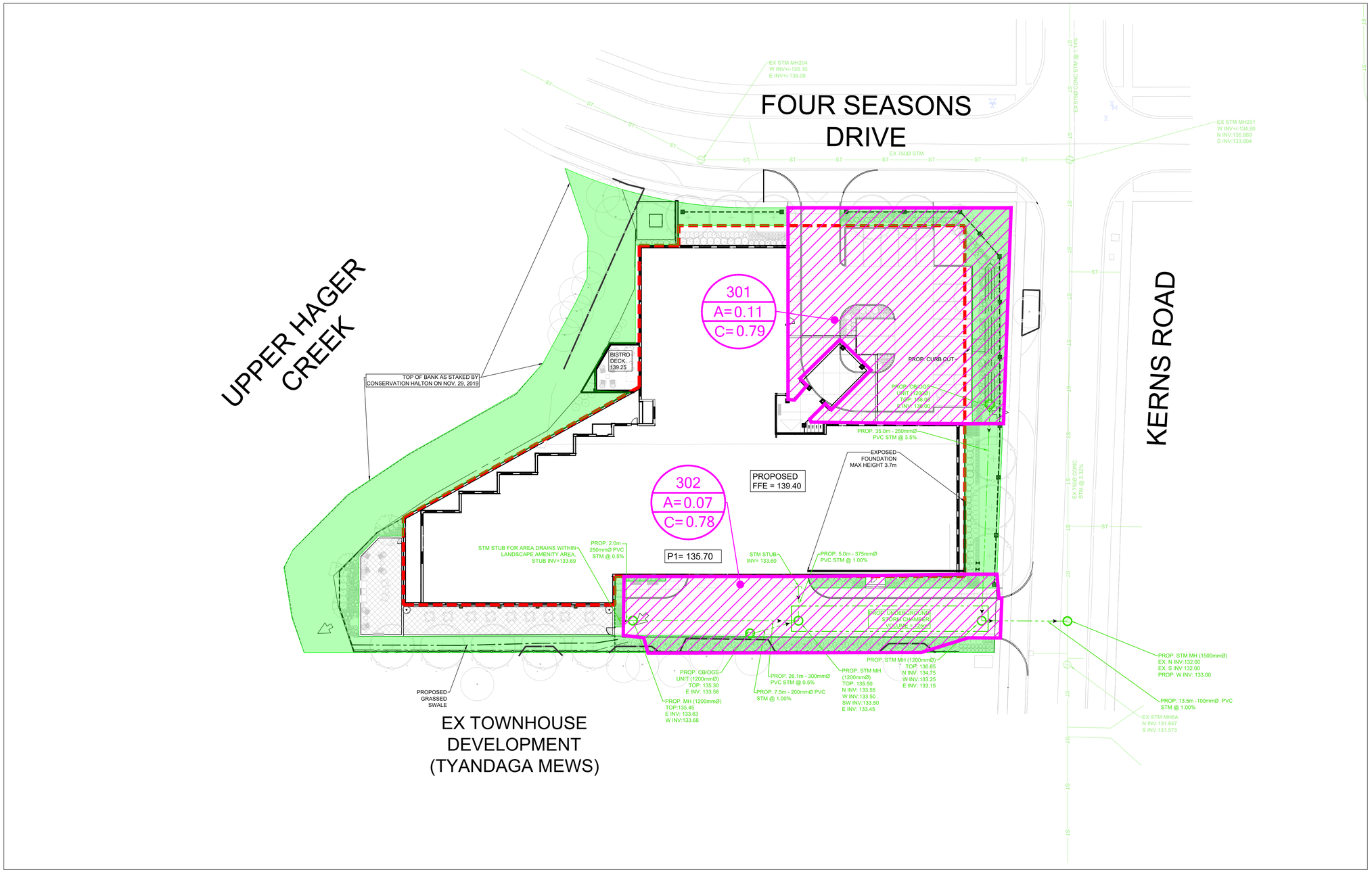
Table 9 – Hard Surface Areas Draining to Kerns Road

Area	Area ID	Catchment Area (ha)
301	North East Parking Area	0.11
302	South Driveway to Underground Parking	0.07

Refer to Proposed Quality Control Plan – PQCP.

For Area 301, the north east parking area has been graded to drain to grassed/rock swale on the east property boundary. A small OGS CB unit at the end of the grassed/rock swale will further treat runoff from this area and satisfy the treatment train recommendation.

For Area 302, the south driveway to the underground parking area will be treated by a OGS CB unit. A stormwater filtration technology, StormTech Isolator Row Plus, will form part of the underground stormwater tank. Flows from both Areas 301 and 302 will outlet to the first row of the underground storm tank. The first row of the stormwater tank, the Isolator Row, will provide additional settling and filtration of sediment as stormwater rises in the chamber. This Isolator Row and will form part of the treatment train approach for these areas. Refer to OGS sizing and Isolator Row manufacturer calculations of TSS removal found in Appendix D.



LEGEND

301 / A=0.04 / C=0.95	CATCHMENT AREA ID / CATCHMENT AREA(ha) / RUNOFF COEFFICIENT
ST — ST	EXISTING STORM SEWER
	PROPOSED STORM SEWER
	EXISTING STORM MANHOLE
	EXISTING CATCHBASIN
	PROPOSED AREA TO BE TREATED
	PROPOSED LANDSCAPE AREA
	PROPOSED UNDERGROUND PARKING LIMIMT
	PROPOSED STORM MANHOLE
	PROPOSED CATCHBASIN MANHOLE

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL

LICENSED PROFESSIONAL ENGINEER
C. DEL BUONO
100137127
03/26/21
PROVINCE OF ONTARIO

KWA SITE DEVELOPMENT CONSULTING INC.
2450 Lancaster Drive
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE
0 9 Metres
1:300

PROPOSED QUALITY CONTROL PLAN

RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO

PROJECT No:20500 | DRAWN BY:BCZ | CHECKED BY:BC

PQCP

6.8 WATER BALANCE

Typically, urbanization and re-development alters the natural hydrologic cycle due to the associated increase in impervious land coverage, thus infiltration and evaporation/transpiration are typically reduced. For this re-development, the pervious land surface area is increasing compared to its existing condition.

A detailed water balance investigation can be found in EXP's Hydrogeological and Water Balance Investigation. Refer to Section 4 and 5 in EXPs report found in Appendix E. EXP presented a pre-development and post-development water balance analysis that concluded the following:

- "Due to the increase of areas available for infiltration by approximately 970 m^2, it is expected that the unmitigated infiltration rate increases of 138 m^3/yr can be maintained under the post-development Site conditions under sound conditions."
- "Due to the increase of groundwater infiltration during post development phase of the project, compared to pre-development groundwater infiltration rate, there will be no decrease in the groundwater discharge rates at the Upper Hagger Creek."
- "Since the groundwater infiltration rate under post development conditions is increased compared to the groundwater infiltration during the pre-construction phase, additional mitigation measures may not be required."

Based on the updated pre-consultation application comments Oct. 2020, the water balance criteria noted was to retain / infiltrate the first 5mm of storm runoff onsite. As infiltration is not feasible, stormwater re-use will be proposed to achieve the water balance criteria.

As noted above, the impervious area that is contributing storm flows to the Kerns Road storm sewer is 0.48 ha (Area 201A).

$$V_{Required} = \text{Depth} \times A_{site} = 5.0 \text{ mm} \times 4{,}800 \text{ m}^2 / 1000 \text{ mm} = \mathbf{24.0\ m^3}$$

To meet the water balance objectives, a separate tank within the building or a section of the proposed stormwater tank used for quantity control could be used for rainwater harvesting. This section will re-use stormwater to irrigate the perimeter landscaping proposed around the Site.

For rainwater harvesting within the underground stormwater tank, the outlet of the tank will be set at an elevation such that a minimum of **24.0 m^3** will be available below the outlet invert for stormwater reuse.

Confirmation and frequency of stormwater re-use for irrigation will be analyzed in collaboration with the landscape and mechanical consultant during detailed design.

7.0 CONCLUSION

In summary:

Water Servicing:

- A new fire and domestic water service will connect to the existing 250mm watermain on the west side of Kerns Road.
- A 250mm fire service and 100mm domestic service, as per Region standard RH 409.010, will be brought to the buildings mechanical room located at the south east corner of the Site.
- A fire hydrant will be provided within the fire access route to ensure a hydrant is located 45m from the buildings siamese connection. The on-site hydrant will be located above the underground parking structure therefore the water service to the hydrant will be serviced internally.
- The design flows applied to the proposed watermain connections are as follows:
    - Maximum Daily Demand + Fire Flow = **194.24 L/s**
    - Maximum Hourly Demand = **2.21 L/s**
- To provide the adequate fire flows in this area, further detail regarding building material construction and collaboration with the architect, mechanical consultant, and fire suppression engineer will be required to confirm the fire pipe service size requirements and fire demands for the re-development.

Sanitary Servicing:

- Based on the post development design flow of **2.49 L/s**, a 150 mm sanitary service connected to the existing 200mm PVC sanitary sewer on Kerns Road will be required to support the proposed development.
- A new 150 mm service connection at 1.00% slope will be brought to the underground parking limit by gravity and connect to the existing sanitary sewer at elevation 133.36.
- The capacity of the proposed 150mm sanitary service at 1.00% is **15.2 L/s**. The existing capacity of the 200mm sanitary sewer at 2.53% on Kerns Road is **52.2 L/s**.
- The 2.49 L/s theoretical sanitary flow from the proposed redevelopment equates to 4.7% of the capacity of the existing 200mm sanitary sewer on Kerns Road.

Stormwater Management:

- Pre-Development Allowable Release Rates are as follows:
    - Area 101 – To Kerns Road = 52 L/s
    - Area 102 – To Upper Hager Creek = 37 L/s

Runoff Quantity Control:

- To attenuate flows, a **125 m^3** stormwater storage tank is proposed at the south east corner of the site to achieve the storage volumes required to adhere to the allowable release rate. A 100mm orifice tube connecting to the 750mm storm sewer on Kerns Road is proposed downstream of the storage tank to attenuate flows to the pre-development design flow for post development drainage areas draining to Kerns Road.
- EXP's Hydrogeological Report estimated short term and long-term dewatering rates as follows:
    - Short Term Construction Dewatering Rate = 250 m^3/day = 2.9 L/s
    - Long Term (Foundation Drain) Dewater Rate = 40 m^3/day = 0.46 L/s
    - The greater dewatering rate, short term construction dewatering rate of 2.9 L/s, has been added to the release rate draining to Kerns Road.
- For the post-development drainage area draining to the Upper Hager Creek, Area 202, the 100-year post development uncontrolled flow of 28 L/s is less than the allowable 5-year allowable release rate of 37 L/s, therefore no stormwater attenuation and storage will be required for this drainage area. It should be noted that the area draining to Upper Hager Creek is predominantly the landscape area within the 7.5m setback area. No new outlet to the Upper Hager Creek is proposed.

Post Development Quality Control:

- The quality control objective for the Site is to provide an enhanced level of protection which is defined as the long-term average removal of 80% of the total suspended solids (TSS).
- Runoff from the rooftop, landscape amenity and landscape areas are considered clean stormwater and do not require treatment.
- The remaining hard surface areas that will be exposed to vehicular traffic will be treated by the following treatment train:
    - For Area 301, the north east parking area has been graded to drain to grassed swale on the east property boundary. A small OGS CB unit at the end of the grassed swale will further treat runoff from this area and satisfy the treatment train recommendation.
    - For Area 302, the south driveway to the underground parking area will be treated by a OGS CB unit. A stormwater filtration technology, StormTech Isolator Row Plus, will form part of the underground stormwater tank. Flows from both Areas 301 and 302 will outlet to the first row of the underground storm tank. The first row of the stormwater tank, the Isolator Row, will provide additional settling and filtration of sediment as stormwater rises in the chamber. This Isolator Row and will form part of the treatment train approach for these areas.

Water Balance:

- A detailed water balance investigation is found in EXP's Hydrogeological and Water Balance Investigation. Refer to Section 4 and 5 in EXPs report found in Appendix E. EXP presented a pre-development and post-development water balance analysis that concluded the following:

- "Due to the increase of areas available for infiltration by approximately 970 m^2, it is expected that the unmitigated infiltration rate increases of 138 m^3/yr can be maintained under the post-development Site conditions under sound conditions."
- "Due to the increase of groundwater infiltration during post development phase of the project, compared to pre-development groundwater infiltration rate, there will be no decrease in the groundwater discharge rates at the Upper Hagger Creek."
- "Since the groundwater infiltration rate under post development conditions is increased compared to the groundwater infiltration during the pre-construction phase, additional mitigation measures may not be required."

- Based on the updated pre-consultation application comments Oct. 2020, the water balance criteria noted was to retain / infiltrate the first 5mm of storm runoff onsite. Total water balance volume required is **24.0 m^3.**
- To meet the water balance objectives, a separate tank within the building or a section of the proposed stormwater tank used for quantity control will be used for rainwater harvesting. This rainwater harvesting tank will re-use stormwater to irrigate the perimeter landscaping proposed around the Site.
- For rainwater harvesting within the underground stormwater tank, the outlet of the tank will be set at an elevation such that a minimum of **24.0 m^3** will be available below the outlet invert for stormwater reuse.
- Confirmation and frequency of stormwater re-use for irrigation will be analyzed in collaboration with the landscape and mechanical consultant during detailed design.

We trust that this report sufficiently addresses the site servicing requirements and that the proposed development is feasible from a municipal servicing and stormwater management perspectives.

Please do not hesitate to contact the undersigned if you have any questions or concerns.

Yours very truly,

KWA Site Development Consulting Inc.



Per:

Carlo Del Buono, P. Eng., LEED AP

carlo.delbuono@kwasitedev.com

Appendix A

FUS Calculation

RH 409.010

Water Model Schematic

Fire Underwriters Survey - Fire Demand Calculations

Project Name: 1600 Kerns Road
Project Number: 20500
Location: City of Burlington
Date: Mar-21
Prepared By: CDB

Guide for Determination of Required Flow Copyright I.S.O

Building Information					System Type							Contents Information		Exposure Charge												Unadjusted Flow (F=220C sqrt A)			Correction Factors				Required Flow	
														East			West			North			South										
Building	NBC Occupancy	Total Floor Area (m^2)	# of Storeys	Construction Class	NFPA 13 Sprinkler	Credit	Standard Water Supply	Credit	Fully Supervised	Credit	**Total Credits**	Contents Factor	**Contents Charge**	Distance (m)	L-Ht^2 (ft)	Charge	Distance (m)	L-Ht^2 (ft)	Charge	Distance (m)	L-Ht^2 (ft)	Charge	Distance (m)	L-Ht^2 (ft)	Charge	**Total Charges (Maximum 75%)**	C	A (m^2)	F (L/min)	Occupancy	FF Adjusted for Occupancy	Sprinkler Decrease	Exposure Charge	**L/min**	**L/s**
BLDG A		8468	4	NC	YES	30%	YES	10%	YES	10%	**50%**	LC	**-15%**	30	over 120	10%	90	over 120	0%	25	over 120	10%	15	over 120	15%	35%	0.8	8468	**16000**	-2400	13600	6800	35%	**11560**	**193**
																																	Total:	**193**

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

where

F = the required fire flow in litres per minute.

C = coefficient related to the type of construction.
= 1.5 for wood frame construction (structure essentially all combustible).
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior).
= 0.8 for non-combustible construction (unprotected metal structural components, masonry or metal walls).
= 0.6 for fire-resistive construction (fully protected frame, floors, roof).

A = The total floor area in square metres (including all storeys, but excluding basements at least 50 percent below grade) in the building being considered.

Legend

Construction Class

WF	wood frame construction
OC	ordinary construction
NC	non-combustible construction
FC	fire-resistive construction

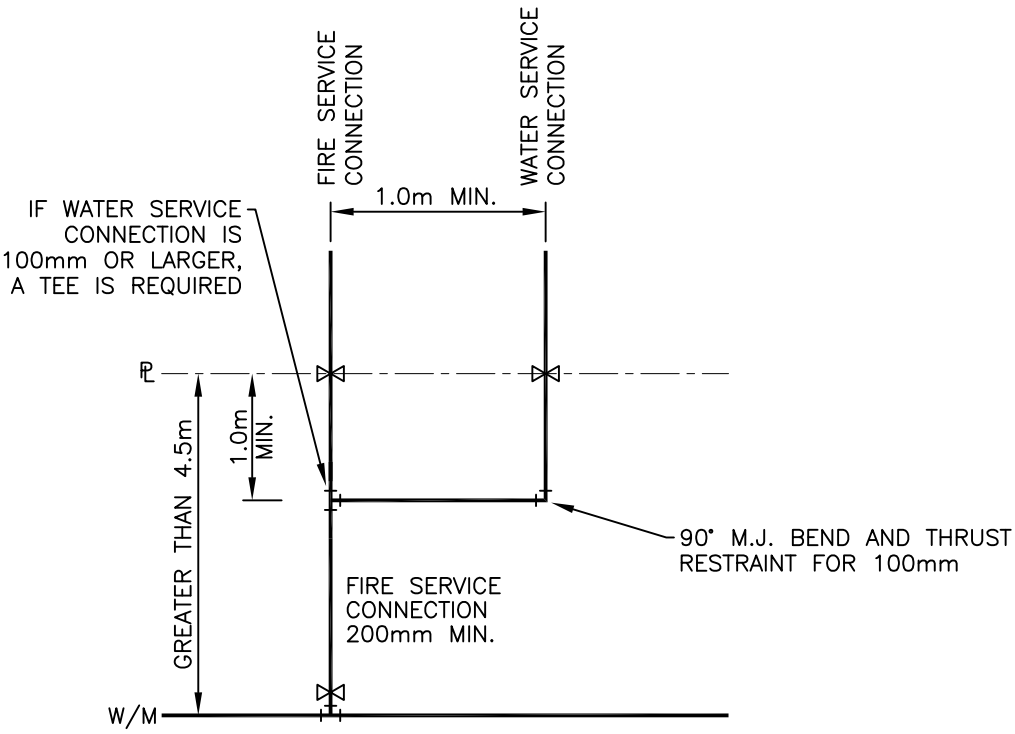
Contents Factor

NC	non-combustible
LC	limited combustible
C	combustible
FB	free burning
RB	rapid burning

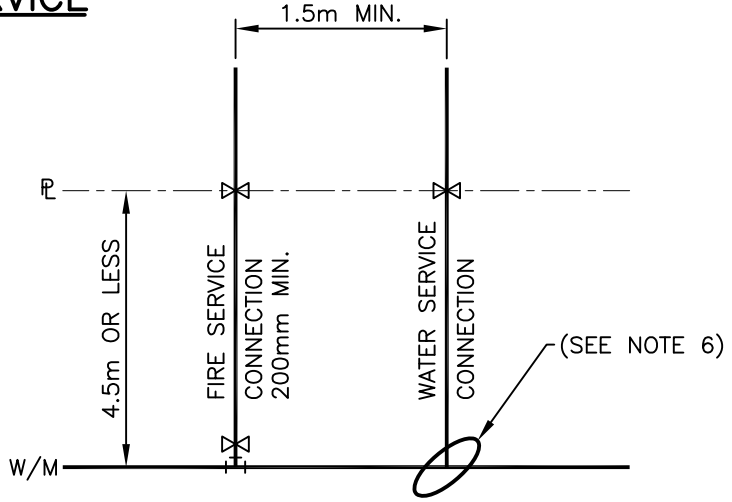
Non-Combustible	-25%	Free Burning	+15%
Limited Combustible	-15%	Rapid Burning	+25%
Combustible	No Charge		

Calculation Notes: 1 Includes all storeys, but excluding basements at least 50% below grade in the building being
2 Length - Height of Facing Wall of Exposure Building determined by multiplying the length of the facing wall of the exposure in feet by the height of the exposure in stories.
3 Maximum exposure charge = 75%
4 Round to Nearest 1,000 L/min

Separation	Charge	Separation	Charge
0 to 3m	25%	20.1 to 30 m	10%
3.1 to 10m	20%	30.1 to 45m	5%
10.1 to 20m	15%		



GREATER THAN 4.5m SERVICE



LESS THAN 4.5m SERVICE

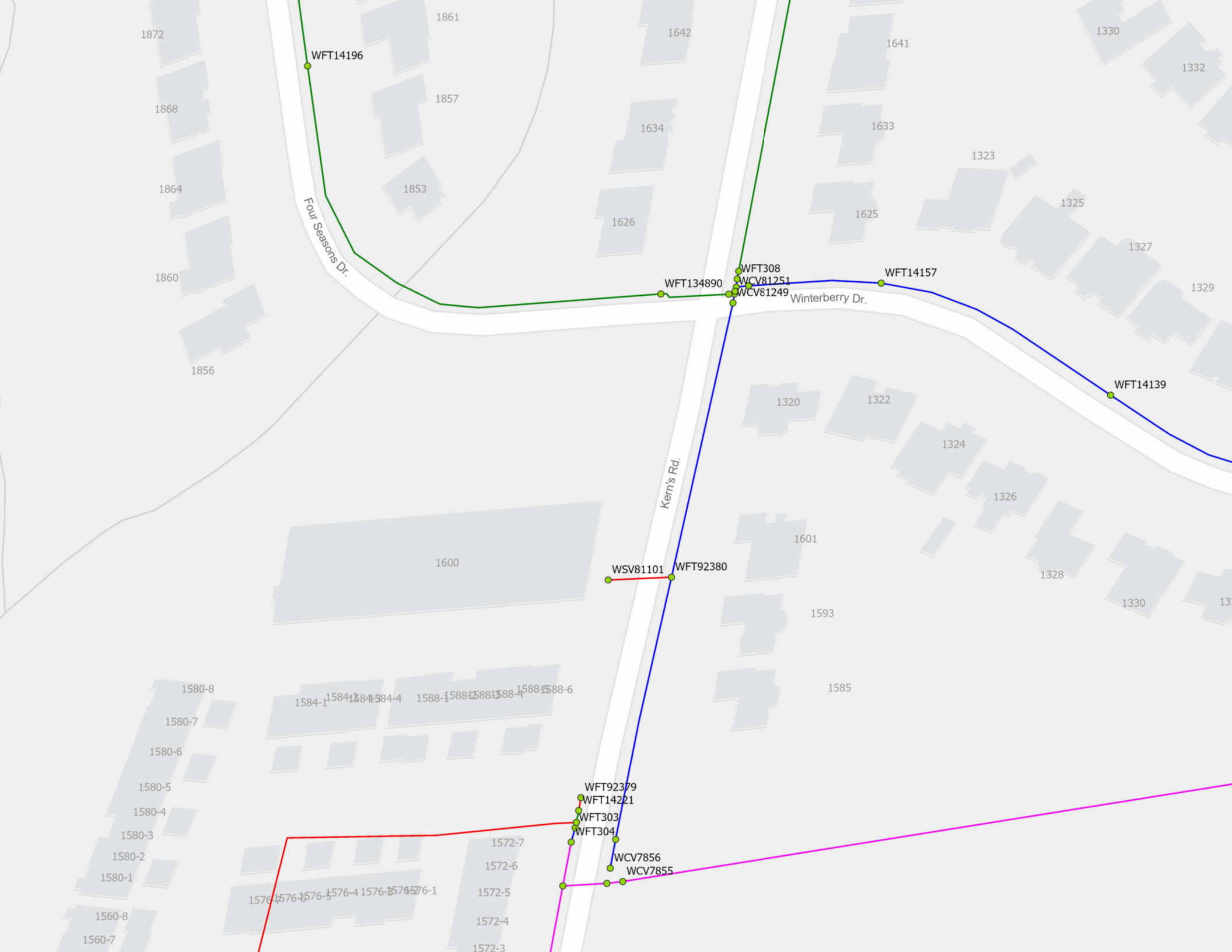
NOTES
1. COMPRESSION TYPE FITTINGS ONLY. NO SOLDERED JOINTS ARE PERMITTED BEFORE THE WATER METER.
2. WATER SERVICE CONNECTION 25, 38, 50 TO BE TYPE 'K' SOFT COPPER, 100 AND LARGER TO BE PVC OR DI.
3. FIRE SERVICE CONNECTION TO BE MIN. 200mm.
4. IF THE WATERMAIN IS 4.5m OR LESS FROM THE PROPERTY LINE, THEN 2 SEPARATE CONNECTIONS ARE REQUIRED.
5. TAPPING SLEEVE TO BE PRESSURE TESTED BY CONTRACTOR BEFORE MAIN IS TAPPED.
6. ALL SERVICE CONNECTIONS TO PVC PIPE REQUIRE SADDLES. ALL SERVICE CONNECTIONS 50mm OR LESS TO HAVE A MAIN STOP, CURB STOP AND BOX. ALL SERVICES GREATER THAN 50mm REQUIRE A TAPPING SLEEVE AND VALVE OR AN ANCHOR TEE WITH A GATE VALVE AND BOX AT THE MAIN IN ADDITION TO A PROPERTY LINE GATE VALVE AND BOX.
7. ALL GATE VALVES TO HAVE VALVE BOXES.
8. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

THE REGIONAL MUNICIPALITY OF HALTON
PUBLIC WORKS DEPARTMENT

Date: August 2018	Rev. 2	NTS

WATER SERVICE AND FIRE SERVICE CONNECTION INSTALLATIONS

REGION STANDARD RH 409.010


WFT14196
WFT134890
WFT308
WCV81251
WCV81249
WFT14157
WFT14139
Four Seasons Dr.
Winterberry Dr.
Kern's Rd.
WSV81101
WFT92380
WFT92379
WFT14221
WFT303
WFT304
WCV7856
WCV7855
1872
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1585
1580-8
1580-7
1580-6
1580-5
1580-4
1580-3
1580-2
1580-1
1572-7
1572-6
1572-5
1572-4
1572-3
1560-8
1560-7

Appendix B

Sanitary Design Calculations

Existing Sanitary Demand

Project Name:	1600 Kerns Road	
Project Number:	20500	
Location:	City of Burlington	
Date:	Mar-21	
Prepared By:	CDB	
Average Daily Domestic Flow	275	litres / capita / day
Land Area	0.63	ha
Population		
Total Population		persons
or		
Equivalent Population Density	90	ppl / ha
Resulting Population:	56.7	persons
Peaking Factor	Harmons Equation	
Constant:	5	
Harmon's Equation:	4.3	
Min. Harmon's Value (typ. 2.0)	2.0	
Peak Dry Weather Flow	0.77	Litres / second
Extraneous Flows	0.18018	Litres / second
Extraneous Flow Rate	0.286	Litres / second / ha
Peak Flow (incl. extraneous)	0.95	Litres / second

Proposed Sanitary Demand

Project Name:	1600 Kerns Road	
Project Number:	20500	
Location:	City of Burlington	
Date:	Mar-21	
Prepared By:	CDB	
Average Daily Domestic Flow	275	litres / capita / day
Land Area	0.63	ha
Population		
Total Population	174	persons
or		
Equivalent Population Density		ppl / ha
Resulting Population:		persons
Peaking Factor	Harmons Equation	
Constant:	5	
Harmon's Equation:	4.2	
Min. Harmon's Value (typ. 2.0)	2.0	
Peak Dry Weather Flow	2.31	Litres / second
Extraneous Flows	0.18018	Litres / second
Extraneous Flow Rate	0.286	Litres / second / ha
Peak Flow (incl. extraneous)	2.49	Litres / second

Appendix C

Stormwater Calculations

Rational Method

Project No: 20500
Project Name: Fieldgate Kerns Road - Burlington

Pre-Development Release Rates

Area 101 - To Kerns Road

Storm Event:		5	years
ABC's:	a	802.04	
	b	5	
	c	0.764	
Time of Concentration:	t	10	min
Runoff Coefficient:	C	0.90	
Site Area	A	0.37	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	101.31	mm/hr
Flow [Q=CiA/360]	Q	0.09	m^3/s
		94	l/s

Area 102 - To Upper Hager Creek

Storm Event:		5	years
ABC's:	a	802.04	
	b	5	
	c	0.764	
Time of Concentration:	t	10	min
Runoff Coefficient:	C	0.72	
Site Area	A	0.26	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	101.31	mm/hr
Flow [Q=CiA/360]	Q	0.05	m^3/s
		53	l/s

AREA ID 101

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.03	0.25	0.0063
Hard Surface	0.35	0.95	0.3278
	0.37	Total	0.3340
		Divided by Total Area =	0.90

AREA ID 102

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.08	0.25	0.0210
Hard Surface	0.18	0.95	0.1672
	0.26	Total	0.1882
		Divided by Total Area =	0.72

Rational Method

Project No: 20500
Project Name: Fieldgate Kerns Road - Burlington

Pre-Development Release Rates

Area 101 - To Kerns Road

Storm Event:		5	years
ABC's:	a	802.04	
	b	5	
	c	0.764	
Time of Concentration:	t	10	min
Runoff Coefficient:	C	0.50	
Site Area	A	0.37	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	101.31	mm/hr
Flow [Q=CiA/360]	Q	0.05	m^3/s
		52	l/s

Area 102 - To Upper Hager Creek

Storm Event:		5	years
ABC's:	a	802.04	
	b	5	
	c	0.764	
Time of Concentration:	t	10	min
Runoff Coefficient:	C	0.50	
Site Area	A	0.26	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	101.31	mm/hr
Flow [Q=CiA/360]	Q	0.04	m^3/s
		37	l/s

AREA ID 101

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.03	0.25	0.0063
Hard Surface	0.35	0.95	0.3278
	0.37	Total	0.3340
	Divided by	Total Area =	0.90

AREA ID 102

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.08	0.25	0.0210
Hard Surface	0.18	0.95	0.1672
	0.26	Total	0.1882
	Divided by	Total Area =	0.72

Rational Method

Project No: 20500
Project Name: Fieldgate Kerns Road - Burlington

Post -Development Release Rates

Area 201A & 201B - To Kerns Road

Storm Event:		100	years
ABC's:	a	1281.34	
	b	5	
	c	0.761	
Time of Concentration:	t	10	min
Runoff Coefficient:	C	1.00	
Site Area	A	0.52	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	163.18	mm/hr
Flow [Q=CiA/360]	Q	0.24	m^3/s
		236	l/s

Area 202 - To Upper Hager Creek

Storm Event:		100	years
ABC's:	a	1281.34	
	b	5	
	c	0.761	
Time of Concentration:	t	10	min
Runoff Coefficient:	C	0.35	
Site Area	A	0.11	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	163.18	mm/hr
Flow [Q=CiA/360]	Q	0.02	m^3/s
		17	l/s

Modified Rational Calculation

Project Name: Fieldgate Kerns Road
Project Number: 20500

Rainfall Data			
Location:	City of Burlington	a	1281.34
Event	100	b	5
		c	0.761

Area ID	Area (ha)	Runoff Coefficient	t_c (min)	Storage Available (m3)	Storage Required (m3)	Release Rate (m3/s)	Description	Orifice Size (mm)	Orifice Release Rate (m3/s)
201A	0.48	1.00	10	125	124	0.046	Controlled to Kerns Road	100	0.043
201B	0.02	0.31	10		0	0.002	Uncontrolled to Kerns Road		
	0.50			125	124	0.048			

NOTES:
5 Year Allowable Release Rate to Kerns Road = 52 L/s
201A Release Rate = 42.5 L/s + 2.9 L/s (short term dewatering rate)

Modified Rational **Area:** **201A**

Project Name: Fieldgate Kerns Road
Project Number: 20500

Rainfall Data			
Location:	City of Burlington	a	1281.340
Event	100	b	5.000
		c	0.761

Site Data		
Area	0.48	ha
Runoff Coefficient	1.00	
AC	0.48	
Tc	10	
Time Increment	5	
Release Rate	46	l/s
Storage Required	124	m^3

Time (min)	Rainfall Intensity (mm/hr)	Storm Runoff (m3/s)	Runoff Volume (m3)	Released Volume (m3)	Storage Volume (m3)
10	163	0.22	131	27	103
15	131	0.17	157	41	116
20	111	0.15	177	55	122
25	96	0.13	193	68	124
30	86	0.11	206	82	124
35	77	0.10	217	96	121
40	71	0.09	226	109	117
45	65	0.09	235	123	112
50	61	0.08	243	137	106
55	57	0.08	250	150	100
60	53	0.07	257	164	93
65	51	0.07	263	178	85
70	48	0.06	269	191	77
75	46	0.06	274	205	69
80	44	0.06	279	219	60
85	42	0.06	284	232	52
90	40	0.05	289	246	43
95	39	0.05	293	260	33
100	37	0.05	297	273	24
105	36	0.05	301	287	14
110	35	0.05	305	301	4

Modified Rational **Area:** **201B**

Project Name: Fieldgate Kerns Road
Project Number: 20500

Rainfall Data			
Location:	City of Burlington	a	1281.340
Event	100	b	5.000
		c	0.761

Site Data		
Area	0.02	ha
Runoff Coefficient	0.31	
AC	0.005	
Tc	10	
Time Increment	5	
Release Rate	2	l/s
Storage Required	0	m^3

Time (min)	Rainfall Intensity (mm/hr)	Storm Runoff (m3/s)	Runoff Volume (m3)	Released Volume (m3)	Storage Volume (m3)
10	163	0.00	1	1	0
15	131	0.00	2	2	0
20	111	0.00	2	2	-1
25	96	0.00	2	3	-1
30	86	0.00	2	4	-2
35	77	0.00	2	4	-2
40	71	0.00	2	5	-3
45	65	0.00	2	5	-3
50	61	0.00	2	6	-4
55	57	0.00	2	7	-4
60	53	0.00	3	7	-5
65	51	0.00	3	8	-5
70	48	0.00	3	8	-6
75	46	0.00	3	9	-6
80	44	0.00	3	10	-7
85	42	0.00	3	10	-7
90	40	0.00	3	11	-8
95	39	0.00	3	11	-9
100	37	0.00	3	12	-9
105	36	0.00	3	13	-10
110	35	0.00	3	13	-10

Orifice Control &
Detention Storage

Job Fieldgate Kerns Road
Job No. 20500

Area: **201A**

Orifice Diameter: 100 mm

Orifice Equation: $Q = C_d A(2gh)^{1/2}$

Area = 0.01 m^2

g = 9.81 m/s^2

C_d = 0.8

	Stage	Head (m)	Storage (m3)	Discharge (m3/s)
Invert E.L.	133.15	0.00	0	0.00
Ground E.L.	135.30	2.10	0.0	0.040
100 year Ponding E.L.	135.55	2.35	125.0	0.043

Appendix C

Excerpts from EXP Hydrogeolgical and Water Balance

6 Construction Dewatering Assessment

It is our understanding that the proposed development plan is to construct a proposed three (3) to four (4) storey residential condominium development with a single basement level (P1). The Site location plan is shown on Figure 1.

As available information, EXP understands a caisson wall system will be installed covering the northern portion of the excavation.

Table 6-1 presents the assumptions used to calculate the dewatering rate for the Site.

Table 6-1 Dewatering Estimate Assumptions

Input Parameter	Assumption	Units	Notes
Ground Surface Elevation	139.0	masl	Approximate elevation based on the borehole logs and Site
Groundwater elevation	137.3	masl	The highest recorded groundwater elevation measured across the Site plus 1 meter to account for some seasonal fluctuation
Top of Slab Elevation	135.5	masl	Based on Concept Architectural Drawings (November 26, 2020)
Lowest Footing Elevation	134.0	masl	Assumed to be approximately 1.5 m below the top of slab elevation
Dewatering Target Elevation	133.0	masl	Assumed to be approximately 1.0 m below the lowest footing elevation
Bottom Elevation of Water-Bearing Zone	130.0	masl	Assumed
Excavation Area (Length x Width)	2,500 (50 x 50)	m^2 (m x m)	Approximate area (length x width) of Site for the proposed development (simplified)
Hydraulic Conductivity (K)	1.1E-05	m/s	Weighted Average of K values for sand and till formations

6.1 Dewatering Flow Rate Estimate and Zone of Influence

The Dupuit-Forcheimer equation for radial flow to both sides of an excavation through an unconfined aquifer resting on a horizontal impervious surface was used to obtain a flow rate estimate. Dewatering flow rate is expressed as follows:

$$Q_w = \frac{\pi K (H^2 - h^2)}{Ln\left[\frac{R_o}{r_e}\right]}$$

$$r_e = \frac{a+b}{\pi}$$

$$R_o = R_{cj} + r_e$$

Where:

Qw = Rate of pumping (m3/sec)
X = Length of excavation (m)
K = Hydraulic conductivity (m/sec)
H = Hydraulic head beyond the influence of pumping (static groundwater elevation) (m)
h = Hydraulic head above the base of aquifer in an excavation (m)
R0 = Radius of Influence (m)
Rcj = Cooper Jacob Radius of Influence (m)
re = Equivalent perimeter (m)
a = Length of the excavation area (m)
b = Width of the excavation area (m)

It is expected that the initial dewatering rate will be higher to remove groundwater from within the overburden formation. The dewatering rates are expected to decrease once the target water level is achieved in the excavation footprint as groundwater will have been removed, primarily from storage, resulting in lower seepage rates into the excavation.

6.2 Cooper-Jacob's Radius of Influence

The radius of influence (Rcj) for the construction dewatering was calculated based on Cooper-Jacob equation. This equation is used to predict the distance at which the drawdown resulting from pumping is negligible.

The estimated radius of influence due to pumping is based on Cooper-Jacob formula as follows:

$$\mathrm{R}_{cj} = \sqrt{2.25KDt/s}$$

Where:

Ro = Estimated radius of influence (m)
D = Aquifer thickness (original saturated thickness) (m)
K = Hydraulic conductivity (m/sec)
S = Storage coefficient
t = Duration of pumping (s)

6.3 Stormwater

Additional pumping capacity may be required to maintain dry conditions within the excavation during and following significant precipitation events. Therefore, the dewatering rates at the Site should also include removing stormwater from the excavation.

A 15 mm precipitation event was utilized for estimating the stormwater volume. The calculation of the stormwater volume is included in Appendix G.

The estimate of the stormwater volume only accounts for direct precipitation into the excavation. The dimensions of the excavation are considered in the dewatering calculations. Runoff from outside of the excavation's footprint is excluded and it should be directed away from the excavation.

During precipitation events greater than 15 mm (ex: 100-year storm), measures should be taken by the contractor to retain stormwater onsite in a safe manner to not exceed the allowable water taking and discharge limits, as necessary. A two (2) and a one hundred (100) year storm event over a 24-hour period are 56.2 and 122.1 mm, respectively, which would produce 449.6 and 976.8 m^3 of water.

6.4 Results of Construction Dewatering Rate Estimate

For this assessment, it was assumed that the proposed construction plans include an excavation with shoring system covers the northern portion of the excavation. EXP should be retained to review the assumptions outlined in this section, should the assumed shoring design change.

Short-term (construction) dewatering calculations are presented in Appendix G.

Based on the assumptions provided in this report, the results of the dewatering rate estimate can be summarized as follows:

Table 6.2 Summary of Construction Dewatering Rate Estimates.

Peak Dewatering Flow Rate Including Rain Collection Volume and Zone of Influence	
Description	With 1 Level of Underground Parking (m^3/day)
Estimated Short Term Dewatering Rate (without safety factor)	140
From Precipitation Event of 15 mm in one day	40
Without Factor of Safety of 1.5 (including Precipitation) for designs, budgeting and permits	180
With Factor of Safety of 1.5 (including Precipitation) for designs, budgeting and permits	250
Radius of Influence (m) for the area without caisson wall	40

This peak dewatering flow rates does not account for flow from utility beddings and variations in hydrogeological properties beyond those encountered during this investigation.

It is expected that, during the construction dewatering activities, because of the proposed caisson wall system, the radius of influence in the northern portion of excavation is negligible (Appendix G).

Local dewatering may be required for pits (elevator pits, sump pits), if these extend deeper than the dewatering target. Local dewatering is not considered to be part of this assessment. Dewatering estimates should be reviewed once the pit dimensions are available.

All grading around the perimeter of the Site should be graded away from the shoring the systems. The dewatering assumptions are based on using shoring system with open cuts.

The maximum flow rate calculated with a high K-value, provides a conservative estimate to account for higher than expected flow rates during the construction dewatering.

If caisson walls are installed, these should be designed for full hydrostatic pressure for shallow and deep water levels, without dewatering on the outside. Soldier pile and lagging and caisson wall systems should be designed to account for shallow groundwater conditions and take into consideration that dewatering systems may not provide fully dewatered conditions.

If caisson walls are used for decreasing long-term dewatering rates, these should be designed as permanent structures to cutoff groundwater inflow in the long-term. All perforations should be sealed permanently (ex: tiebacks, breaches and cold joints) with no leakages and inspected. Fillers should extend into low permeability deposits (ex: sound bedrock or till) to cutoff groundwater from water bearing zones. Inspections should be conducted to confirm the depth of low permeability deposits along shoring system and that fillers are keyed into low permeability soil deposits.

All grading around the perimeter of the construction Site should be graded away from the shoring the system.

The contractor is responsible to ensure that dry conditions are always maintained within the excavation at all costs.

6.5 Construction MECP Water Taking Permit

In accordance with the Ontario Water Resources Act, if the water taking for the construction dewatering is more than 50 m^3/day but less than 400 m^3/day, then an online registration in the Environmental Activity and Sector Registry (EASR) with the MECP will be required. If groundwater dewatering rates onsite exceed 400 m^3/day, a Category 3 Permit to Take Water (PTTW) will be required from the MECP.

It is recognized that the maximum flow estimate calculated with a high K-value, provides a conservative estimate to account for higher than expected flow rates during the construction dewatering. Based on the dewatering estimate of approximately 390 m^3/day for this project, an EASR will be required to facilitate the construction dewatering program of the Site.

A Discharge Plan (dewatering sketch, sewer discharge agreement) must be developed and applied for any discharges from the Site. Monitoring of both water quantity and water quality must be carried out for the entire duration of the construction dewatering phase. During this phase the Discharge Plan and the daily water taking records must be available onsite.

The EASR, Discharge Plan, hydrogeological investigation report, and geotechnical assessment of settlements must also be available at the construction Site during the entire construction dewatering. EXP should be notified immediately about any changes to the construction dewatering schedule or design, since the EASR will need to be updated to reflect these modifications. The hydrogeological report, EASR, Discharge Plan and geotechnical assessment constitutes the Water Taking Plan which needs to be available onsite during the construction dewatering.

7 Sub-Drain Discharge Estimate

7.1 Long-Term Dewatering Rate Assessment

It is our understanding that the development plan includes a permanent foundation sub-drain system that will ultimately discharge to the municipal sewer system if conventional footings are installed.

To estimate the groundwater flow to the future sub-drain, the Dupuit-Forcheimer equation for steady-state radial flow to the entire excavation through an unconfined aquifer resting on a horizontal impervious surface was used (Section 4.1).

The calculation for the estimated flow to the future sub-drain system is provided in Appendix H. The dewatering target for the foundation drainage system is taken at 0.5 m below the lowest slab elevation.

The foundation drain analysis provides a flow rate estimate. Actual flow rate measurements of the sump discharge, once it is built, will be required to confirm these flow rates.

7.2 Sub-Drain Flow Rate Estimate

Based on the assumptions provided in this report, the estimated sub-drain discharge volumes are summarized in Appendix H. Seasonal and daily fluctuations are expected. These estimates may be affected by hydrogeological conditions beyond those encountered at this time, fluctuations in groundwater regimes, surrounding Site alterations, and existing and future infrastructures.

The long-term dewatering rates was estimated based on the installation of caisson wall along the north portion of the excavation (Appendix G).

Table 7.1: Summary of Long-Term Dewatering Rate

Long-Term Dewatering Flow Rate for P1 (m^3/day)	
Groundwater Model Estimated Long-Term Dewatering Rate	25
Long-Term Dewatering Rate with Safety Factor of 1.5 for design, budgeting and permitting	40

For the design of water foundations without perimeter and foundation drainage systems, shallower wells need to be considered to evaluate the shallow groundwater table. The hydrogeologist needs to be consulted during the design process.

Intermittent cycling of sump pumps and seasonal fluctuation in groundwater regimes should be considered for pump specifications. A safety factor was applied to the flow rate to account for water level fluctuations due to seasonal changes.

These estimates assume that pits (elevator and/or sump pits) are made as watertight structures (without drainage), if their depths extend below the dewatering target, as previously stated. The dewatering assumptions are based on without shoring system with open cuts. Open cuts can act as preferential groundwater pathways in the long-term and cause foundation drainage volumes to increase.

If caisson walls are used for decreasing long-term dewatering rates, these should be designed as permanent structures to allow for groundwater cutoff in the long-term. All perforations should be sealed permanently (ex: tiebacks, breaches and cold joints) with no leakages and inspected. Fillers should extend into low permeability deposits (ex: sound bedrock or till) to cutoff

groundwater from water bearing zones. Inspections should be conducted to confirm the depth of low permeability deposits along shoring system and that the fillers are keyed into low permeability soil deposits.

The sub-drain rate estimate is based on the assumptions outlined in this report. Any variations in hydrogeological conditions beyond those encountered as part of this investigation may significantly influence the sub-drain discharge volumes.

Once the sub-drain system is in place, it is recommended to install a flow meter at the sump(s) to record daily discharge volumes during the commissioning stage of the system.

7.3 Post-Development MECP Water Taking Permit

In accordance with the Ontario Water Resources Act, if the water taking for the construction dewatering is more than 50 m^3/day, then an application for a Category 3 Permit to Take Water (PTTW) will be required from the MECP.

It is recognized that the maximum flow rate calculated with a high K-value, provides a conservative estimate to account for higher than expected flow rates during the post-development dewatering. Based on the dewatering estimate of approximately 40 m^3/day for this project, a Category 3 Permit to Take Water (PTTW) would not be required to facilitate the post-development phase.

Appendix D

Stormwater Quality Control

Excerpts from EXP Hydrogeolgical and Water Balance

3 Results

3.1 Monitoring Well Details

The monitoring well network was installed as part of the Hydrogeological, Geotechnical and Environmental Investigation at the Site. It consists of the following:

- One (1) overburden monitoring well (BH/MW 1001) was installed by EXP in January 2021;
- Three (3) overburden monitoring wells (BH 19-01, BH 19-02, BH19-04) were installed by Watters Environmental Group Inc. in 2020; and,
- Two (2) overburden monitoring wells (BH 5 and BH 6) were installed by EXP in 2018.

The diameter of all monitoring wells is 50 mm. All wells were installed with a flush mount protective casing. Borehole logs and monitoring well installation details are provided in Appendix B. The monitoring well locations are shown on Figure 4.

3.2 Water Level Monitoring

As part of the Hydrogeological and Water Balance Investigation, static water levels in the monitoring wells installed outside of the existing building were recorded in three (3) monitoring events, including January 26, February 5 and February 11, 2021. A summary of all static water level data as it relates to the elevation survey is given in Table 3-1 below.

The groundwater elevation recorded in the wells ranged from 129.68 masl (7.90 mbgs at BH 6 on January 26, 2021) to 136.32 masl (2.43 mbgs at BH19-04 on January 26, 2021).

Table 3-1: Summary of Measured Groundwater Elevations

Monitoring Well ID	Ground Surface Elevation (masl)	Approximate Full Well Depth (mbgs)	Unit	26-Jan-21	5-Feb-21	11-Feb-21
EXP - 2021						
BH/MW 1001	136.47	7.60	mbgs	Dry	2.14	2.47
			masl	-	134.33	134.01
EXP - 2018						
BH 5	138.16	9.14	mbgs	2.84	2.93	3.04
			masl	135.32	135.23	135.13
BH 6	137.58	7.90	mbgs	7.90	3.77	3.81
			masl	129.68	133.81	133.78
Watters Environmental Group Inc. - 2020						
BH/MW 19-01	138.57	6.05	mbgs	4.73	4.75	4.79
			masl	133.85	133.82	133.79
BH/MW 19-02	137.31	8.20	mbgs	3.36	3.57	3.86

Monitoring Well ID	Ground Surface Elevation (masl)	Approximate Full Well Depth (mbgs)	Unit	26-Jan-21	5-Feb-21	11-Feb-21
			masl	133.95	133.75	133.45
BH/MW 19-04	138.74	9.70	mbgs	2.43	2.78	3.18
			masl	136.32	135.97	135.57

Notes:
mbgs - meters below ground surface
masl - meters above mean sea level

A map was created for the Site to show groundwater contours for the overburden (Figures 6). Accordingly, the groundwater flow direction in the overburden is interpreted to be towards southwest of the Site, towards Upper Hager Creek.

Groundwater levels are expected to show seasonal fluctuations and vary in response to prevailing climate conditions. This may also affect the direction and rate of flow. It is recommended to conduct seasonal groundwater level measurements to provide more information on seasonal groundwater level fluctuations.

3.3 Hydraulic Conductivity Testing

Six (6) Single Well Response Tests (SWRT's) were completed on monitoring wells BH 5, BH 6, BH 19-01, BH 19-02 and BH19-04 on January 26, 2021 and on monitoring wells BH/MW 1001, BH 5 and BH19-01 on February 4, 2021. The tests were completed to estimate the saturated hydraulic conductivity (K) of the soils at the well screen depths.

The static water level within each monitoring well was measured prior to the start of testing. In advance of performing SWRTs, each monitoring well underwent development to remove fines introduced into the screens following installation. The development process involved purging of the monitoring wells to induce the flow of fresh formation water through the screen. Each monitoring well was permitted to fully recover prior to performing SWRTs.

Hydraulic conductivity values were calculated from the SWRT and constant rate test data as per Hvorslev's solution included in the Aqtesolv Pro. V.4.5 software package. The semi-log plots for normalized drawdown versus time are included in Appendix C.

A summary of the hydraulic conductivities (K-values) estimated from the SWRTs are provided in Table 3-2.

Table 3-2: Summary of Hydraulic Conductivity Testing

Monitoring Well	Monitoring Well Depth (mbgs)	Screen Interval (mbgs)		Formation Screened	Estimated Hydraulic Conductivity (m/s)
		from	to		
BH/MW 1001	7.60	4.60	7.60	Clayey Silt Till	8.7E-09
BH 5	9.14	6.14	9.14	Clayey Silt Till	1.1E-08
BH 6	7.90	4.90	7.90	Clayey Silt Fill / Clayey Silt /Sand and Gravel Layers (Creek bed deposits)	6.6E-05
BH 19-01	6.05	3.05	6.05	Clayey Silt Fill with Sand	3.1E-05
BH 19-02	8.20	5.20	8.20	Clayey Silt	2.7E-08
BH 19-04	9.70	6.70	9.70	Clayey Silt	1.5E-08
				Highest Estimated K Value	6.6E-05
				Geometric Mean of Estimated K Values	2.1E-07
				Weighted Average of K Values for fill and sand	1.1E-05

SWRTs provide K-estimates of the geological formation surrounding the well screens and may not be representative of bulk formation hydraulic conductivity. As shown in Table 3-2, the highest K-value of the tested water-bearing zone is 6.6E-5 m/s, and the geometric mean of the K-values is 2.1E-7 m/s.

Since the sandy formation was encountered in the western portion of the Site, the weighted average of sandy formation and till formations, 1.1E-05 m/s was used for dewatering rate estimates.

3.4 Groundwater Quality

To assess the suitability for discharging pumped groundwater into the sewers owned by the City of Burlington / Regional Municipality of Halton during dewatering activities, one (1) groundwater sample was collected from monitoring well BH 19-02 on February 5, 2021 using a peristaltic pump.

Prior to collecting the noted water sample, approximately three (3) standing well volumes of groundwater were purged from the referred well. The samples were collected unfiltered and placed into pre-cleaned laboratory-supplied vials and/or bottles provided with analytical test group specific preservatives, as required. Dedicated nitrile gloves were used during sample handling. The groundwater samples were submitted for analysis to Bureau Veritas Laboratory, a CALA certified independent laboratory in Mississauga, Ontario. Analytical results are provided in Appendix D.

When comparing the chemistry of the collected groundwater samples to the Regional Municipality of Halton Storm, Sanitary and Combined Sewer Discharge Criteria (Table 1 and Table 2), there were no parameter exceedances to be reported.

Reporting detection limits (RDLs) were below the Sewer Use By-Law parameter criteria of Tables 1 and 2.

Phase Two Environmental Site Assessment (2018) by EXP was reviewed for exceedances in groundwater samples taken from the Site. Two (2) groundwater samples from two (2) monitoring wells (BH5 and BH6) were collected for groundwater analysis. When comparing the chemistry of the collected groundwater samples to the Halton Storm, Sanitary and Combined Sewer Discharge Criteria (Table 1 and Table 2), there were no parameter exceedances to be reported. Analytical results are provided in Appendix D.

For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented (filtration or decantation facilities and/ or any other applicable treatment system) during construction dewatering activities to discharge to the applicable sewer system.

For the long-term dewatering discharge to the City of Burlington / Regional Municipality of Halton storm, sanitary and combined sewer system (post-development phase) and based on the water quality test results, the water is suitable to be discharged without a treatment system.

For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, the water is suitable to be discharged without a treatment system. However, additional water quality testing may be required depending on the receiving water body.

The water quality results presented in this report may not be representative of the long-term condition of groundwater quality onsite. As such, regular water quality monitoring is recommended for the post-construction phase, as required by the City.

An agreement to discharge into the sewers owned by the City of Burlington / Regional Municipality of Halton will be required prior to releasing dewatering effluent.

3.5 Infiltration Rate Testing Results

EXP completed one (1) infiltration rate tests (IT 1) at the proposed location for the infiltration facility, within the Site area on January 20, 2021. This test was conducted in proximity of selected monitoring well BH/MW 1001.

Infiltration test (IT) were conducted in boreholes at depths ranging from approximately 3.5 mbgs, depending on the measured groundwater elevation at the testing location.

The stratigraphy encountered in the shallow subsurface is comprised of a sandy silt till matrix. Table 3.3 shows the summary of field saturated hydraulic conductivity (K) testing and design infiltration rates as per the Low Impact Development (LID) Stormwater Management Planning and Design Guide, CVC – TRCA, 2010, Appendix G. The estimated field saturated hydraulic conductivities were correlated to infiltration rates based on the approximate relationship provided in Appendix D of the guideline.

Infiltration rate testing locations are shown on Figure 4 and infiltration rate analysis is provided in Appendix E.

Table 3.3: Summary of Infiltration Testing Results

Infiltration Test Location/ MW ID	Depth of Hole (mbgs)	Formation tested	Field Saturated Hydraulic Conductivity, K_{fs} (m/s)	Infiltration Rates (mm/hr)
INF 1	3.2	Clayey Silt	9.4×10^{-8}	25
		Design Infiltration Rate*		**5**

Notes:
*Safety Factor of 4.5 was applied to calculate the design infiltration rate (Low Impact Development (LID) Stormwater Management Planning and Design Guide, CVC – TRCA, 2010, Appendix D).

The estimated design infiltration rate based on percolation rate testing for the Site is 5 mm/hr (Appendix E).

ADS OGS Sizing Summary

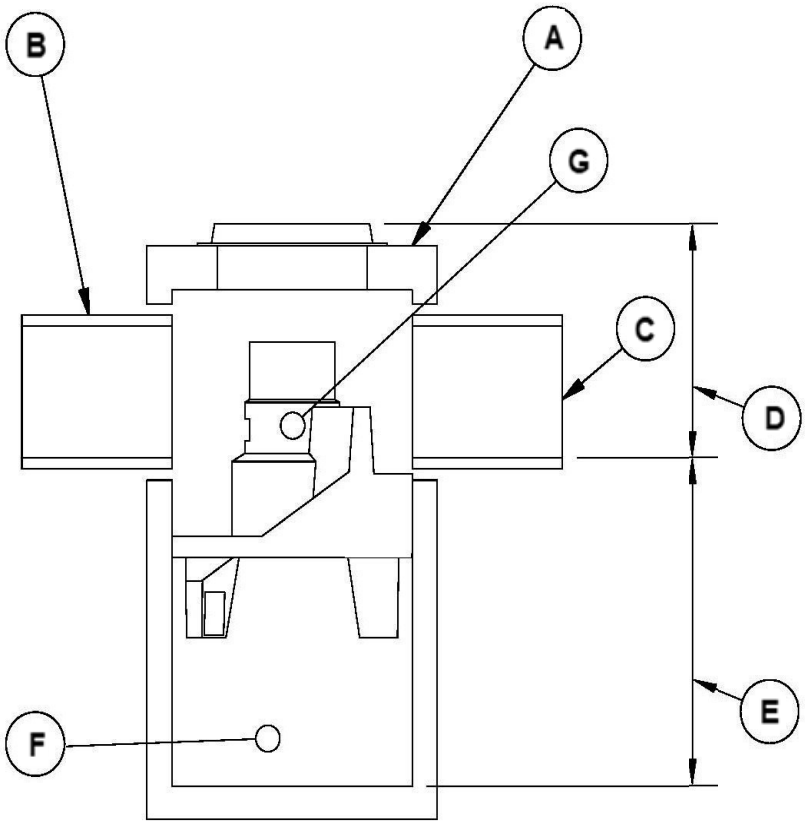
Project Name:	1600 Kerns Road - North East Parking Area
Consulting Engineer:	KWA Site Development Consulting Inc.
Location:	Burlington, Ontario
Sizing Completed By:	Haider Nasrullah
Email:	haider.nasrullah@ads-pipe.com

Treatment Requirements		
Treatment Goal:	Enhanced (MOE)	
Selected Parameters:	80% TSS	90% Volume
Selected Unit:	**FD-4HC**	

Site Details	
Site Area:	0.11 ha
% Impervious:	100%
Rational C:	0.90
Rainfall Station:	Hamilton, ONT
Particle Size Distribution:	Fine
Peak Flowrate:	510 L/s

Summary of Results		
Model	TSS Removal	Volume Treated
FD-4HC	97.7%	99.9%
FD-5HC	98.7%	99.9%
FD-6HC	99.1%	99.9%
FD-8HC	99.8%	99.9%

FD-4HC Specification	
Unit Diameter (A):	1,200 mm
Inlet Pipe Diameter (B):	0 mm
Outlet Pipe Diameter (C):	0 mm
Height,T/G to Outlet Invert (D):	0 mm
Height, Outlet Invert to Sump (E):	1,500 mm
Sediment Storage Capacity (F):	0.78 m³
Oil Storage Capacity (G):	723 L
Recommended Sediment Depth for Maintenance:	440 mm
Max. Pipe Diameter:	600 mm
Peak Flow Capacity:	510 L/s



Site Elevations:	
Rim Elevation:	0.00
Inlet Pipe Elevation:	0.00
Outlet Pipe Elevation:	0.00

Notes:

Removal efficiencies are based on NJDEP Test Protocols and independently verified.

All units supplied by ADS have numerous local, provincial, and international certifications (copies of which can be provided upon request). The design engineer is responsible for ensuring compliance with applicable regulations.

Project Name: 1600 Kerns Road - North East Parking Area
Consulting Engineer: KWA Site Development Consulting Inc.
Location: Burlington, Ontario

Net Annual Removal Efficiency Summary: FD-4HC

Rainfall Intensity[1]	Fraction of Rainfall[1]	FD-4HC Removal Efficiency[2]	Weighted Net-Annual Removal Efficiency
mm/hr	%	%	%
0.50	0.4%	100.0%	0.4%
1.00	18.8%	100.0%	18.8%
1.50	10.9%	100.0%	10.9%
2.00	13.9%	100.0%	13.9%
2.50	4.5%	100.0%	4.5%
3.00	2.7%	100.0%	2.7%
3.50	7.8%	100.0%	7.8%
4.00	4.2%	100.0%	4.2%
4.50	2.2%	100.0%	2.2%
5.00	4.7%	100.0%	4.7%
6.00	4.2%	99.3%	4.1%
7.00	4.2%	97.8%	4.1%
8.00	3.2%	96.6%	3.1%
9.00	1.9%	95.6%	1.9%
10.00	2.6%	94.7%	2.5%
20.00	9.2%	88.7%	8.2%
30.00	2.3%	85.5%	1.9%
40.00	1.1%	83.2%	0.9%
50.00	0.5%	81.5%	0.4%
100.00	0.6%	76.4%	0.4%
150.00	0.1%	73.6%	0.0%
200.00	0.0%	71.6%	0.0%
		Total Net Annual Removal Efficiency:	97.7%
		Total Runoff Volume Treated:	99.9%

Notes:

(1) Rainfall Data: 1970:2007, HLY03, Hamilton, ON, 6153194.

(2) Based on third party verified data and appoximating the removal of a PSD similar to the STC Fine distribution

(3) Rainfall adjusted to 5 min peak intensity based on hourly average.

ADS Treatment Train Sizing

Project Name: 1600 Kerns Road - South Driveway

Consulting Engineer: KWA Site Development Consulting Inc.

Location: Ontario

Sizing Completed By: Haider Nasrullah **Email:** haider.nasrullah@ads-pipe.com

Summary of Results	
Isolator Row PLUS TSS Removal:	81.2%
FD-4HC TSS Removal:	90.8%
Combined TSS Removal:	98.3%
Total Volume Treated:	100.0%

Site Details	
Site Area (ha):	0.5
Rational C:	0.90
Particle Size Distribution:	Fine
Rainfall Station:	Hamilton, ONT

Note: Isolator Row PLUS removal efficiencies based solely on ETV/NJDEP PSD, above-noted PSD is for OGS sizing only

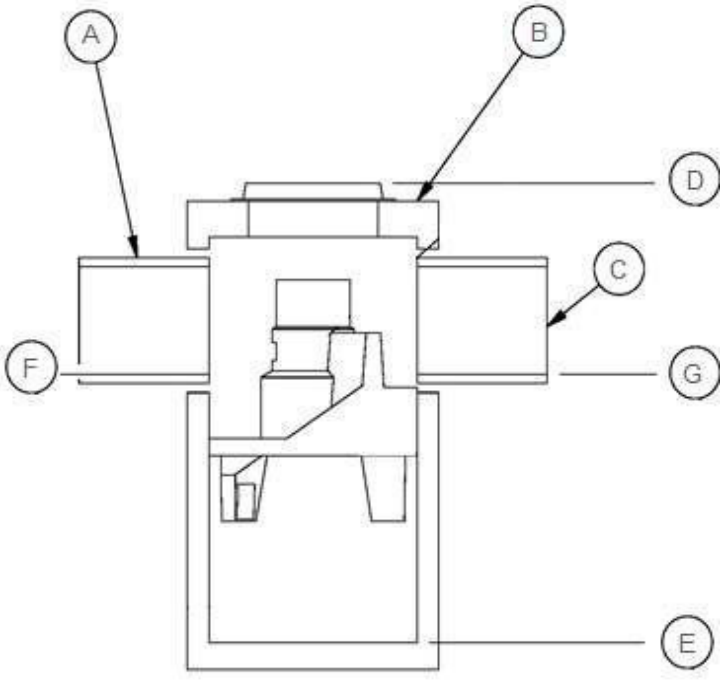
Individual OGS Results		
Model	TSS Removal	Volume Treated
FD-4HC	90.8%	99.8%
FD-5HC	93.8%	100.0%
FD-6HC	95.3%	99.9%
FD-8HC	97.4%	99.9%

Stormtech Details	
Chamber Model:	MC-3500
No. Chambers in Isolator Row PLUS:	13
Volume Treated by Isolator Row PLUS:	100.0%

Notes: Refer to Stormtech drawings for full IR+ configuration.

Isolator Row PLUS must include Flared End Ramp (FLAMP) for proper performance.

Overall System Capacities	
Total Sediment Storage Capacity:	11.6 m³
Oil Storage Capacity:	723 L
Max. OGS Pipe Diameter:	600 mm
Peak OGS Flow Capacity:	510 L/s
Peak Isolator Row PLUS Flowrate:	145.5 L/s



OGS Specifications	
Inlet Pipe Diameter (A):	1,200 mm
Unit Diameter (B):	0 mm
Outlet Pipe Diameter (C):	0 mm
Top of Grade Elevation (D):	0.00 m
Bottom of Sump Elevation (E):	-1.50 m
Inlet Pipe Elevation (F):	0.00 m
Outlet Pipe Elevation (G):	0.00 m

Notes:

Isolator Row PLUS removal efficiency based on verified ETV test report. For dimensions and configuration of Isolator Row PLUS, please see Stormtech drawing package.

Project Name: 1600 Kerns Road - South Driveway
Consulting Engineer: KWA Site Development Consulting Inc.
Location: Ontario

Net Annual Removal Efficiency Summary

Rainfall Intensity	Fraction of Rainfall	Removal Efficiency		Combined Removal Efficiency	Combined Weighted Removal Efficiency
		FD-4HC	IR PLUS		
mm/hr	%	%	%	%	%
0.50	0.4%	100.0%	81.2%	100.0%	0.4%
1.00	18.8%	100.0%	81.2%	100.0%	18.8%
1.50	10.9%	98.1%	81.2%	99.6%	10.9%
2.00	13.9%	95.5%	81.2%	99.2%	13.7%
2.50	4.5%	93.5%	81.2%	98.8%	4.5%
3.00	2.7%	92.0%	81.2%	98.5%	2.7%
3.50	7.8%	90.7%	81.2%	98.2%	7.7%
4.00	4.2%	89.5%	81.2%	98.0%	4.1%
4.50	2.2%	88.6%	81.2%	97.8%	2.1%
5.00	4.7%	87.7%	81.2%	97.7%	4.6%
6.00	4.2%	86.2%	81.2%	97.4%	4.0%
7.00	4.2%	85.0%	81.2%	97.2%	4.1%
8.00	3.2%	83.9%	81.2%	97.0%	3.1%
9.00	1.9%	83.0%	81.2%	96.8%	1.9%
10.00	2.6%	82.2%	81.2%	96.7%	2.5%
20.00	9.2%	77.1%	81.2%	95.7%	8.8%
30.00	2.3%	74.2%	81.2%	95.2%	2.2%
40.00	1.1%	72.3%	81.2%	94.8%	1.0%
50.00	0.5%	70.8%	81.2%	94.5%	0.5%
100.00	0.6%	66.4%	81.2%	93.7%	0.5%
150.00	0.1%	63.9%	63.0%	86.6%	0.1%
200.00	0.0%	62.2%	47.2%	80.1%	0.0%
		Total Net Annual Removal Efficiency			**98.3%**
		Total Runoff Volume Treated			**100.0%**

Notes:

(1) Rainfall Data: 1970:2007, HLY03, Hamilton, ON, 6153194.

(2) Based on third party verified data and appoximating the removal of a PSD similar to the STC Fine distribution

(3) Rainfall adjusted to 5 min peak intensity based on hourly average.

(4) Combined removal efficiencies calculated based on NCDENR Stormwater BMP Manual, Section 3.9.4, where Total Removal Efficiency = 1st BMP Efficiency + 2nd BMP Efficiency - (1st BMP Efficiency x 2nd BMP Efficiency)

ADS OGS Sizing Summary

Project Name:	1600 Kerns Road - South Driveway
Consulting Engineer:	KWA Site Development Consulting Inc.
Location:	Burlington, Ontario
Sizing Completed By:	Haider Nasrullah

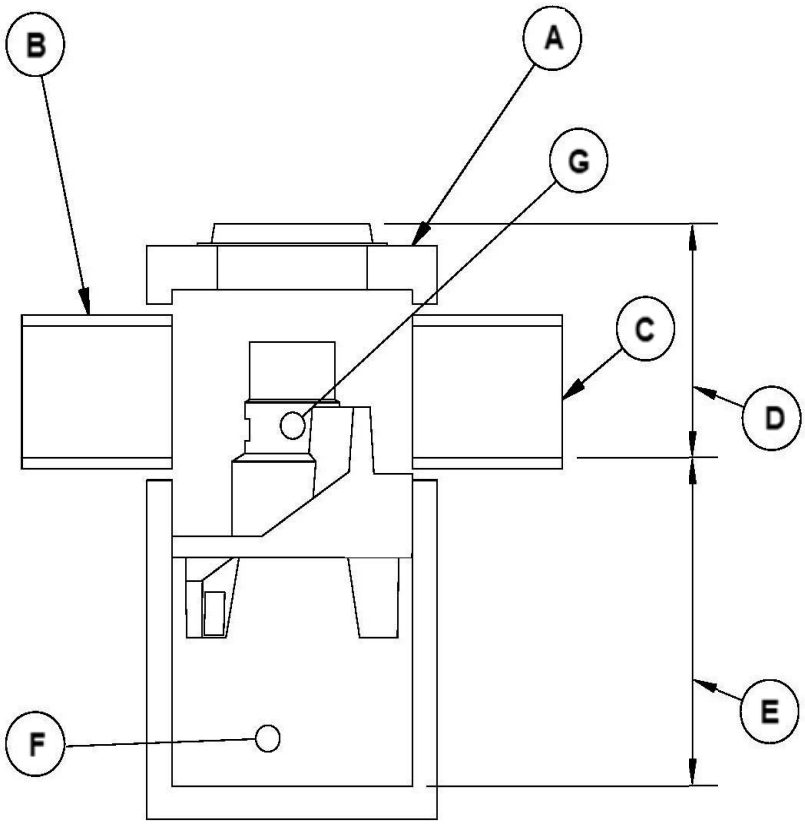
Email: haider.nasrullah@ads-pipe.com

Treatment Requirements		
Treatment Goal:	Enhanced (MOE)	
Selected Parameters:	80% TSS	90% Volume
Selected Unit:	**FD-4HC**	

Site Details	
Site Area:	0.068 ha
% Impervious:	100%
Rational C:	0.90
Rainfall Station:	Hamilton, ONT
Particle Size Distribution:	Fine
Peak Flowrate:	510 L/s

Summary of Results		
Model	TSS Removal	Volume Treated
FD-4HC	98.7%	99.9%
FD-5HC	99.3%	99.9%
FD-6HC	99.7%	99.9%
FD-8HC	99.9%	99.9%

FD-4HC Specification	
Unit Diameter (A):	1,200 mm
Inlet Pipe Diameter (B):	0 mm
Outlet Pipe Diameter (C):	0 mm
Height,T/G to Outlet Invert (D):	0 mm
Height, Outlet Invert to Sump (E):	1,500 mm
Sediment Storage Capacity (F):	0.78 m³
Oil Storage Capacity (G):	723 L
Recommended Sediment Depth for Maintenance:	440 mm
Max. Pipe Diameter:	600 mm
Peak Flow Capacity:	510 L/s



Site Elevations:	
Rim Elevation:	0.00
Inlet Pipe Elevation:	0.00
Outlet Pipe Elevation:	0.00

Notes:

Removal efficiencies are based on NJDEP Test Protocols and independently verified.

All units supplied by ADS have numerous local, provincial, and international certifications (copies of which can be provided upon request). The design engineer is responsible for ensuring compliance with applicable regulations.

Project Name: 1600 Kerns Road - South Driveway
Consulting Engineer: KWA Site Development Consulting Inc.
Location: Burlington, Ontario

Net Annual Removal Efficiency Summary: FD-4HC

Rainfall Intensity[1]	Fraction of Rainfall[1]	FD-4HC Removal Efficiency[2]	Weighted Net-Annual Removal Efficiency
mm/hr	%	%	%
0.50	0.4%	100.0%	0.4%
1.00	18.8%	100.0%	18.8%
1.50	10.9%	100.0%	10.9%
2.00	13.9%	100.0%	13.9%
2.50	4.5%	100.0%	4.5%
3.00	2.7%	100.0%	2.7%
3.50	7.8%	100.0%	7.8%
4.00	4.2%	100.0%	4.2%
4.50	2.2%	100.0%	2.2%
5.00	4.7%	100.0%	4.7%
6.00	4.2%	100.0%	4.2%
7.00	4.2%	100.0%	4.2%
8.00	3.2%	100.0%	3.2%
9.00	1.9%	100.0%	1.9%
10.00	2.6%	99.0%	2.6%
20.00	9.2%	92.8%	8.6%
30.00	2.3%	89.4%	2.0%
40.00	1.1%	87.0%	0.9%
50.00	0.5%	85.2%	0.4%
100.00	0.6%	79.9%	0.5%
150.00	0.1%	76.9%	0.1%
200.00	0.0%	74.9%	0.0%
		Total Net Annual Removal Efficiency:	98.7%
		Total Runoff Volume Treated:	99.9%

Notes:

(1) Rainfall Data: 1970:2007, HLY03, Hamilton, ON, 6153194.

(2) Based on third party verified data and appoximating the removal of a PSD similar to the STC Fine distribution

(3) Rainfall adjusted to 5 min peak intensity based on hourly average.

Appendix E

Water Balance

Excerpts from EXP Hydrogeolgical and Water Balance

4 Water Balance Study

4.1 Background Information

It is our understanding that the proposed development plan is to demolish the existing building and construct a proposed three (3) to four (4) storey residential condominium development with a single basement level (P1). The Site location plan is shown on Figure 1.

The Site is in an urban area. The topography has a relatively flat gradual south slope towards Lake Ontario. The west section of the site steeply slopes down towards the Upper Hager Creek.

The surficial geology can be described as Glaciolacustrine-derived Silty to Clayey Silt on the north eastern section of the site and Paleozoic bedrock on the south western section of the site (Ontario Geological Survey, 2010). The surficial geology of the Site and surrounding areas is shown on Figure 2.

Upper Hager Creek is located on the western border of the site. The Creek flows in a southerly direction. The shallow groundwater elevations reflect a general southwestern groundwater flow direction, towards Upper Hager Creek located near the western boundary of the Site.

4.2 Methodology

The Thornthwaite water balance (Thornthwaite, 1948; Mather, 1978; 1979) is an accounting type method used to analyze the allocation of water among various components of the hydrologic cycle. This methodology was used to complete the pre-construction (existing conditions) and post-development water balance. Inputs to the model are monthly temperature, precipitation and Site latitude. Outputs include monthly potential and actual evapotranspiration, soil moisture storage, soil moisture storage change, surplus, infiltration and runoff.

When precipitation (P) occurs, it can either runoff (R) through the surface water system, infiltrate (I) to the water table including an interflow component, or evapotranspiration (ET) from the earth's surface and vegetation. The difference between total precipitation (P) and the total of evaporation and evapotranspiration (ET) is defined to be the water surplus (S) which is available for both infiltration (recharge to the groundwater system including interflow) and for runoff. When long-term averages of P, R, I and ET are used, there is no net change in groundwater storage (ST). Annually, however, there is a potential for small changes in ST.

The annual water budget can be stated as follows:

$$P = ET + R + I + ST$$

Where:

P = precipitation
ET = evapotranspiration
R = surface water runoff
I = Infiltration
ST = change in groundwater storage

For this assessment, the Thornthwaite and Mather method was used to estimate average annual infiltration rates.

Infiltration is governed by the surficial soil types, topography and land cover.

For ease of calculation, a spreadsheet model was used for the computation. The Thornthwaite and Mather Model is based on the United Stated Geological Survey (USGS) graphical user interface (Thornthwaite Monthly Water-Balance program, 2007).

4.3 Meteorological Data

Meteorological data including average monthly precipitation and average temperatures were obtained from the National Climate Data and Information Archive (Environment Canada) for the Hamilton Airport (Station ID No. 6153194), which is the closest climate station to the Site having continues 30-year meteorological data.

Climate stations Hamilton RBG and Hamilton RBG CS are located closer than the Hamilton Airport climate station, however these two stations do not have 30-year climate data posted.

Meteorological data for 30 years from 1980 to 2010 was utilized for the assessment. Summary of input data is provided in Appendix F-1.

4.4 Pre- and Post-Development Site Characteristics

4.4.1 Pre-Development Site Characteristics

A summary of the existing (pre-development) landscape features is provided in Table 4.1:

Table 4.1: Pre-Development (Existing) Land Use

Description	Pre-Construction (Existing) (m^2)	Percentage (%)
Total Site Area	8,000	100
Existing Buildings	1,900	23.75
Paved Surfaces	3,200	40.00
Landscaped Area	1,000	12.50
Woodlot	1,700	21.25
Surface Water (Upper Hager Creek)	200	2.50
Site Area Available for Infiltration Contribution	2,700	33.75

It should be noted that the areas provided in Table 4.1 above were determined based on a review of available Site plans and these estimates are considered appropriate for estimating the water balance.

As evident from the information provided in Table 4.1, under pre-development conditions, approximately 33.75% of the Site area is pervious and available for groundwater infiltration (Figure 7).

4.4.2 Post-Development Site Characteristics

As provided in draft Site Plan, Table 4.2 provides a summary of the post-development Site characteristics.

Table 4.2: Post-Development Site Characteristics

Description	Impervious Areas (ex: Buildings, Roads) m^2	Pervious Areas / Areas available for Infiltration m^2	Total Areas Post-Construction (Proposed) m^2
ROW (roads, sidewalks, parking)-paved	1,200	0	1,200
Building Roofs (Condominium Building)	2,930	0	2,930
Landscaped	0	1,970	1,970
Woodlot	0	1,700	1,700
Surface Water (Upper Hager Creek)	200	0	200
Totals	**4,330**	**3,670**	**8,000**

Note: Full basement area is included in the areas identified as impermeable

Under post-development conditions, the total pervious area is increased to 45.9% of the total Site area (Figure 8).

4.5 Pre-Development Water Balance Estimates

4.5.1 Climate Data Analysis

The mean annual water surplus was calculated by using the Thornthwaite and Mather (1955) method. Monthly average precipitation values were obtained for 30 years (1980 to 2010) from the National Climate Data and Information Archive (Environment Canada) for the Hamilton A (Station ID No. 6153194). Hamilton Airport is approximately 20 km away from the Site.

Moisture storage of 200 mm/yr was assumed for soils and considered to be representative of pre-construction Site conditions. The closest Latitude to the Site is 43^0, which was used in the USGS model (2007).

Table 4.3 summarizes the climatic water balance analysis. Appendix F-1 and F-2 provide the model input and output, respectively.

Table 4.3: Summary of Climatic Water Balance Analysis in Pre-Development Conditions

Soil Moisture Storage (mm/yr)	Precipitation (mm/yr)	Actual ET (mm/yr)	Surplus (mm/yr)
200 mm/yr Clayey Silt	928.1	564.8	363.3

Note: ET = Evapotranspiration

The results of climatic water balance analysis for the Site suggest that a surplus of 363.3 mm/year of water is available for surface runoff and infiltration.

4.5.2 Infiltration

The infiltration is expected to be controlled by soil type, topography and soil cover type. Surplus water is portioned between runoff and infiltration based on the controlling factors provided by MOE (1995). It is noted that the controlling factors provided by the MOE were used for estimating infiltration factors.

Using this method, a total infiltration factor for the Site was estimated by using the individual sub-factors, which are representative of the topography, soil type and land cover conditions (Figures 2, 9A and 9B). Appendix H-3 provides a summary of the sub factors and total factor based on the Site conditions. The infiltration sub-factors were determined for estimating pre-development infiltration rates of the entire Site.

The estimated pre-development total infiltration factor of 0.323 (or 32.3 %) represents the fraction of the water surplus available for infiltration. The complementary fraction of the available water for runoff is 0.677. The infiltration factor is utilized to calculate the amount of annual infiltration (in units of m^3/yr) at the Site by multiplying it with the average yearly water surplus estimate and with the Site area available for infiltration.

Applying the infiltration factor of 0.323 and a water surplus of 363.3 mm/yr, the estimated pre-development infiltration rate of the whole Site is 117.35 mm/yr.

In areas with water table at surface, the infiltration rate was considered negligible for existing and proposed grade.

5 Pre-Development Water Balance Analysis

The water balance analysis is based on available information on a regional scale and considered representative for the Site. Table 5.1 provides a summary of water balance analysis for the Site.

Table 5.1: Summary of Overall Pre-Development Water Balance Results

Location	Total Site Area (m^2)	Area Available for Infiltration (m^2)	Total Precipitation (m^3/yr)	Actual Evapotranspiration (m^3/yr)	Runoff (m^3/yr)	Infiltration (m^3/yr)
Total Site	8,000	2,700	7,425	1,711	5,397	317
Percentage of Total Precipitation			100%	23.0%	72.7%	4.3%

The total property area was used to estimate the annual precipitation volume for the Site (Appendix H-4). As summarized in Table 5.1, the breakdown of the pre-development water balance is as follows: 23.0% of the total precipitation will be subject to evapotranspiration, 72.7% to runoff and 4.3% to infiltration.

The pre-development water balance, on a weighted average depth basis (in mm) is as follows:

P (928.10) = ET (564.80) + R (245.95) + I (117.35) + ST (0).

5.1 Post-Development Water Balance Estimates

5.1.1 Post-Development Water Balance

Based on the proposed development drawings, the total area of impervious surfaces under post-development conditions is approximately 4,330 m^2, representing approximately 54.1% of the Site area (total site area: 8,000 m^2), (Table 4.2). The remaining 3,670 m^2 is available to contribute to infiltration during the post-development stage (approximately 45.9% of the total land area).

Post-development infiltration sub-factors were determined in a similar manner as for estimating infiltration sub-factors for pre-development Site conditions, both based on the method recommended by MOE (1995). For post-development infiltration sub-factors, the landscaped areas were assumed to be consistent with cultivated cover with an infiltration sub-factor of 0.1 (Appendix F-3).

Table 5-2 presents a summary of the overall post-development water balance assessment.

Table 5-2: Summary of Overall Post-Development Water Balance Forecast

Location	Total Site Area (m^2)	Area Available for Infiltration (m^2)	Total Precipitation (m^3/yr)	Evapo-transpiration (m^3/yr)	Runoff (m^3/yr)	Infiltration (m^3/yr)
Total Site	8,000	3,670	7,425	2,259	4,712	455
Percentage of Total Precipitation			100%	30.4%	63.5%	6.1%

If no remedial measures are implemented to maintain infiltration, it is expected that the annual infiltration volume will be increased from approximately 317 m^3/yr to 455 m^3/yr in post-development, resulting in an excess of 138 m^3/yr (Appendix F-4).

Infiltration deficits based on pre- and post-development Site conditions can be utilized to guide mitigation measures under idealized soil and groundwater conditions. Due to the increase of areas available for infiltration by approximately 970 m^2, it is expected that the unmitigated infiltration rate increase of 138 m^3/yr can be maintained under the post-development Site conditions under sound conditions.

Under unmitigated post-development conditions, an increase in annual infiltration volume may occur, as compared to pre-development conditions. As a result, water contribution from infiltration to downgradient surface water features would increase.

5.2 Impact and Proposed Mitigation Measures

As shown in Appendix F-4, Due to the increase of areas available for infiltration by approximately 970 m^2, it is expected that the unmitigated infiltration rate increase of 138 m^3/yr can be maintained under the post-development Site conditions under sound conditions.

Due to the increase of groundwater infiltration during the post development phase of the project, compared to the pre-development groundwater infiltration rate, there will be no decrease in the groundwater discharge rates at the Upper Hager Creek.

Since the groundwater infiltration rate under the post development conditions is increased compared to the groundwater infiltration during the pre-construction phase, additional mitigation measures may not be required.