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

February 25th, 2022

Table of Contents

1.0 Introduction 1
2.0 Study Parameters 1
3.0 Design Population 2
4.0 Water Supply 2
4.1 Supply Demands 2
4.2 Proposed Connection and Layout 3
5.0 Sanitary Drainage 4
5.1 Sanitary Design Flow 4
5.2 Existing Sanitary Flow 4
5.3 Proposed Sanitary Flow 4
5.4 Proposed Connection and Layout 5
6.0 Stormwater Management 5
6.1 Pre-Development Drainage Conditions 5
6.2 Existing Pre-Development Peak Flows 6
6.3 Design Criteria 6
6.4 Allowable Release Rate 7
6.5 Post-Development Drainage Conditions 7
6.6 Runoff Quantity Control – 100 year Post to 5 Year Pre (C=0.5) 8
6.6.1 Short Term and Long-Term Dewatering 8
6.6.2 Quantity Control 9
6.7 Post Development Quality Control 10
6.7.1 Chemical Analysis 10
6.7.2 Low Impact Development Techniques 11
6.7.3 Quality Control 11
6.8 Water Balance 12
7.0 Conclusion 13

List of Tables

- Table 1 – Water Supply Demand
- Table 2 – 2021 and 2031 Fire Flows and Domestic Pressures at Node WFT92380
- Table 3 – Existing Drainage Area, Composite Runoff Coefficient and Drainage Direction
- Table 4 – Existing 5-year Pre-Development Release Rates
- Table 5 – Allowable Release Rate with Maximum Runoff Coefficient (C=0.5) for Re-development Sites
- Table 6 – Proposed Drainage Areas, Runoff Coefficient and Drainage Direction
- Table 7 - Area 201A and 202B - Summary of 100-Year Release Rate and Storage Requirements
- Table 8 – 100-Year Release Rates and Storage Provided
- Table 9 – Hard Surface Areas Draining to Kerns Road
- Table 10 – Proposed Emergency Overland Drainage Direction

List of Figures

- Figure 1 – Site Location Plan
- Figure 2 – Proposed Site Servicing Plan

List of Appendices

- Appendix A – Water Supply Calculations
- Appendix B – Sanitary Design Calculations
- Appendix C – Stormwater Quantity Calculations / Excerpts from EXP Slope Stability Analysis / Excerpts from EXP Hydrogeological and Water Balance
- Appendix D – Stormwater Quality Control / Excerpts from EXP Hydrogeological and Water Balance
- Appendix E – Water Balance / Excerpts from EXP Hydrogeological and Water Balance
- Appendix F – Drawings

List of Drawings

- 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. (February 25, 2022)
- Slope Stability Analysis by EXP Services Inc. (February 11, 2022)
- 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.
- Re-zoning comments from City of Burlington, Halton Region, and Conservation Halton (received July – Nov 2021)



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

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL
2.	02/25/22	RE-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

3.0 DESIGN POPULATION

The proposed re-development contemplates a total of 118 (1-bedroom) residential units with approximately 23 staff members and 24 visitors per day. The commercial area has an equivalent population of 3 people. The projected design population for the re-development is **168 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
- Equivalent Population Density for Light Commercial Areas = 90 person/ha
- Maximum Daily Demand Peaking Factor = 2.25
- Maximum Hourly Demand Peaking Factor for Residential = 4.00 and Commercial = 2.25
- 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 – Residential 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	165	0.53	2.25	4.00	**1.18**	**2.10**	**193**
Retail	3	0.01	2.25	2.25	**0.02**	**0.02**	

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 = **193.87 L/s**
- Maximum Hourly Demand = **2.12 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 rectangular water chamber with three valves, as per Region standard RH 402.080, is placed at the connection point within Kerns Road. A 250mm fire service and 100mm domestic service, as per Region standard RH 409.010, will be brought to the building's 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.

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 was completed on September 1, 2021 for a hydrant located in front of 1548 and 1588 Kerns Road. The recorded static pressure was 78 psi. The available flow rate of 201 L/s was observed while maintaining a pressure of 20 psi. The hydrant flow test confirms available capacity within the system.

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 = 168 persons
- 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.

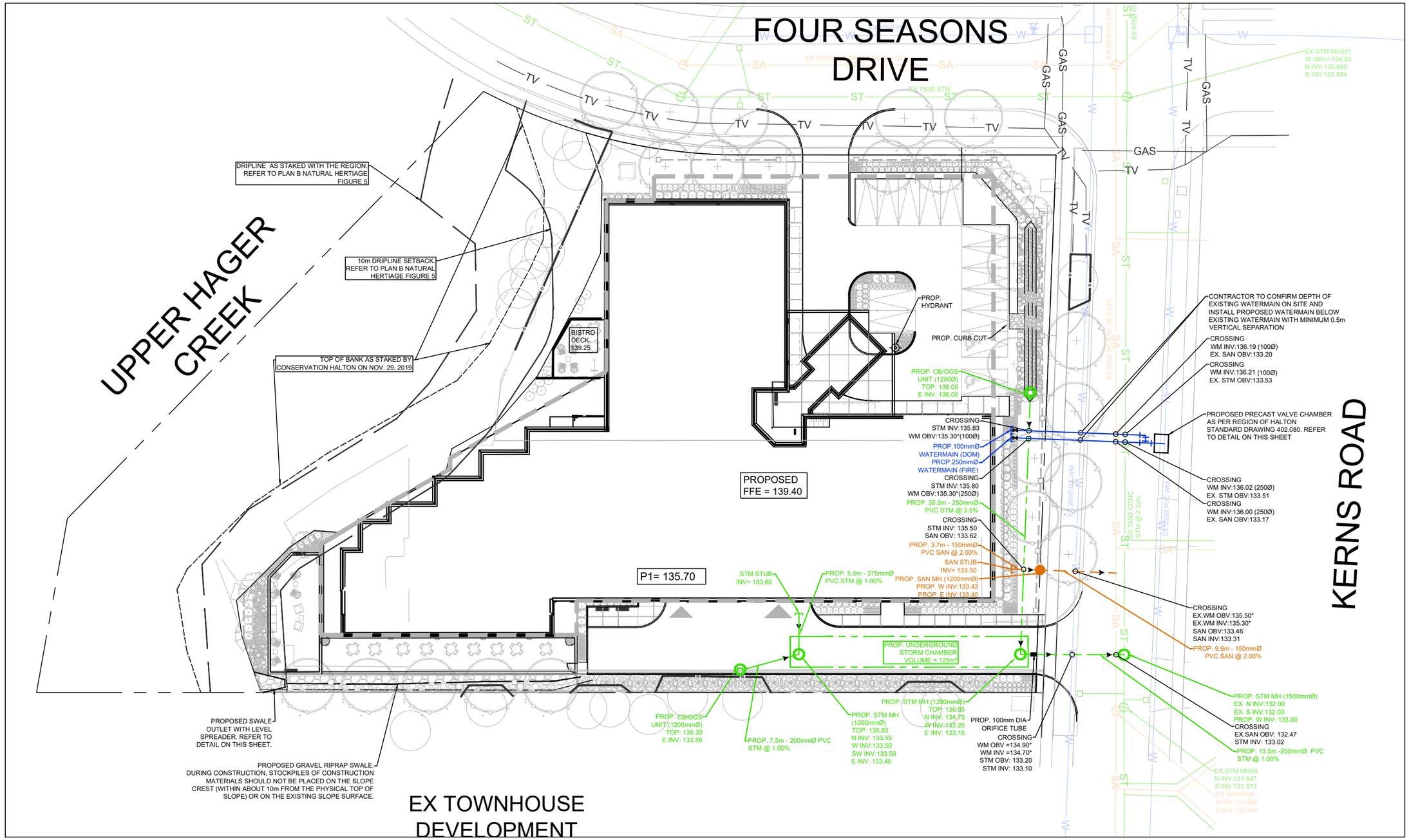
Using a total site area of 0.63 ha, equivalent population density of 57 persons, calculated Harmon's Peaking factor of 4.3, and infiltration flow rate of 0.286 L/s/ha, the pre-development sanitary sewer flow is:

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

5.3 PROPOSED SANITARY FLOW

Using a proposed total population density of 168 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:

$$Q_{san\ post} = (275\ Lpcd \times 168\ persons \times 4.2_{peaking\ factor} / 86400) + (0.286\ L/s/ha \times 0.63\ ha) = \mathbf{2.41\ L/s}$$



LEGEND

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

REVISION BLOCK

#	DATE	DESCRIPTION
1.	03/26/21	ISSUED FOR APPROVAL
2.	02/25/22	RE-ISSUED FOR APPROVAL

KWA SITE DEVELOPMENT CONSULTING INC.
4063 Mainway, 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: 20509 | DRAWN BY: BCJ | CHECKED BY: BC

FIG 2

The proposed peak sanitary sewer flow from this Site will increase by **1.46 L/s** (2.41 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 2.00% slope will be brought to the underground parking limit by gravity and connect to the existing sanitary sewer at elevation 133.20 m.

The capacity of the proposed 150mm sanitary service at 2.00% is **21.5 L/s**. The existing capacity of the 200mm sanitary sewer at 2.53% on Kerns Road is **52.2 L/s**. The 2.41 L/s theoretical sanitary flow from the proposed redevelopment equates to 4.6% 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.

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 0.48 ha will drain to underground storm sewers that outlet to Kerns Road. The remaining site area, 0.15 ha of landscaped and amenity area will drain to 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	0.46	0.89	To Kerns Road

201B	East Landscape Areas	0.02	0.25	To Kerns Road
202	North, South, West Landscape Area Amenity Area	0.15	0.44	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

$Q_{100yr} = A \times R \times I_{100yr} \times 360 = 0.48 \times 1.0 \times 163.18/ 360 = 0.22$ m3/s = **218 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

$Q_{100yr} = A \times R \times I_{100yr} \times 360 = 0.15 \times 0.55 \times 163.18/ 360 = 0.04$ m3/s = **37 L/s**

For Area 201A and 201B, the post development uncontrolled flow of 218 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 37 L/s is equal to 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 and an amenity area near the south-west corner of the site. No new piped outlet to the Upper Hager Creek is required. A swale along the south property line is proposed to convey flows from the amenity area. Flows from the swale are directed to a flow level spreader to prevent soil erosion. Refer to Section 6 in EXP's Slope Stability Analysis for flow velocity at the spreader outlet. Excerpts from the EXP Report are included in Appendix C.

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.

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. EXP's Hydrogeological and Water Balance Investigation indicated high groundwater elevations; therefore, the underground storage tank will be water tight. 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 (m^3)	Storage Required (m^3)	Release Rate (m^3/s)	Description	Orifice Size (mm)	Orifice Release Rate (m^3/s)
201A	0.46	1.00	10	135	116	0.046	Controlled to Kerns Road	100	0.043
201B	0.02	0.31	10		0	0.002	Uncontrolled to Kerns Road		
	0.48			135	116	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 (m^3)
201A, 201B	0.48	1.00	To Kerns Road	52	48	135
202	0.15	0.55	To Upper Hager Creek	37	37	0

Refer to Appendix C for detailed Quantity Control calculations.

Storms up to and including the 100-year storm event are retained on-site. For areas 201A and 201B, flows are stored in the underground storage tank and discharged to the Kerns Road storm sewer at a controlled allowable release rate. For area 202, no controls are proposed as the post-development 100-year peak flows are equal to the pre-development 5-year peak flows.

Some ponding can be expected during the 100-year storm event in the south driveway area in 201A. Flows from this area are directed into a catch basin manhole (top elevation of 135.30 m). This catch basin manhole is subject to a maximum ponding of 0.13 m (100-year ponding elevation of 135.43 m) with the conversative assumption that the inlet grate at this catch basin manhole is 50% blocked. Catch basin inlet capacity calculations are included in Appendix C.

The south swale can provide emergency overland flow for storms above the 100-year storm event. Emergency overland flow in the south parking lot can not be directed to Kerns Road (entrance elevation of 136.90 m). The highest ponding elevation along the emergency overland drainage route (above the 100-year storm event) is 135.55 m and the top of underground parking ramp elevation is a minimum 0.15 metres higher at 135.70 m.

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."
- "It is recommended to discharge groundwater effluent from under-drain system to the storm sewer system of City of Burlington after treatment. Additional water sampling and analysis to compare groundwater quality with the Storm Sewer Use By-Law including the new list of parameters currently under review and applying suitable pre-treatment will be required prior to directing any groundwater discharge to the storm sewer system. A treatment system associated with an Environmental Compliance Approval (ECA) will be required to treat any exceedances of the storm sewer use by law."

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 a significant portion of the site, approximately 0.36 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. Refer to OGS sizing and Environmental Technology Verification statement included in Appendix D.

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 0.46 ha (Area 201A).

$$V_{Required} = \text{Depth} \times A_{site} = 5.0 \text{ mm} \times 4{,}600 \text{ m}^2 / 1000 \text{ mm} = \mathbf{23.0\ m^3}$$

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 **23.0 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 = **193.87 L/s**
    - Maximum Hourly Demand = **2.12 L/s**
- A hydrant flow test was completed on September 1, 2021 for a hydrant located in front of 1548 and 1588 Kerns Road. The available flow rate of 201 L/s was observed while maintaining a pressure of 20 psi. The hydrant flow test confirms available capacity within the system.

Sanitary Servicing:

- Based on the post development design flow of **2.41 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 2.00% slope will be brought to the underground parking limit by gravity and connect to the existing sanitary sewer at elevation 133.20 m.
- The capacity of the proposed 150mm sanitary service at 2.00% is **21.5 L/s**. The existing capacity of the 200mm sanitary sewer at 2.53% on Kerns Road is **52.2 L/s**.
- The 2.41 L/s theoretical sanitary flow from the proposed redevelopment equates to 4.6% 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 **135 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 37 L/s is equal to 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 and amenity area within the 7.5m setback area.

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.

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."

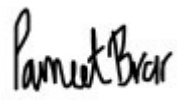
- Based on the updated pre-consultation application comments Oct. 2020, the water balance criteria noted was to retain / infiltrate the first 5mm of storm runoff onsite. Total water balance volume required is **23.0 m^3.**
- To meet the water balance objectives, a separate tank within the building or 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.
- 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:

Pavneet Brar, MASc, EIT

pavneet.brar@kwasitedev.com

Carlo Del Buono, P. Eng., LEED AP

carlo.delbuono@kwasitedev.com

APPENDIX A

FUS Calculation

RH 402.080

RH 409.010

Water Model Schematic

Hydrant Flow Test

Fire Underwriters Survey - Fire Demand Calculations

Project Name: 1600 Kerns Road
Project Number: 20500
Location: City of Burlington
Date: Feb-22
Prepared By: PB

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		8289	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	8289	**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

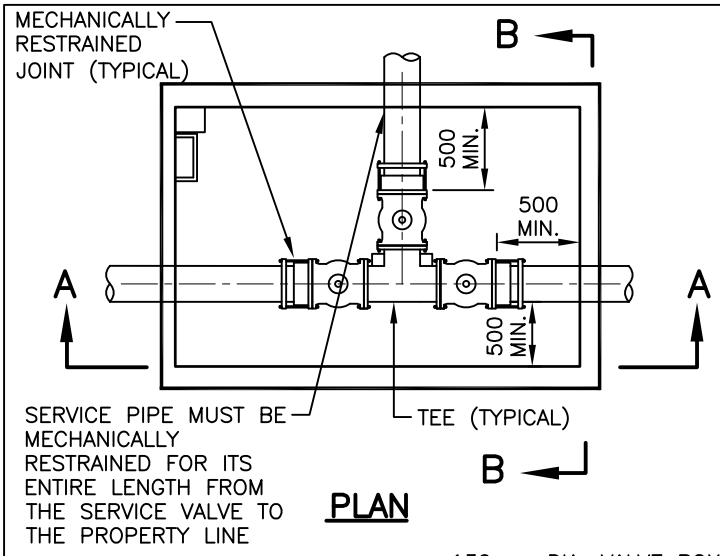
Construction Class
- WF wood frame construction
- OC ordinary construction
- NC non-combustible construction
- FC fire-resistive construction

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

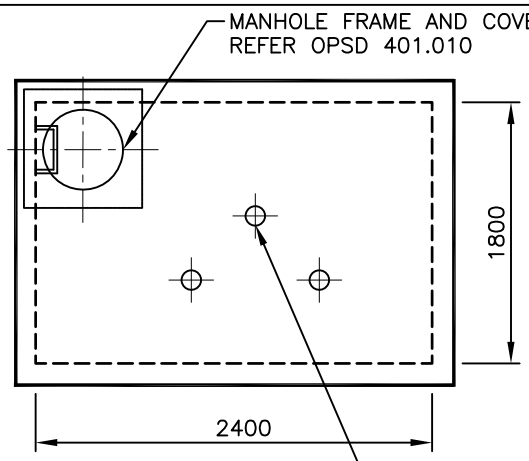
Non-Combustible	-25%	Free Burning	+15%
Limited Combustible	-15%	Rapid Burning	+25%
Combustible	No Charge		

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

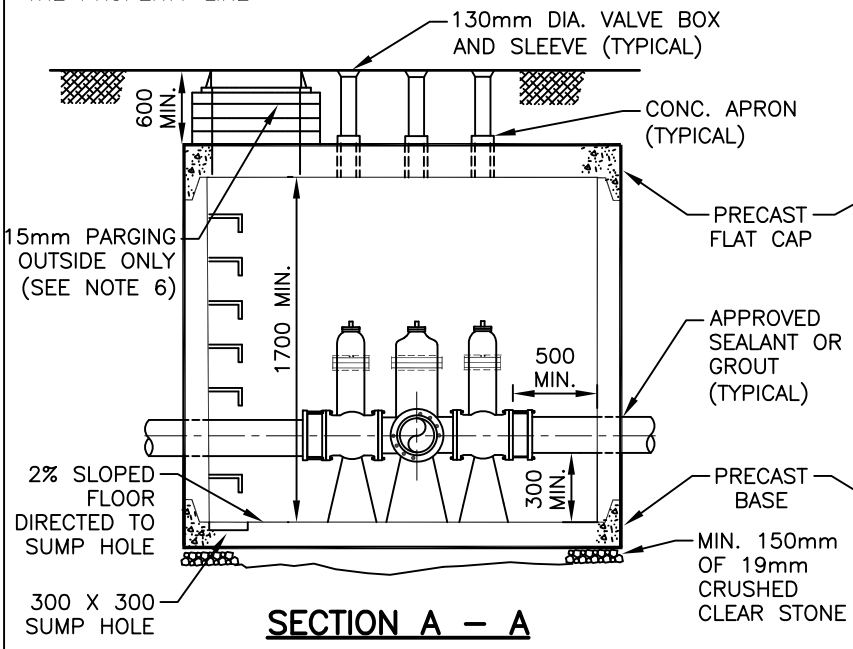
Separation	Charge	Separation	Charge
0 to 3m	25%	20.1 to 30 m	10%
3.1 to 10m	20%	30.1 to 45m	5%
10.1 to 20m	15%		



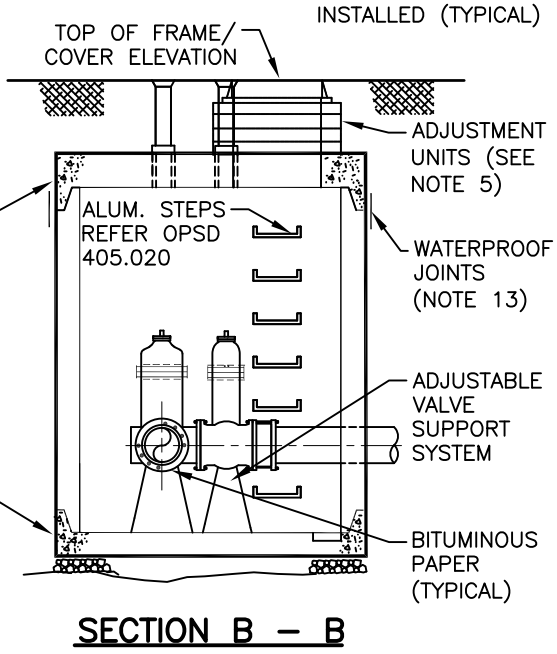
PLAN



TOP VIEW



SECTION A – A



SECTION B – B

NOTES:

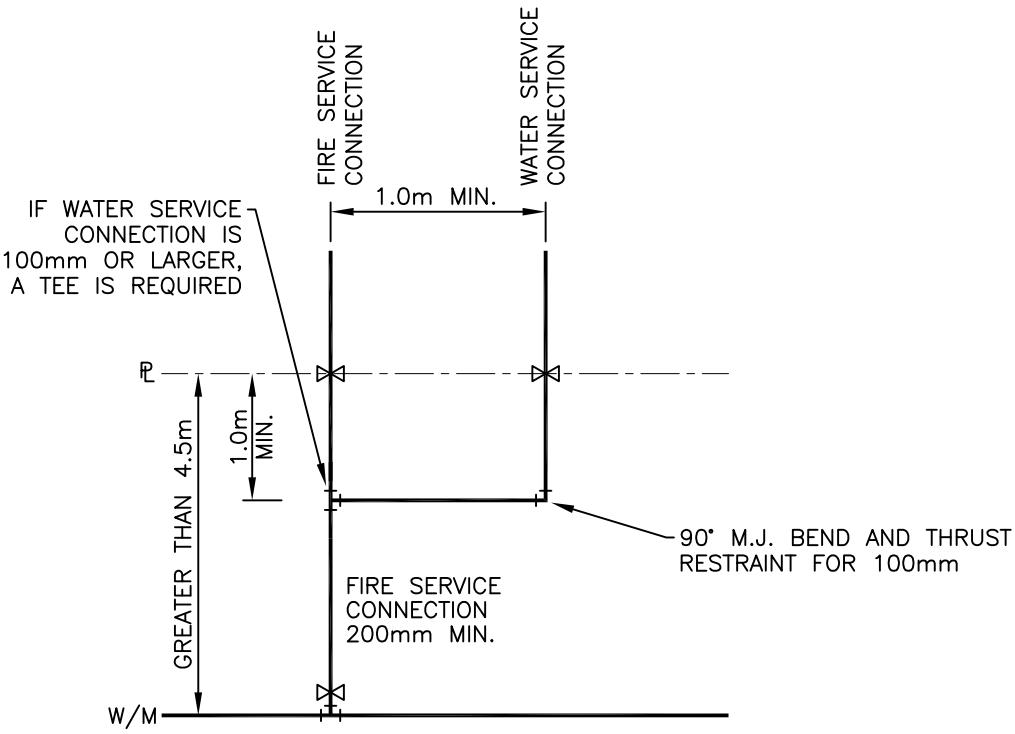
1. REINF. CONC. PRECAST CHAMBER OR MONOLITHIC CHAMBER TO MEET CURRENT OPSD.
2. STEPS: 1ST STEP TO BE 450mm BELOW FINISHED ROAD GRADE, LAST STEP TO BE 300mm ABOVE BASE.
3. VALVES TO BE FLANGED.
4. THE TOP OF THE ROOF SLAB OF VALVE CHAMBERS SHALL BE MIN. 600mm BELOW FINISHED GRADE. AN ENGINEERED COLLAR MAY BE REQUIRED TO MAKE UP FOR ELEVATION DIFFERENCES.
5. ADJUSTMENT UNITS TO BE MIN. 150mm TO MAX. 300mm.
6. PARGING MIX ON ALL BRICK WORK TO BE 1:3 MORTAR MIX AND BE APPLIED 15 mm THICK.
7. ALL JOINTS AND LIFTING HOLES IN CHAMBER SECTIONS TO BE COMPLETELY FILLED WITH 1:3 MORTAR MIX AND POINTED BEFORE BACKFILLING.
8. JOINT FILLER RUBBER GASKET BETWEEN ALL PRECAST SECTIONS (TYPICAL).
9. RESTRAINING RODS AND T–HEAD BOLT WITH NUT ARE CORROSION–RESISTANT, FLUOROPOLYMER COATED, HIGH–STRENGTH LOW–ALLOY STEEL THAT CONFORMS TO ANSI/AWWA C111/A21.11.
10. FOR 300mm AND 400mm TEES, THE MAIN LINE CAN BE OFFSET FROM THE CENTRELINE OF CHAMBER UP TO 200mm AS NEEDED.
11. PETROLATUM COATING ON ALL FITTINGS AND BOLTS
12. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
13. WATERPROOF MEMBRANE SHALL BE SELF–ADHESIVE, RUBBERIZED ROLL–TYPE BITUMINOUS WATERPROOFING MEMBRANE AND PRIMER. WATERPROOFING SHALL EXTEND COMPLETELY AROUND ALL JOINTS WITH A MINIMUM 300mm WIDE STRIP CENTERED ON THE JOINT, INSTALLED AS PER MANUFACTURER'S RECOMMENDATIONS.

THE REGIONAL MUNICIPALITY OF HALTON
PUBLIC WORKS DEPARTMENT

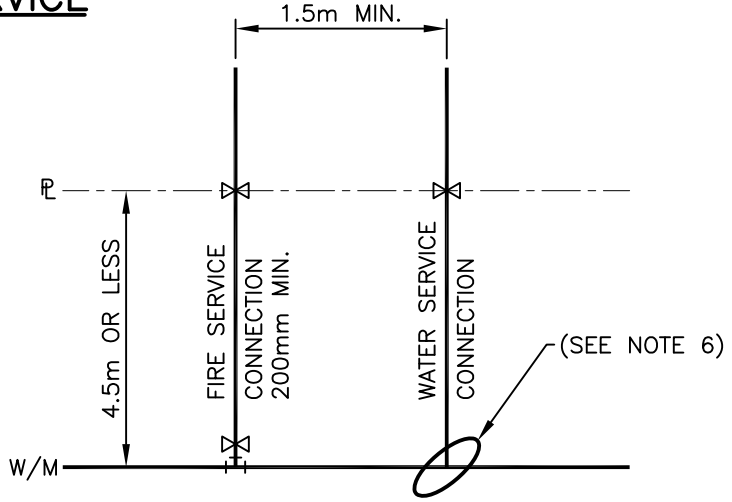
Date: July 2019	Rev. 0	NTS

PRECAST VALVE CHAMBERS FOR MAXIMUM 3 VALVES 150mm TO 300mm

REGION STANDARD RH 402.080



GREATER THAN 4.5m SERVICE



LESS THAN 4.5m SERVICE

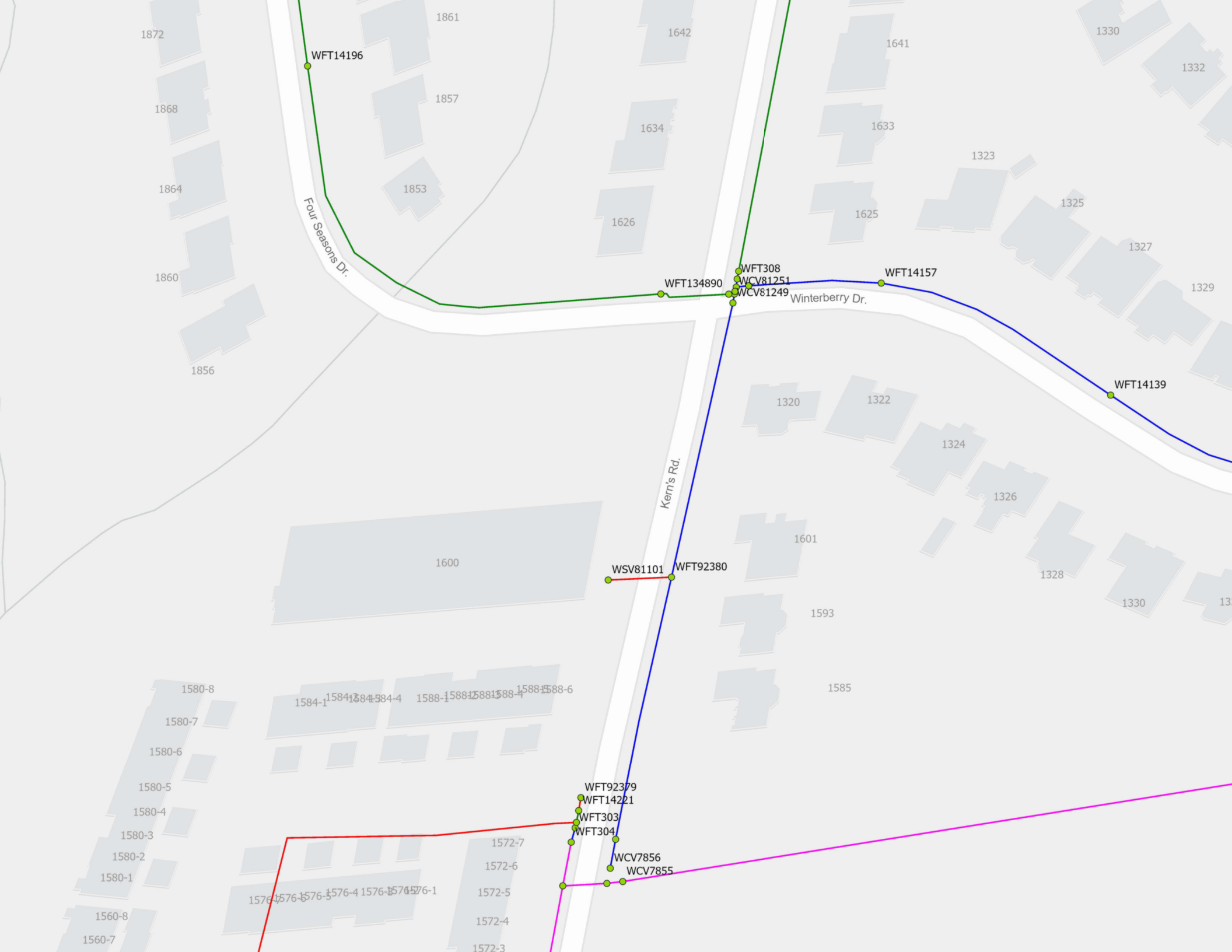
NOTES
1. COMPRESSION TYPE FITTINGS ONLY. NO SOLDERED JOINTS ARE PERMITTED BEFORE THE WATER METER.
2. WATER SERVICE CONNECTION 25, 38, 50 TO BE TYPE 'K' SOFT COPPER, 100 AND LARGER TO BE PVC OR DI.
3. FIRE SERVICE CONNECTION TO BE MIN. 200mm.
4. IF THE WATERMAIN IS 4.5m OR LESS FROM THE PROPERTY LINE, 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
1861
1642
1641
1330
1332
1868
1857
1634
1633
1323
1864
1853
1626
1625
1325
1327
1860
1329
1856
1320
1322
1324
1326
1601
1600
1328
1330
1593
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

5-200 Connie Cres. Concord ON L4K 1M1 Phone 416-883-9777 Fax 905-303-6977

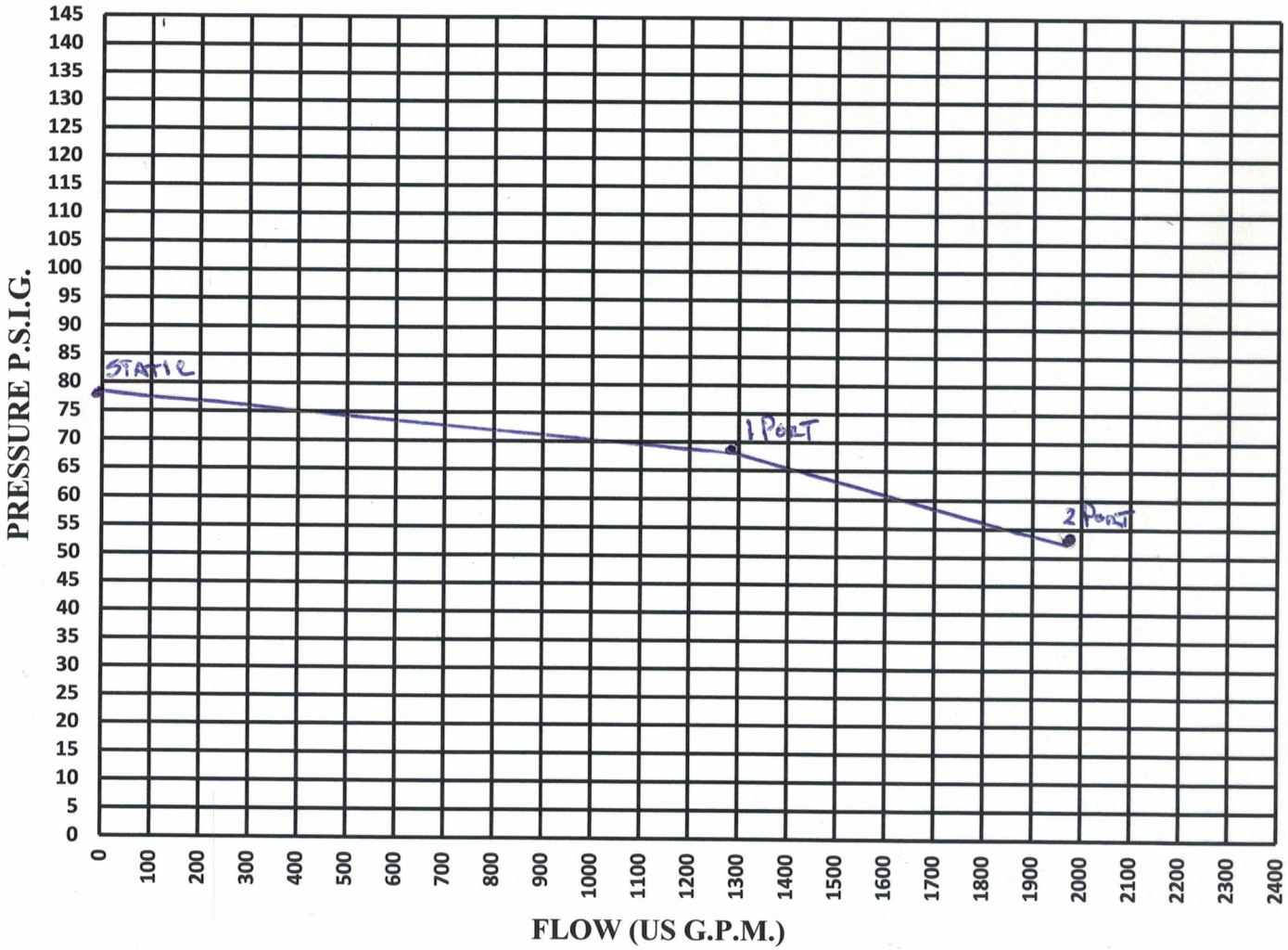
FLOW TEST REPORT

LOCATION OF RESIDUAL HYDRANT Corner of Four Seasons & KERNS McAuley 3-port

LOCATION OF FLOW HYDRANT Infront of 1548 & 1588 KERNS RD McAuley 3-port

TIME OF TEST 2:10 pm WATERMAIN SIZE 250 mm STATIC PRESSURE 78

NUMBER OF OUTLETS	PITOT PRESSURE	FLOW (US G.P.M.)	RESIDUAL PRESSURE
One 2 ½" hydrant port	60	1297	69
Two 2 ½" hydrant port	35	1981	54



PROJECT LOCATION 1600 KERNS RD, Burlington DATE Sept 1/21

COMPANY NAME Fieldgate Development & Construction (PRINT NAME) AQUAZITION EMPLOYEE Earl Rowe (PRINT NAME)

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:	Feb-22
Prepared By:	PB

Average Daily Domestic Flow	275	litres / capita / day
Land Area	0.63	ha
Population		
Total Population	168	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	2.23	Litres / second
Extraneous Flows	0.18018	Litres / second
Extraneous Flow Rate	0.286	Litres / second / ha
Peak Flow (incl. extraneous)	2.41	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	0.48	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	163.18	mm/hr
Flow [Q=CiA/360]	Q	0.22	m^3/s
		218	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	0.55	
Site Area	A	0.15	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	163.18	mm/hr
Flow [Q=CiA/360]	Q	0.04	m^3/s
		37	l/s

AREA ID 201

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.06	0.25	0.0150
Hard Surface	0.42	0.95	0.3990
	0.48	Total	0.4140
		Divided by Total Area =	0.86

AREA ID 202

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.11	0.25	0.0275
Hard Surface	0.04	0.95	0.0380
	0.15	Total	0.0655
		Divided by Total Area =	0.44

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	0.46	1.00	10	135	116	0.046	Controlled to Kerns Road	100	0.043
201B	0.02	0.31	10		0	0.002	Uncontrolled to Kerns Road		
	0.48			135	116	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	0.46	ha
Runoff Coefficient	1.00	
AC	0.46	
Tc	10	
Time Increment	5	
Release Rate	46	l/s
Storage Required	116	m^3

Time (min)	Rainfall Intensity (mm/hr)	Storm Runoff (m3/s)	Runoff Volume (m3)	Released Volume (m3)	Storage Volume (m3)
10	163	0.21	125	27	98
15	131	0.17	151	41	110
20	111	0.14	170	55	115
25	96	0.12	185	68	116
30	86	0.11	197	82	115
35	77	0.10	208	96	112
40	71	0.09	217	109	108
45	65	0.08	225	123	102
50	61	0.08	233	137	96
55	57	0.07	240	150	89
60	53	0.07	246	164	82
65	51	0.06	252	178	74
70	48	0.06	258	191	66
75	46	0.06	263	205	58
80	44	0.06	268	219	49
85	42	0.05	272	232	40
90	40	0.05	277	246	31
95	39	0.05	281	260	21
100	37	0.05	285	273	11
105	36	0.05	289	287	2
110	35	0.04	292	301	-8

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

Inlet Capacity Analysis

Project Name: Kerns Rd
Project Number: 20500
Location: Burlington
Date: 30-Dec-21
Prepared By: PB

Rainfall Data			
Location:	Burlington	a	1281.34
Event	100	b	5.00
		c	0.76

100-yr CB Flow

Time of Concentration:	t	10	min
Runoff Coefficient:	C	1.00	
Site Area	A	0.10	ha
Intensity [i=a/(t+b)c] or [i=a*tb] if B=0	i	163.18	mm/hr
Flow [Q=CiA/360]	Q	0.045	m^3/s
		45	l/s

CB Inlet Capacity

Drain ID	Drain Catchment Area (m^2)	Runoff Coefficient	Tc (min)	Intensity (mm/hr)	Flow (m^3/s)	Drain Type	Depth of Ponding (m)	Inlet Capacity (m^3/s)	Inlet Capacity with 50% Blockage (m^3/s)	OK with 50% Blockage?
101	1000	1.00	10.00	163.2	0.045	Single OPSD 400.01	0.13	0.095	0.0475	OK

27,000 LITRE PRECAST POTABLE WATER CISTERN

WILKINSON HEAVY PRECAST LIMITED
DUNDAS, ONTARIO 1-800-263-8503
www.wilkinsonheavyprecast.com

CONSTRUCTION DETAILS *

Concrete: 35 MPa at 28 Days, 5 to 8% Air Entrainment.

Reinforcing: 10 M bars at 200 mm centres each way in walls, floor and roof.
Eight extra 10 M bars around roof access opening.
Minimum cover over reinforcing steel - 25 mm.

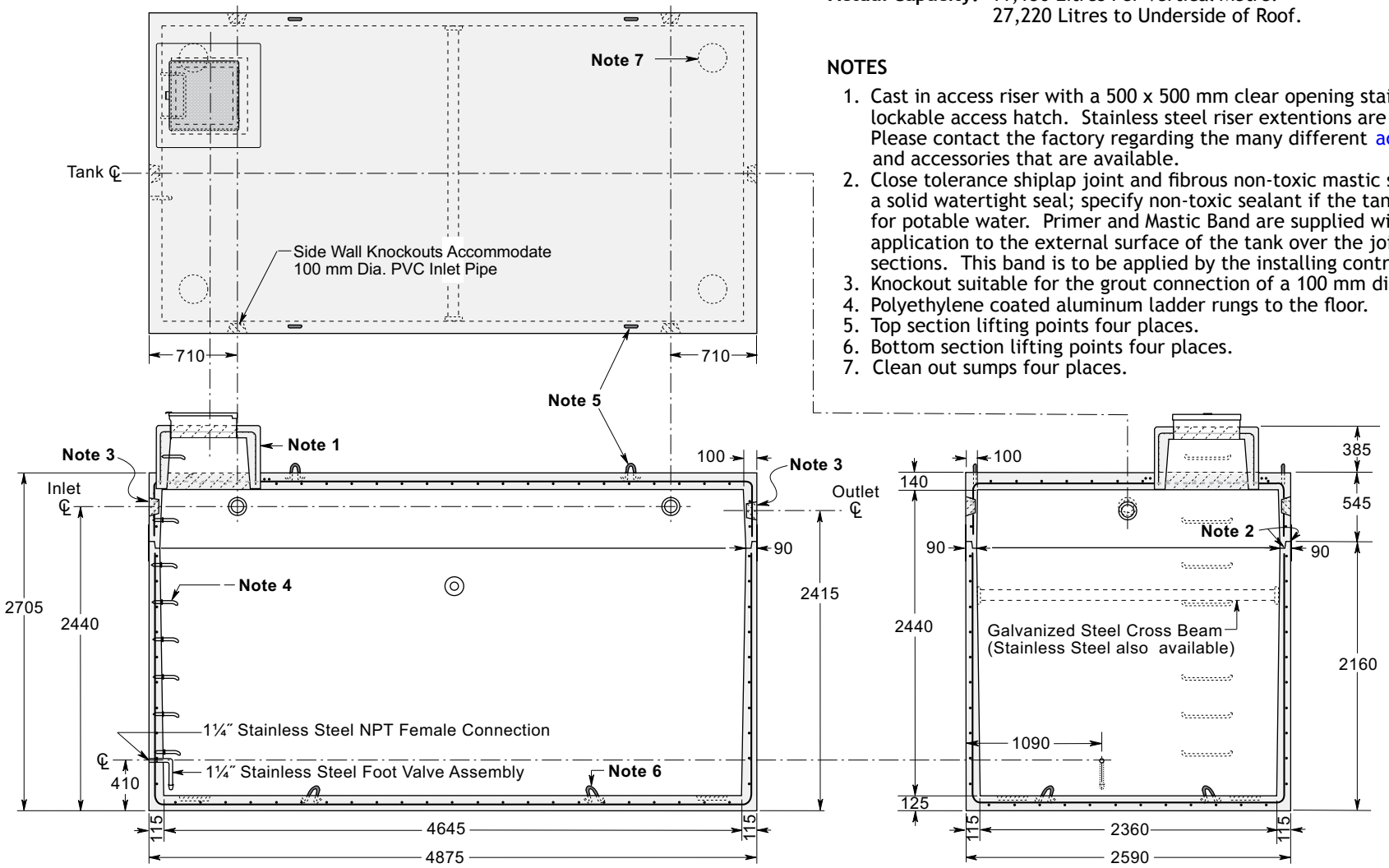
Weight:

Top Section	5760 kg
Bottom Section	11,800 kg
Total	17,560 kg

Actual Capacity: 11,156 Litres Per Vertical Metre.
27,220 Litres to Underside of Roof.

NOTES

1. Cast in access riser with a 500 x 500 mm clear opening stainless steel lockable access hatch. Stainless steel riser extentions are available. Please contact the factory regarding the many different access risers and accessories that are available.
2. Close tolerance shiplap joint and fibrous non-toxic mastic sealant ensures a solid watertight seal; specify non-toxic sealant if the tank is to be used for potable water. Primer and Mastic Band are supplied with each tank for application to the external surface of the tank over the joint between sections. This band is to be applied by the installing contractor.
3. Knockout suitable for the grout connection of a 100 mm diameter pipe.
4. Polyethylene coated aluminum ladder rungs to the floor.
5. Top section lifting points four places.
6. Bottom section lifting points four places.
7. Clean out sumps four places.



Dimensions in mm
N.T.S.

*Product designed for a **Maximum 1 Metre** burial over the top slab in firm soil away from any area of vehicular traffic.

For recommended installation procedures refer to Wilkinson Installation Guidelines.

WARNING! IMPROPER INSTALLATION ESPECIALLY IN UNSTABLE SOIL CAN RESULT IN THE STRUCTURAL FAILURE OF THIS PRODUCT

APPENDIX C

Excerpts from EXP Slope Stability Analysis

6. General Recommendations

For the long-term stability of the slope, the vegetation on the slope face should be maintained and surface water should not be discharged uncontrolled over the slope face. Discharge points down the slope should be provided with surface protections to minimize the risk of surface erosion. During construction, stockpiles of construction materials should not be placed on the slope crest (within about 10 m from the physical top of slope) or on the existing slope surface.

Based on the Civil Engineer's (KWA Site Development Consulting) level spreader design, the calculated flow velocity at the spreader outlet is about 0.003 m/s. This value is well below the critical velocity of a hard clayey silt till (0.079 m/s)[2] to resist erosion. The clayey silt till is expected near the spreader outlet at the ground surface. No erosion is anticipated to occur on the slope below the spreader outlet.

[2] Technical Guide, River & Stream Systems: Erosion Hazard Limit. Ontario Ministry of Natural Resources (2002).

APPENDIX C

Excerpts from EXP Hydrogeological and Water Balance Report

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

ADS OGS Sizing Summary

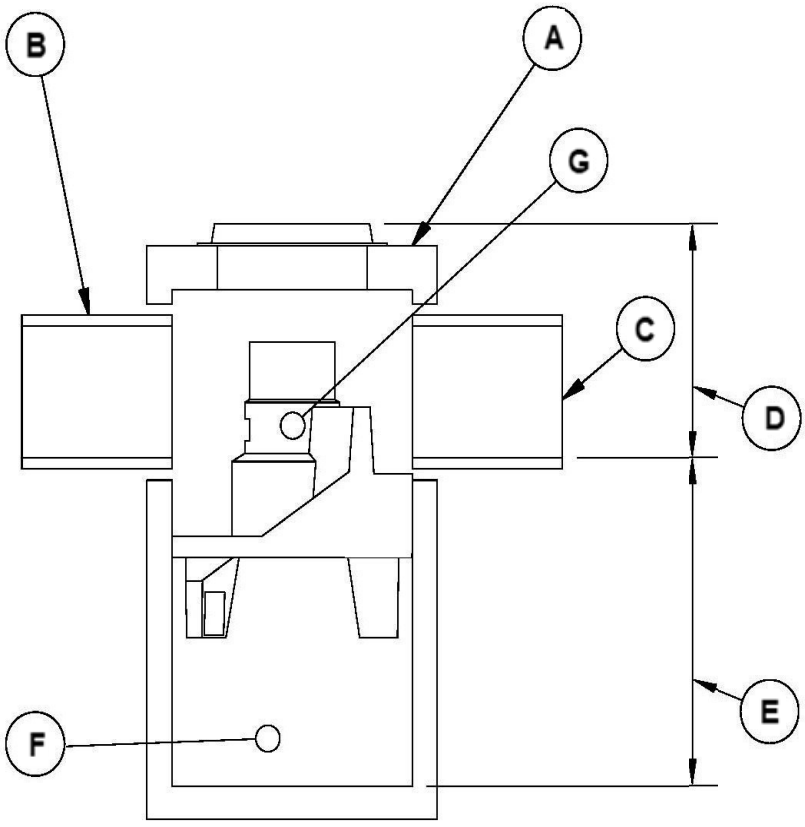
Project Name:	1600 Kerns Road - North East Parking Area
Consulting Engineer:	KWA Site Development Consulting Inc.
Location:	Burlington, Ontario
Sizing Completed By:	Haider Nasrullah
Email:	haider.nasrullah@ads-pipe.com

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 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

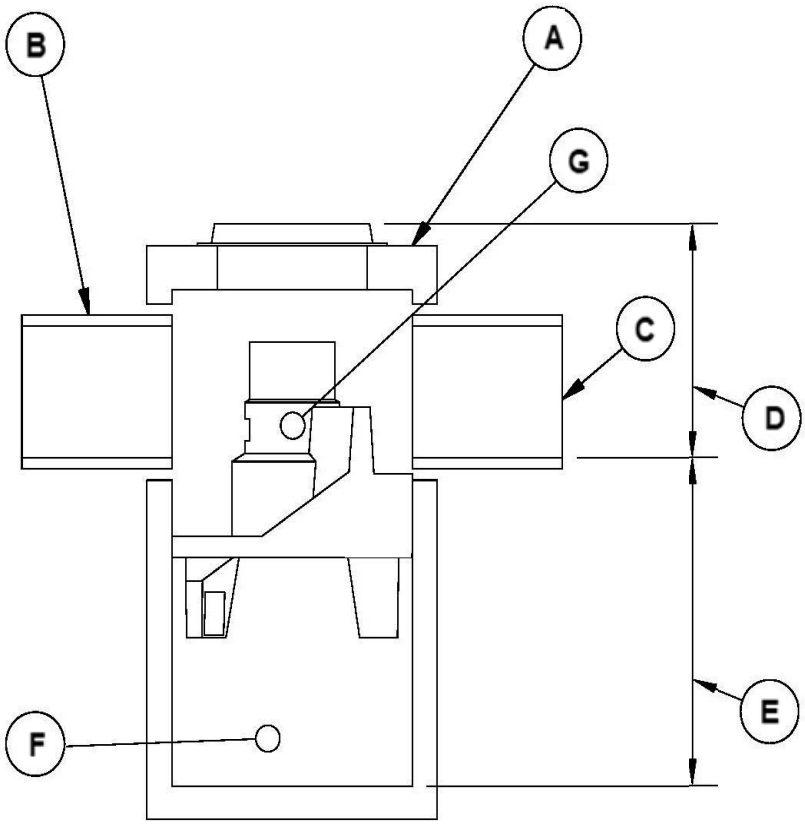
Email: haider.nasrullah@ads-pipe.com

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.

Verification Statement



Hydro International First Defense® HC Oil Grit Separator
Registration number: (V-2018-10-01)
Date of issue: 2018-October-15 (rev 2019-02-01)

Technology type	Oil Grit Separator
Application	Technology to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals
Company	Hydro International
Address	94 Hutchins Drive, Portland, Maine USA 04102 **Phone** +1-207-756 6200
Website	https://www.hydro-int.com
E-mail	dscott@hydro-int.com

Verified Performance Claims

The Hydro International First Defense® High Capacity (HC) Oil Grit Separator (OGS) was tested by Good Harbour Laboratories Inc. (GHL), Mississauga, Ontario, Canada in 2018. The performance test results were verified by Toronto and Region Conservation Authority (TRCA), Vaughan, Ontario, Canada following the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol. The following performance claims were verified:

Capture test[1]:
With a false floor set to 50% of the manufacturer's recommended maximum sediment storage depth and an influent test sediment concentration of 200 mg/L, the First Defense® HC OGS device removes 67, 60, 55, 50, 45, 45, and 41 percent of influent sediment by mass at surface loading rates of 40, 80, 200, 400, 600, 1000, and 1400 L/min/m^2, respectively.

Scour test[1]:
With 10.2 cm (4 inches) of test sediment pre-loaded onto a false floor reaching 50% of the manufacturer's recommended maximum sediment storage depth, the First Defense® HC OGS device generates adjusted effluent[2] concentrations of 0, 0, 11, 2, and 0 mg/L at 5-minute duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m^2, respectively.

[1] The claims can be applied to other units smaller or larger than the tested unit as long as the untested units meet the scaling rule specified in the Procedure for Laboratory of Testing of Oil Grit Separators (Version 3.0, June 2014)

[2] The effluent suspended sediment concentration is adjusted based on the background concentration and the smallest 5% of particles captured during the 40 L/min/m^2 sediment capture test (see Table 2)

Technology Application

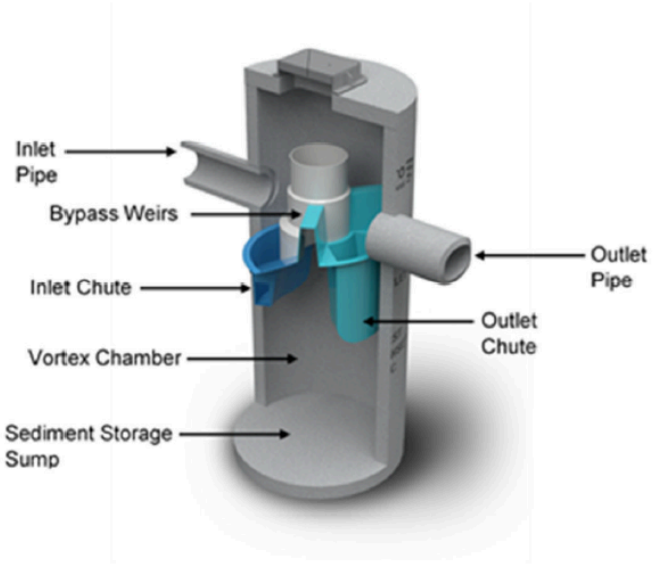
The First Defense® HC (FDHC) Oil Grit Separator can be used as a stand-alone stormwater treatment technology, depending on water quality objectives, or as a pretreatment component in a treatment train when higher TSS removals are required and polishing or volume reduction best management practices (BMPs), such as infiltration or bio-infiltration, are installed downstream. FDHC applications include: stormwater treatment at the point of entry into the drainage line; sites constrained by space, topography or drainage profiles with limited slope and depth of cover; retrofit installations where stormwater treatment is placed on or tied into an existing storm drain line; pretreatment for filters, infiltration, other sedimentation BMPs and storage.

Technology Description

The Hydro International First Defense® HC (FDHC) is an Oil Grit Separator designed to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals. The patented flow modifying internal components are designed to be inserted into standard precast concrete manholes where they collect and treat runoff as part of the drainage system (Figure 1).

Flow entering the manhole via an inlet pipe or inlet grate is diverted into a vortex chamber beneath a separation module that includes both inlet/outlet chutes and bypass weirs. The internal bypass weirs divert flows greater than the maximum design treatment flow rate over the separation module and away from the vortex chamber where oil, sediment, debris and attached pollutants are accumulating. This function prevents high velocities from re-suspending previously captured pollutants during large storm events. The FDHC can be designed and sized to function effectively in either online or offline configurations.

Figure 1: Hydro International First Defense® HC Oil Grit Separator



The test unit was 1.2 m (4 foot) in diameter with a 1.51 m (59 5/8 inches) sump depth measured from the outlet invert to the floor of the unit. The effective treatment area (also known as the effective sedimentation area) is 1.2 m^2 (12.6 ft^2). The maximum sediment storage depth is 0.457 m (18 inches).

Description of Test Procedure

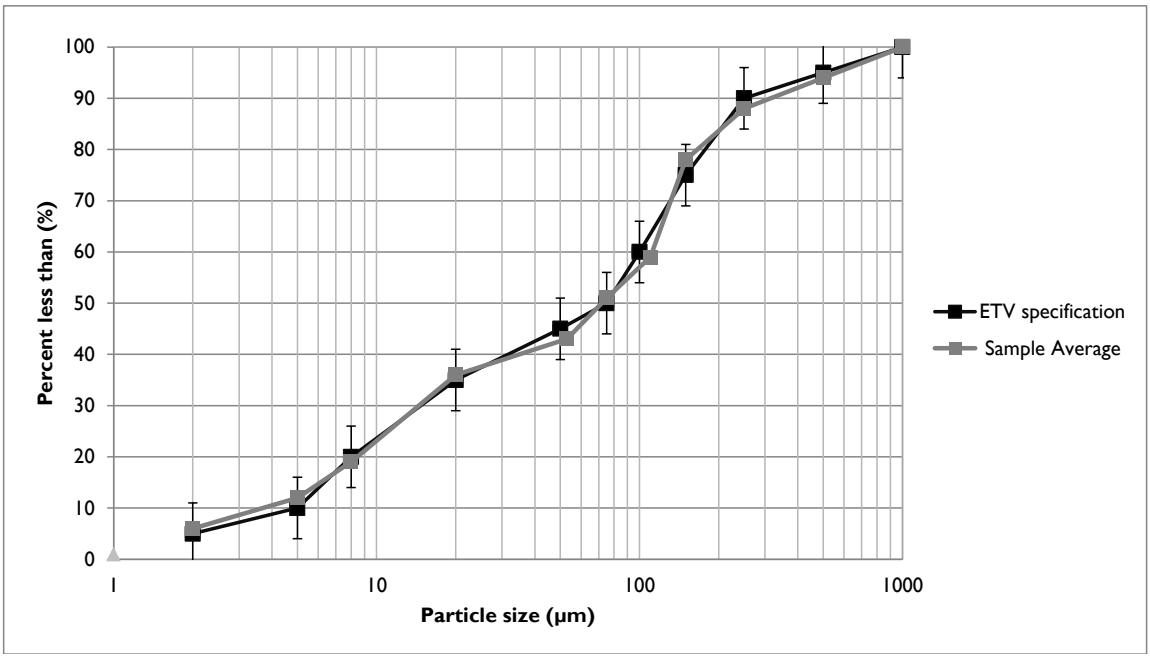
The test data and results for this verification were obtained from independent testing conducted on a 1.2 m (48 inch) diameter Hydro International First Defense® HC OGS device, in accordance with the *Procedure for Laboratory Testing of Oil-Grit Separators (Version 3.0, June 2014)*. The laboratory test procedure was originally prepared by the Toronto and Region Conservation Authority (TRCA) in association with a 31 member advisory committee from various stakeholder groups.

Verification Results

Toronto and Region Conservation Authority verified the performance test data and other information pertaining to the First Defense® HC Oil Grit Separator. A Verification Plan was prepared to guide the verification process based on the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol.

The test sediment consisted of ground silica (1 – 1000 micron) with a specific gravity of 2.65, uniformly mixed to meet the particle size distribution specified in the testing procedure. The *Procedure for Laboratory Testing of Oil Grit Separators* requires that the three sample average of the test sediment particle size distribution (PSD) meet the specified PSD percent less than values within a boundary threshold of 6%, and a median particle size no greater than 75 µm. Comparison of the individual sample and average test sediment PSD to the specified PSD shown in Figure 2 indicates that the test sediment used for the capture and scour tests met this condition. The median particle size was 73 µm. Samples from test sediment batches used for each run met the specified PSD within the required tolerance thresholds.

Figure 2 - The three sample average particle size distribution (PSD) of the test sediment used for the capture and scour test compared to the specified PSD



The capacity of the device to retain sediment was determined at seven surface loading rates using the modified mass balance method. This method involved measuring the mass and particle size distribution of the injected and retained sediment for each test run. Performance was evaluated with a false floor simulating the technology filled to 50% of the manufacturer's recommended maximum sediment storage depth. The test was carried out with clean water that maintained a sediment concentration below 20 mg/L. Based on these conditions, removal efficiencies for individual particle size classes and for the test sediment as a whole were determined for each of the tested surface loading rates (Table 1).

In some instances, the removal efficiencies were above 100% for certain particle size fractions. These discrepancies are not unique to any one test laboratory and are attributed to errors relating to the blending of sediment, collection of representative samples for laboratory submission, and laboratory analysis of PSD. Due to these errors, caution should be exercised in applying the removal efficiencies by particle size fraction for the purposes of sizing the tested device (see Bulletin # CETV 2016-11-0001). The results for "all particle sizes by mass balance" (see Table 1) are based on measurements of the total injected and retained sediment mass, and are therefore not subject to blending, sampling or PSD analysis errors.

Table 1 - Removal efficiencies (%) of the First Defence HC at specified surface loading rates

Particle size fraction (µm)	Surface loading rate (L/min/m²)						
	40	80	200	400	600	1000	1400
>500	100*	100*	100*	81	72	86	80
250 - 500	100*	97	99	100*	100*	59	88
150 - 250	100*	91	95	93	47	100*	84
105 - 150	96	89	94	89	90	70	75
75 - 105	100*	90	95	77	-20**	100	51
53 - 75	74	100*	97	62	100*	46	37
20 - 53	60	33	10	5	4	0	0
8 - 20	29	16	8	3	3	1	1
5 – 8	8	5	8	4	4	4	3
<5	5	3	0	0	0	3	3
All particle sizes By mass balance	**66.5**	**59.9**	**55.4**	**50.2**	**44.9**	**45.2**	**40.5**

* Removal efficiencies were calculated to be above 100%. Calculated values ranged between 101 and 184% (average 115%). See text and Bulletin # CETV 2016-11-0001 for more information.
** An outlier in the retained sediment sample sieve data resulted in negative removal for this size fraction. The outlier at the 75 um particle size is shown in Figure 3.

Figure 3 - Particle size distribution of sediment retained in the First Defense HC in relation to the injected test sediment average

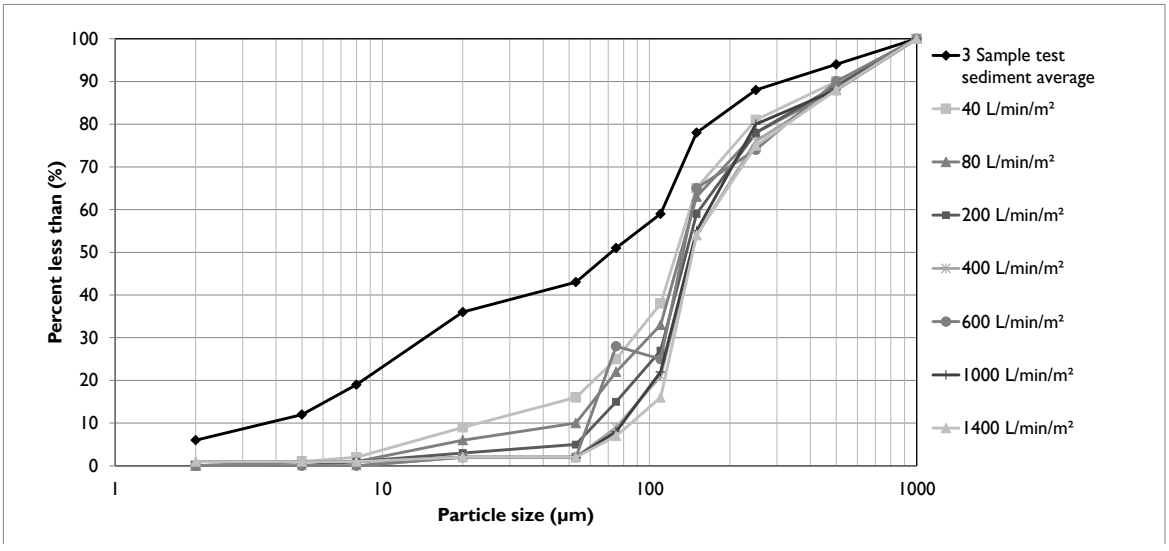


Figure 3 compares the particle size distribution (PSD) of the three sample average of the test sediment to the PSD of the sediment retained by the FDHC device at each of the tested surface loading rates. As expected, the capture efficiency for fine particles was generally found to decrease as surface loading rates increased, particularly in the 40 to 400 L/min/m² range.

Table 2 shows the results of the sediment scour and re-suspension test for the First Defense® HC unit. The scour test involved preloading 10.2 cm (4 inches) of fresh test sediment into the sedimentation sump of the device. The sediment was placed on a false floor to mimic a device filled to 50% of the maximum recommended sediment storage depth. Clean water was run through the device at five surface loading rates over a 30 minute period. Each flow rate was maintained for 5 minutes with a one minute transition time between flow rates. Effluent samples were collected at one minute sampling intervals and analyzed for Suspended Sediment Concentration (SSC) and PSD by recognized methods. The effluent samples were subsequently adjusted based on the background concentration of the influent water. The smallest 5% of particles captured during the 40 L/min/m^2 sediment capture test (13.5 μm in this case) was used to further adjust the effluent sediment concentrations, as per the method described in Bulletin # CETV 2016-09-0001. Results showed average adjusted effluent sediment concentrations below 11 mg/L at all surface loading rates. Effluent concentrations would be expected to decrease at higher flow rates since bypass over the insert bypass weirs was observed to begin at 1,032 L/min/m^2.

Table 2 - Scour test adjusted effluent sediment concentration at each surface loading rate

Run	Surface loading rate (L/min/m^2)	Run time (min)	Background sample concentration (mg/L)	Average adjusted effluent suspended sediment concentration (mg/L)*
1	200	1:00 – 6:00	0.8	0
2	800	7:00 – 12:00	1.0	0
3	1400	13:00 – 18:00	1.1	10.6
4	2000	19:00 – 24:00	2.8	2.4
5	2600	25:00 – 30:00	6.6	0

*The effluent suspended sediment concentration is adjusted based on the background concentration and the smallest 5% of particles captured during the 40 L/min/m^2 sediment capture test, as per the method described in Bulletin # CETV 2016-09-0001.

Variances from the Procedure

Minor variances from the *Procedure for Laboratory Testing of Oil-Grit Separators* used as the basis of testing for this verification were as follows:

1. The *Procedure* states that the tested device "must be a full scale commercially available device with the same configuration and components as would be typical for an actual installation." The unit tested for this verification had the same internal components as would be typical for a commercial installation, but the internal components were placed inside a structure constructed of composite materials, rather than a manhole made of concrete, the latter of which is typical for most installations. The dimensions of the structure were the same as would have been the case had the manhole been concrete. The use of alternate materials for the structure was not believed to significantly affect system performance.

2. As part of the capture test, evaluation of the 40 and 80 L/min/m^2 surface loading rate was split into 3 and 2 parts, respectively. The test was conducted in parts because of the long duration (i.e. over 10 hours) needed to feed the required minimum 11.3 kg of test sediment into the unit. At the end of the first and second parts of the test, the flow rates were gradually decreased to prevent capture of particles that would have been washed out under normal circumstances. The requirement to split the test into parts was not anticipated in the *Procedure for Laboratory Testing of Oil-Grit Separators*, but has been a common feature of testing at the 40 L/min/m^2 surface loading rate. Conducting the test in two parts for the 80 L/ min/m^2 surface loading rate is less common. The testing did not assess the significance of the breaks, however, the test laboratory and verifier do not believe that the breaks significantly affected the test results.

3. During the sediment scour test, the flow rate coefficient of variation (COV) at the 200 L/min/m^2 surface loading rate of 0.045 slightly exceeded the target COV of 0.04. The average flow rate during the test remained within ±10% of the target flow rate.

Quality assurance

Performance testing and verification of the First Defense® HC Oil Grit Separator were performed in accordance with the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol. The verifier, Toronto and Region Conservation Authority, has confirmed that quality assurance requirements were addressed throughout the performance testing process and in the generation of performance test results. This includes reviewing all data sheets and data downloads, as well as overall management of the test system, quality control and data integrity.

Verification Summary

In summary, the First Defense® HC Oil Grit Separator is designed to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals. Verification of performance claims for the Hydro International First Defense® HC Oil Grit Separator was conducted by Toronto and Region Conservation Authority based on independent third-party performance test results provided by Good Harbour Laboratories, as well as additional information provided by Hydro International. Table 3 summarizes the verification results in relation to the technology performance parameters that were identified to determine the efficacy of the First Defense® HC Oil Grit Separator.

Table 3 - Summary of Verification Results Against Performance Parameters

Performance Parameter	Verified Performance
Sediment Removal Rate	The sediment removal rate of the FDHC is dependent upon flow rate, particle density and particle size. Removal efficiency decreased with increasing surface loading rate from 67% at 40 L/min/m^2 to 41% at 1400 L/min/m^2. The weighted average removal efficiency achieved by the unit will vary depending on the rainfall distribution of the jurisdiction in which it is installed, and site characteristics.
Sediment Scour	When pre-loaded with sediment with a particle size distribution matching that of the feed sediment used in the sediment capture test, the FDHC generated effluent suspended solids concentrations of less than 11 mg/L at surface loading rates ranging from 200 to 2600 L/min/m^2.
Bypass flow rate	The flow rate at which bypass occurs will vary based on model size. For the 1.2 m (4 foot) diameter test unit, the flow rate at which bypass occurred over the insert bypass weirs was 1238 L/min (327 gpm).
Head loss	The loss of hydraulic head across the FDHC was determined by measuring the water elevation difference between the inlet and outlet sides of the insert. Head loss may vary based on model size. For the tested unit the head loss ranged from 2 mm (0.08 inches) at 93.5 L/min (12.3 gpm) to 100 mm (3.94 inches) at 1238 L/min (327 gpm) when bypass was observed to occur. At 327 gpm, when bypass occurred, the depth of the water was 177 mm upstream and 77 mm downstream for a difference of 100 mm (3.94 inches). The highest water elevation difference was 111mm (4.37 inches) at a flow rate of 1635 L/min (431.8 gpm), after which head loss declined up to the maximum measured flow rate of 3036 L/min (801.9 gpm).

What is ISO 14034?

The purpose of environmental technology verification is to provide a credible and impartial account of the performance of environmental technologies. Environmental technology verification is based on a number of principles to ensure that verifications are performed and reported accurately, clearly, unambiguously and objectively. The International Organization for Standardization (ISO) standard for environmental technology verification (ETV) is ISO 14034, which was published in November 2016.

Benefits of ETV

ETV contributes to protection and conservation of the environment by promoting and facilitating market uptake of innovative environmental technologies, especially those that perform better than relevant alternatives. ETV is particularly applicable to those environmental technologies whose innovative features or performance cannot be fully assessed using existing standards. Through the provision of objective evidence, ETV provides an independent and impartial confirmation of the performance of an environmental technology based on reliable test data. ETV aims to strengthen the credibility of new, innovative technologies by supporting informed decision-making among interested parties.

For more information on the First Defense® HC Oil Grit Separator, contact:	For more information on VerifiGlobal, contact:
Hydro International 94 Hutchins Drive, Portland, Maine USA 04102 t +1-207-756 6200 e: dscott@hydro-int.com w: www.hydro-int.com	VerifiGlobal c/o ETA-Danmark A/S Göteborg Plads 1, DK-2150 Nordhaven t +45 7224 5900 e: info@verifiglobal.com w: www.verifiglobal.com
Signed for Hydro International: *Original signed by:* *David Scott* David Scott Technical Product Manager, Americas Stormwater	Signed for VerifiGlobal: *Original signed by:* *Thomas Bruun* Thomas Bruun, Managing Director *Original signed by:* *John Neate* John Neate, Managing Director

NOTICE: Verifications are based on an evaluation of technology performance under specific, predetermined operational conditions and parameters and the appropriate quality assurance procedures. VerifiGlobal and the Verification Expert, Toronto and Region Conservation Authority, make no expressed or implied warranties as to the performance of the technology and do not certify that a technology will always operate as verified. The end user is solely responsible for complying with any and all applicable regulatory requirements. Mention of commercial product names does not imply endorsement.

VerifiGlobal and the Verification Expert, Toronto and Region Conservation Authority, provide the verification services solely on the basis of the information supplied by the applicant or vendor and assume no liability thereafter. The responsibility for the information supplied remains solely with the applicant or vendor and the liability for the purchase, installation, and operation (whether consequential or otherwise) is not transferred to any other party as a result of the verification.

APPENDIX D

Excerpts from EXP Hydrogeological and Water Balance Report

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 sewer use groundwater sample 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. However, the City of Burlington's storm sewer use by law parameters is currently under review with the city and additional testing will be completed to include these additional parameters.

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

The analytical results of the groundwater samples collected as part of Phase Two ESA compared with Table 1 SCS, concentration of some metals (Sb, Se, B, Mo, and U) reported exceedances. A Risk Assessment is planned to assess of the risk due to the contamination at the site and it is our understanding that some contamination will remain on site. Therefore, treatment will be required for construction and post construction to ensure compliance with the sewer use by law.

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.

It is recommended to discharge groundwater effluent from under-drain system to the storm sewer system of City of Burlington after treatment. Additional water sampling and analysis to compare groundwater quality with the Storm Sewer Use By-Law including the new list of parameters currently under review and applying suitable pre-treatment will be required prior to directing any groundwater discharge to the storm sewer system. A treatment system associated with an Environmental Compliance Approval (ECA) will be required to treat any exceedances of the storm sewer use by law.

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).

APPENDIX E

Water Balance

Excerpts from EXP Hydrogeological 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.

5.2.1 Impact of Water Taking on Hager Creek Baseflow

The sub-watershed area and the perimeter of the Upper Hager Creek are approximately 1,300,000 m^2 and 5,900 m, respectively. The average runoff rate sub-watershed is 260 mm/year (1,126,667 L/day and 13.04 L/sec) and the average infiltration is 100 mm (361,111 L/day and 4.18 L/sec).

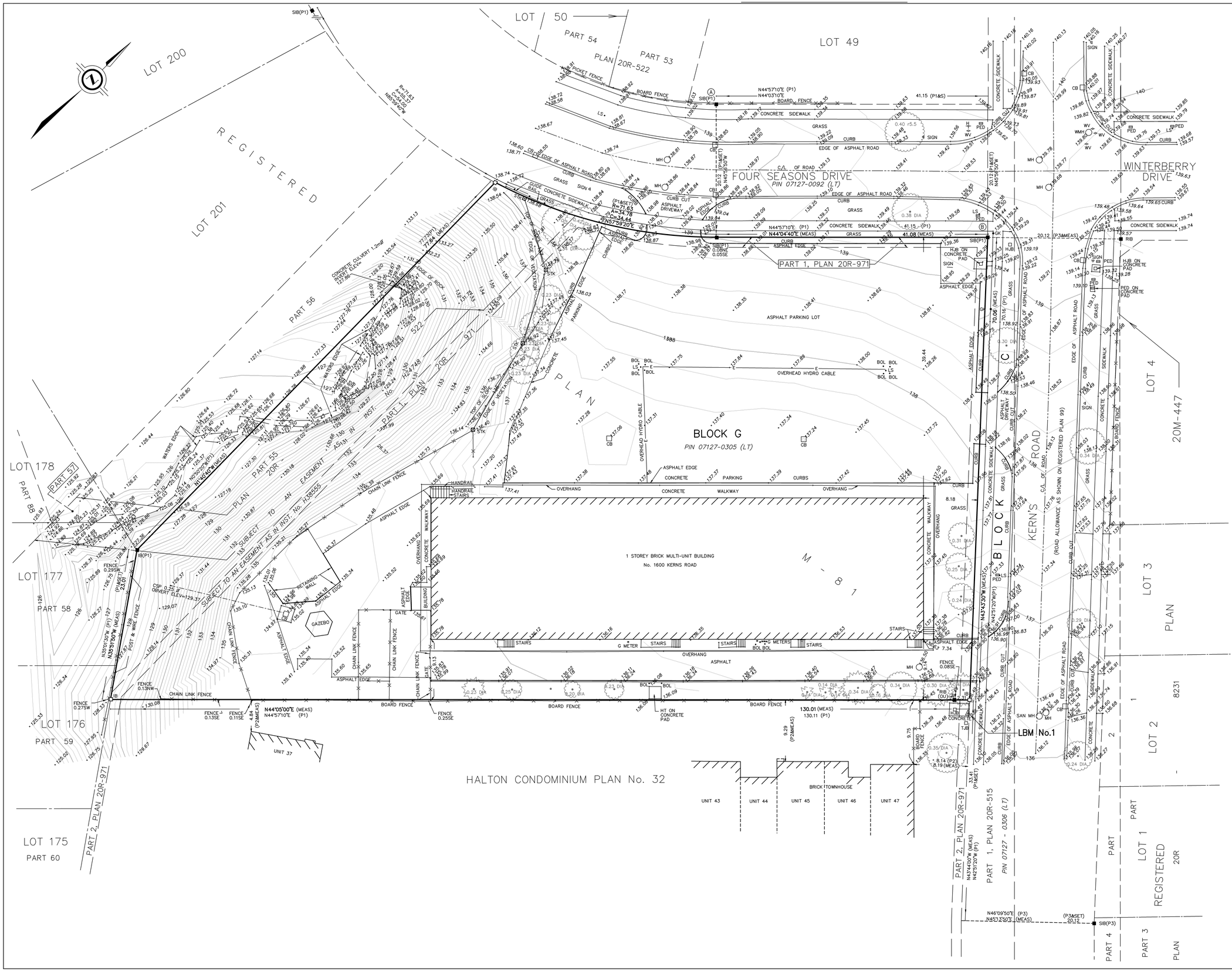
The estimated maximum long-term (under-drain) permanent dewatering rate (under-flow drain) from the proposed building is 40,000 L/day (0.46 L/sec) with a factor of safety and 25,000 L/day (0.29 L/sec) without a factor of safety. The runoff and the infiltration throughout the sub-watershed contribute to the base flow of the Upper Hager Creek, the permanent dewatering

rate for the proposed building is approximately 2.7% with factor of safety and 1.7% without factor of safety of the total contribution for the baseflow, which is considered negligible (Appendix F-5).

It is recommended to discharge groundwater effluent from under-drain system to the storm sewer system of City of Burlington after treatment. Additional water sampling and analysis to compare groundwater quality with the Storm Sewer Use By-Law including the new list of parameters currently under review, and applying suitable pre-treatment will be required prior to directing any groundwater discharge to the storm sewer system. A treatment system associated with an Environmental Compliance Approval (ECA) will be required to treat any exceedances of the storm sewer use by law.

APPENDIX F

Drawings



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:

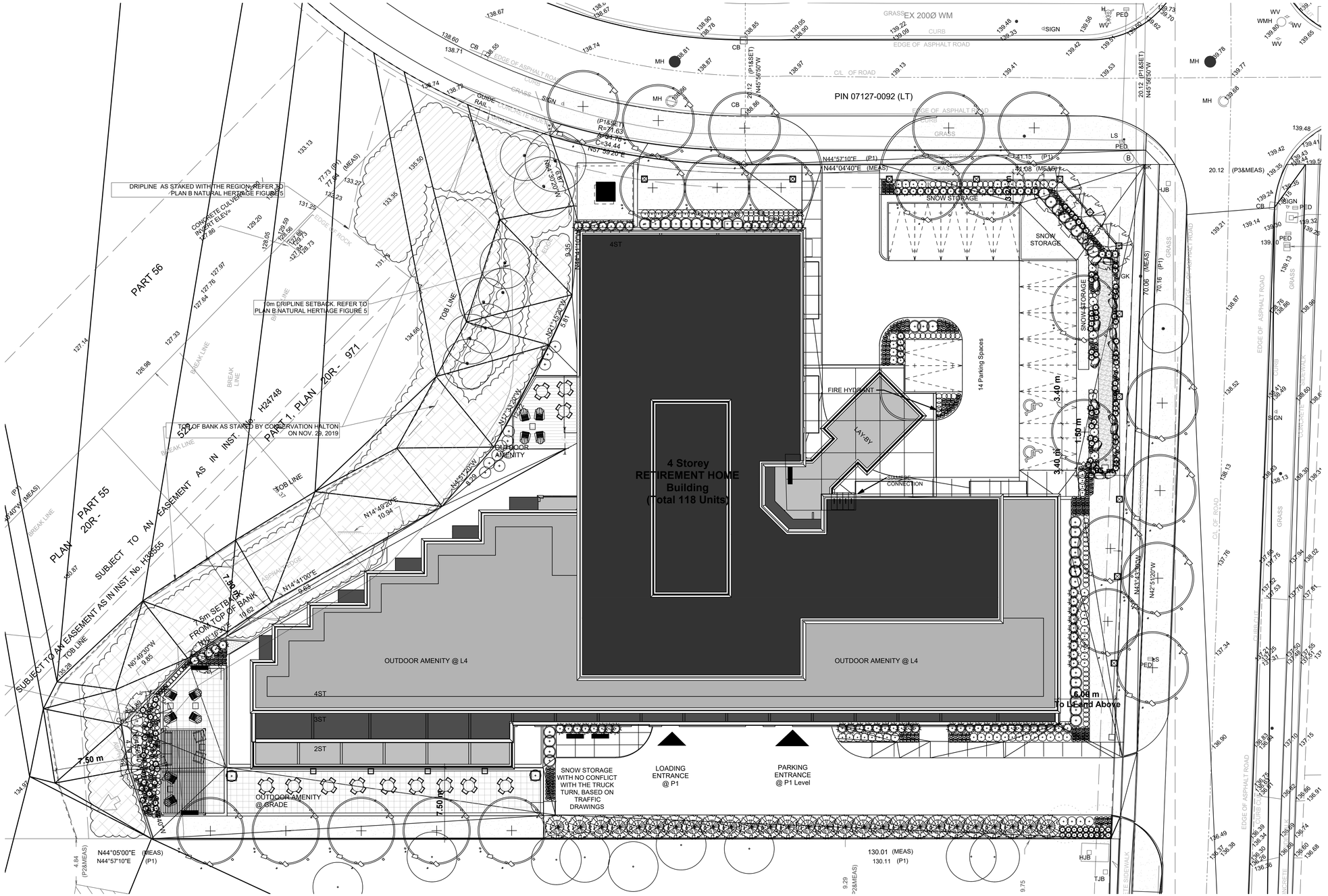
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON SEPTEMBER 13, 2018.

DATE

THOMAS J. SALB
ONTARIO LAND SURVEYOR

J.D. BARNES LIMITED
LAND INFORMATION SPECIALISTS
SURVEYING MAPPING GIS
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

v. 2021.11.03

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Do not scale the drawings.

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

Date:

V:\2021\19062\CAD\Revit Files\Issue Recording\19062 - Feb25,2022-CAD-ZBA.RTC Model Package for 1600Kerns_Ryan Jackson's Login 10:09:54 PM

Revisions:

No.:	Revision:	Date:
02	Rezoning Re-Submission	Feb 25, 2022
01	Rezoning	Mar 30, 2021

Drawing Title:

Site Plan

Client:

FIELDGATE PROPERTIES

Project:

1600 KERNS RETIREMENT HOME

1600 KERNS ROAD, BURLINGTON, ON

Scale: 1 : 200

Drawn by: Author

Checked by: Checker

Project No.: 19062

Date: Feb 25,2022

Drawing No.:

dA1.03



LEGEND

102 / A= 0.26 / C= 0.72	CATCHMENT AREA ID
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	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
2.	02/25/22	RE-ISSUED FOR APPROVAL

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

NORTH ARROW

METRIC SCALE
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1:300

EXISTING DRAINAGE PLAN

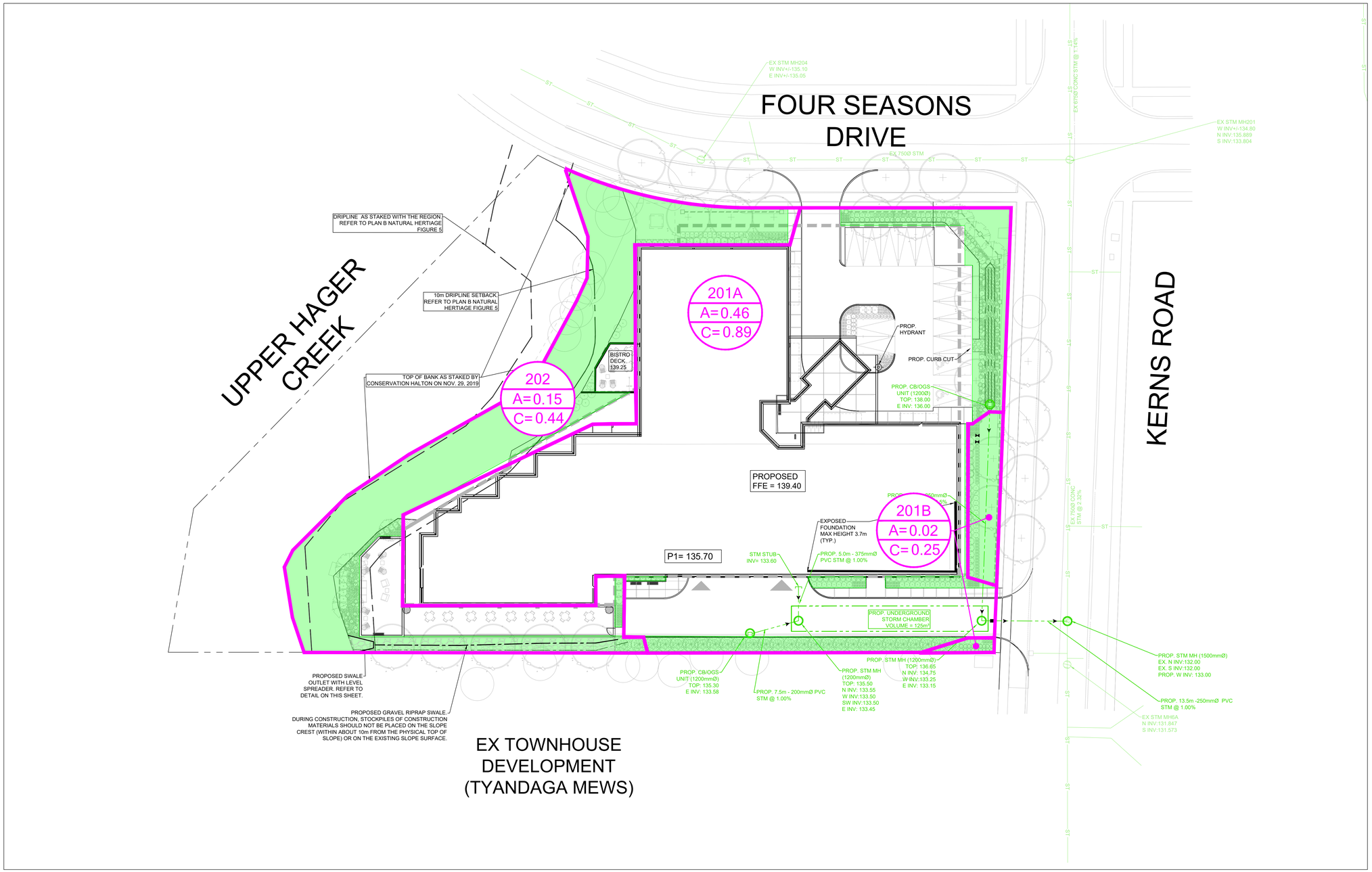
RETIREMENT HOME DEVELOPMENT

1600 KERNS ROAD

BURLINGTON, ONTARIO

PROJECT No.20500 | DRAWN BY:BC2 | CHECKED BY:BC

EDP



LEGEND

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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

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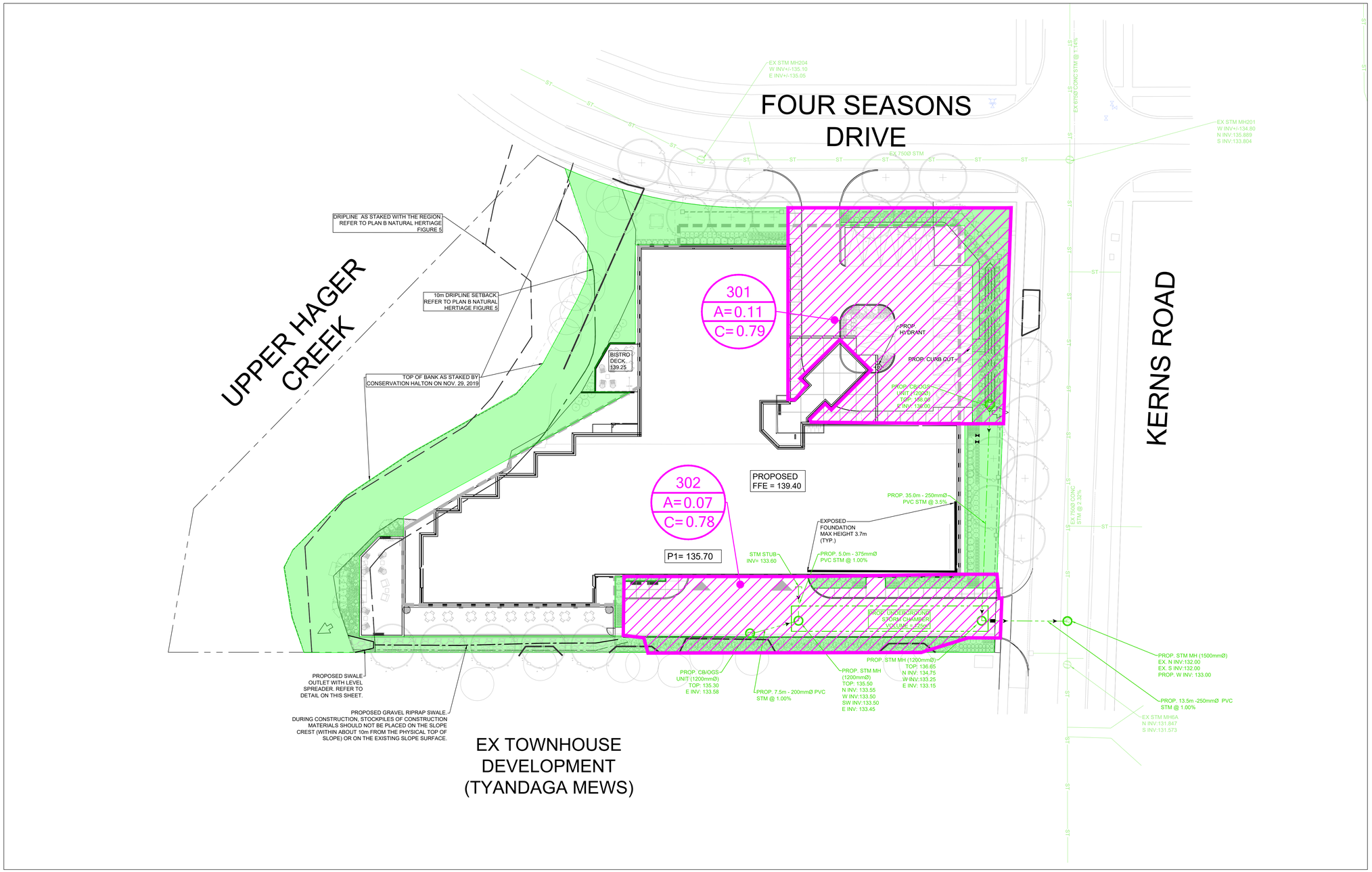
KWA SITE DEVELOPMENT CONSULTING INC.
2400 Lakeshore Road West
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE
0 9 Metres
1:300

PROPOSED DRAINAGE PLAN
RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO
PROJECT No:20509 | DRAWN BY:RCz | CHECKED BY:BC

PDP



LEGEND

301
A= 0.04
C= 0.95

CATCHMENT AREA ID
CATCHMENT AREA(ha)
RUNOFF COEFFICIENT

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
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2.	02/25/22	RE-ISSUED FOR APPROVAL

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

NORTH ARROW

METRIC SCALE
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1:300

PROPOSED QUALITY CONTROL PLAN

RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO

PROJECT No.20500 | DRAWN BY:BC2 | CHECKED BY:BC

PQCP



LEGEND

402 / A=0.49	CATCHMENT AREA ID / CATCHMENT AREA(ha)
	PROPOSED SWALE
× 135.70	PROPOSED PAVEMENT ELEVATION
× 135.55(EX)	PROPOSED ELEVATION TO MATCH EXISTING ELEVATION
	PROPOSED OVERLAND FLOW ARROW
	PROPOSED DRAINAGE AREA DIVIDE
	PROPOSED LANDSCAPE AREA
	PROPOSED UNDERGROUND PARKING LIMIT
× 135.31SW)	PROPOSED SWALE ELEVATION
× 135.75TC)	PROPOSED TOP OF CURB ELEVATION

REVISION BLOCK		
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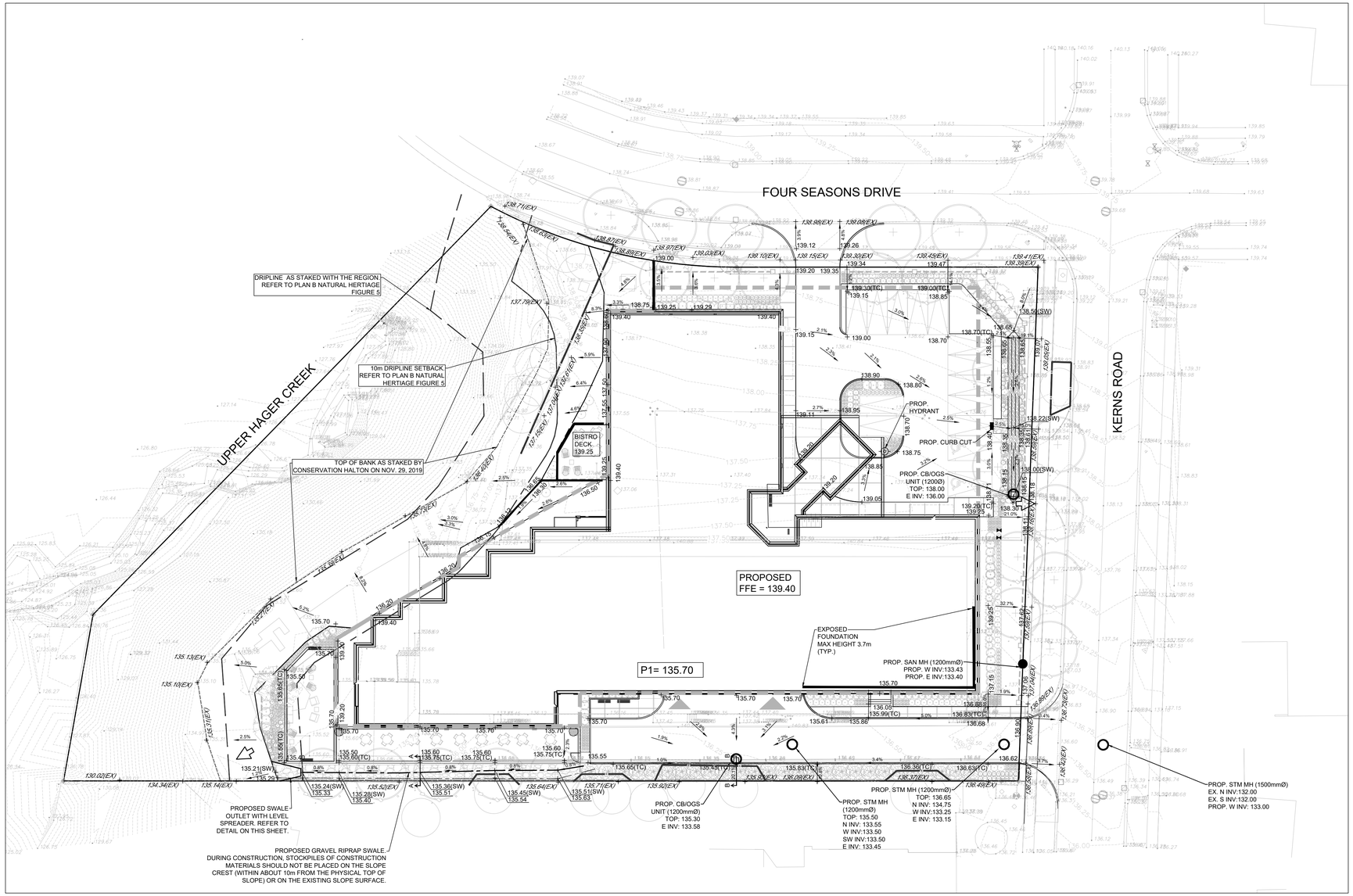
KWA SITE DEVELOPMENT CONSULTING INC.
2451 Lancaster Drive
Burlington, ON L7L 7A9
Burlington Woodstock Vancouver

NORTH ARROW

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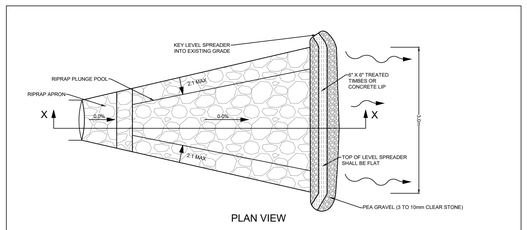
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BURLINGTON, ONTARIO
PROJECT No:20500 | DRAWN BY:BC2 | CHECKED BY:BC

PEODP

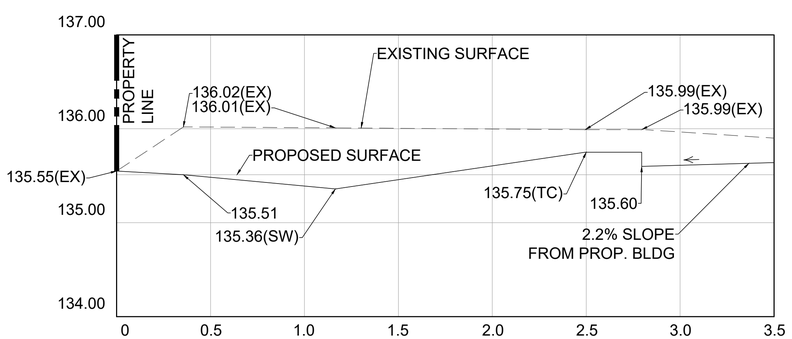


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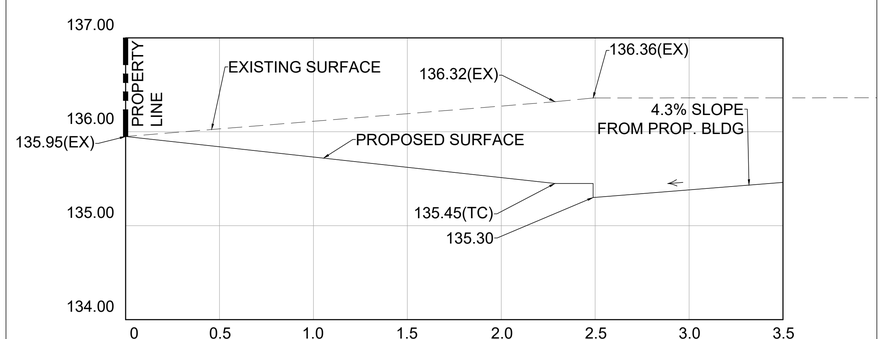
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- × 100.00(EX) — PROPOSED ELEVATION TO MATCH EXISTING ELEVATION
- × 100.00(SW) — PROPOSED SWALE ELEVATION
- × 100.00(TC) — PROPOSED TOP OF CURB ELEVATION
- PROPOSED CATCHBASIN MANHOLE
- EXISTING STORM MANHOLE
- EXISTING CATCHBASIN
- PROPOSED SANITARY MANHOLE
- EXISTING SANITARY MANHOLE
- EXISTING HYDRANT
- EXISTING VALVE AND BOX
- EXISTING CONTOUR
- DIRECTION OF EMERGENCY OVERLAND FLOW
- PROPOSED SWALE BOTTOM
- PROPOSED TOP OF SWALE
- PROPOSED UNDERGROUND PARKING LIMIT
- PROPOSED TREE PRESERVATION FENCING LOCATION (REFER TO TREE INVENTORY AND PRESERVATION PLAN PREPARED BY JACKSON ARBORICULTURE INC. DATED MARCH 2021)
- PROPOSED GRAVEL RIPRAP AREA
- PROPOSED HYDRANT



PROPOSED SWALE OUTLET WITH LEVEL SPREADER DETAIL
N.T.S.



CROSS SECTION A-A
HORZ. SCALE=1:15
VERT. SCALE=1:30



CROSS SECTION B-B
HORZ. SCALE=1:15
VERT. SCALE=1:30

REVISION BLOCK

#	DATE	DESCRIPTION
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2.	10/28/20	ISSUED FOR COORDINATION
3.	11/03/20	ISSUED FOR COORDINATION
4.	11/26/20	ISSUED FOR COORDINATION
5.	03/26/21	ISSUED FOR APPROVAL
6.	02/25/22	RE-ISSUED FOR APPROVAL

LICENSED PROFESSIONAL ENGINEER
C. DEL BUONO
100137127
02/25/22
PROVINCE OF ONTARIO

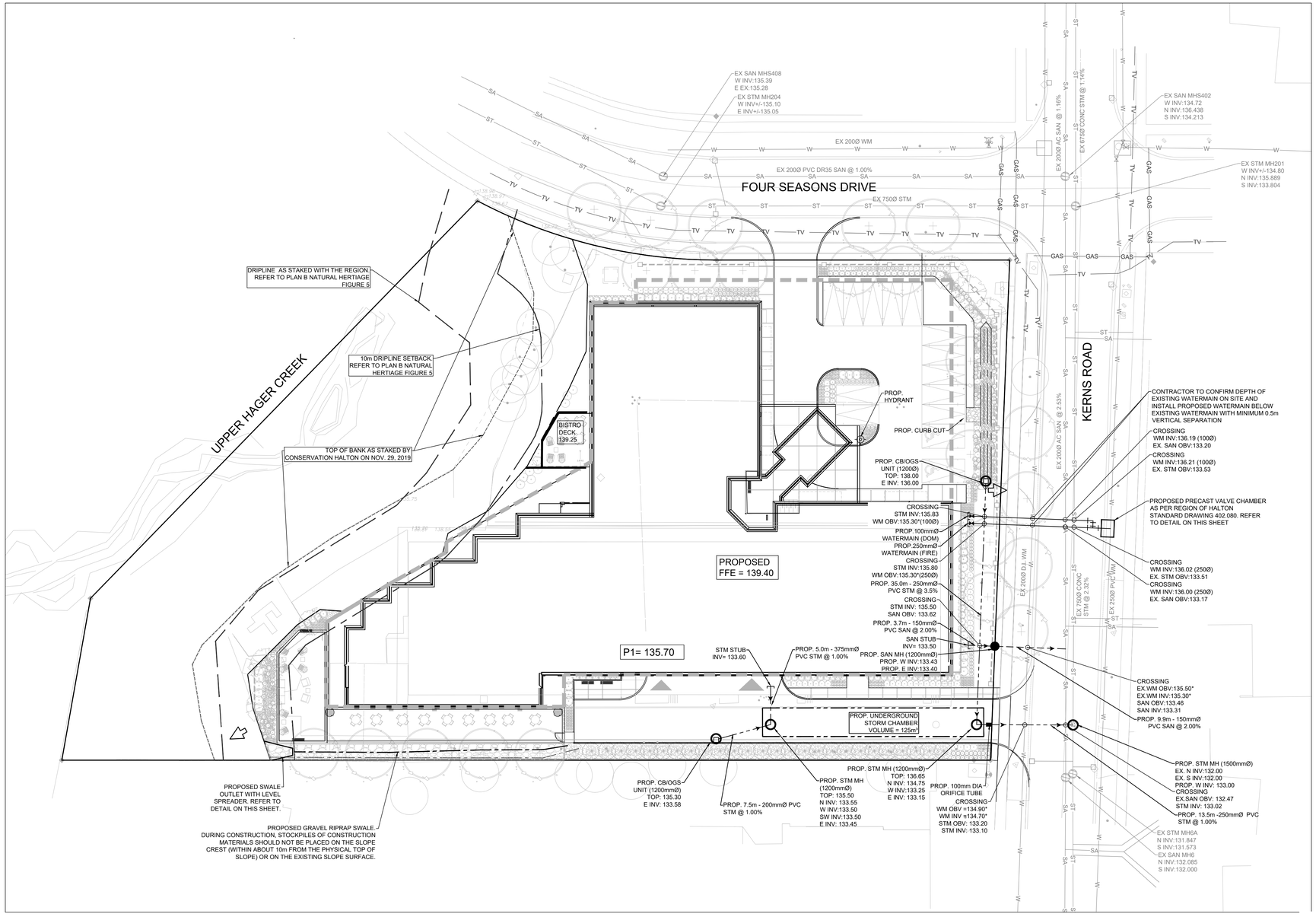
KWA SITE DEVELOPMENT CONSULTING INC.
2055 LA. Shore Rd.
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE
0 9 Metres
1:300

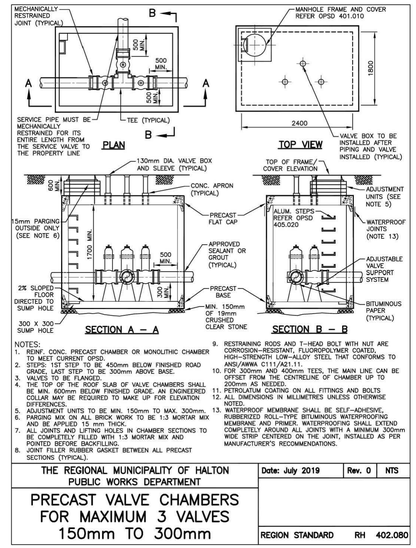
SITE GRADING PLAN
RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO
PROJECT No:20500 | DRAWN BY:BCZ | CHECKED BY:CDB

SGP



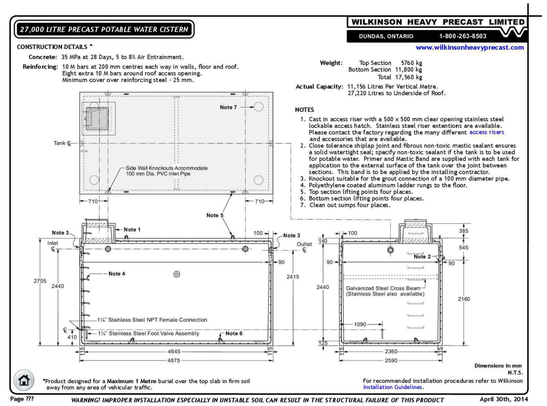
LEGEND

- PROPOSED CATCHBASIN MANHOLE
- EXISTING STORM MANHOLE
- EXISTING CATCHBASIN
- PROPOSED SANITARY MANHOLE
- EXISTING SANITARY MANHOLE
- EXISTING HYDRANT
- EXISTING VALVE AND BOX
- EXISTING STORM SEWER
- PROPOSED STORM SEWER
- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER
- PROPOSED WATERMAIN
- EXISTING WATERMAIN
- PROPOSED SWALE BOTTOM
- PROPOSED TOP OF SWALE
- PROPOSED UNDERGROUND PARKING LIMIT
- PROPOSED HYDRANT



NOTES:
1. REINF. CONC. PRECAST CHAMBER OR MONOLITHIC CHAMBER TO MEET CURRENT OPSD.
2. STEPS: 1ST STEP TO BE 450mm BELOW FINISHED ROAD GRADE, LAST STEP TO BE 300mm ABOVE BASE.
3. VALVES TO BE FLANGED.
4. THE TOP OF THE ROOF SLAB OF VALVE CHAMBERS SHALL BE MIN. 600mm BELOW FINISHED GRADE. AN ENGINEERED COLLAR MAY BE REQUIRED TO MAKE UP FOR ELEVATION DIFFERENCES.
5. ADJUSTMENT UNITS TO BE MIN. 150mm TO MAX. 300mm.
6. PARGING MIX ON ALL BRICK WORK TO BE 1:3 MORTAR MIX AND BE APPLIED 15 mm THICK.
7. ALL CONCRETE FILLETS IN CHAMBER SECTIONS TO BE CONSTRUCTED WITH 1:3 MORTAR MIX AND POINTED BEFORE BACKFILLING.
8. JOINT FILLER RUBBER GASKET BETWEEN ALL PRECAST SECTIONS (TYPICAL).
9. RESTRAINING RODS AND T-HEAD BOLT WITH NUT ARE CORROSION-RESISTANT, FLUOROPOLYMER COATED, HIGH-STRENGTH LOW-ALLOY STEEL THAT CONFORMS TO ANSI/AWWA C111/A21.11.
10. FOR 300mm AND 400mm TEES, THE MAIN LINE CAN BE OFFSET FROM THE CENTRELINE OF CHAMBER UP TO 200mm AS NEEDED.
11. PETROLATUM COATING ON ALL FITTINGS AND BOLTS
12. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
13. WATERPROOF MEMBRANE SHALL BE SELF-ADHESIVE, RUBBERIZED ROLL-TYPE BITUMINOUS WATERPROOFING MEMBRANE AND PRIMER. WATERPROOFING SHALL EXTEND FROM BASE TO TOP OF CHAMBER, A MINIMUM 300mm WIDE STRIP CENTERED ON THE JOINT, INSTALLED AS PER MANUFACTURER'S RECOMMENDATIONS.

THE REGIONAL MUNICIPALITY OF HALTON PUBLIC WORKS DEPARTMENT	Date: July 2019	Rev. 0	NTS
PRECAST VALVE CHAMBERS FOR MAXIMUM 3 VALVES 150mm TO 300mm			
	REGION STANDARD	RH	402.080



REVISION BLOCK

#	DATE	DESCRIPTION
1.	10/22/20	ISSUED FOR COORDINATION
2.	10/28/20	ISSUED FOR COORDINATION
3.	11/03/20	ISSUED FOR COORDINATION
4.	11/26/20	ISSUED FOR COORDINATION
5.	03/26/21	ISSUED FOR APPROVAL
6.	02/25/22	RE-ISSUED FOR APPROVAL

LICENSED PROFESSIONAL ENGINEER
C. DEL BUONO
100137127
02/25/22
PROVINCE OF ONTARIO

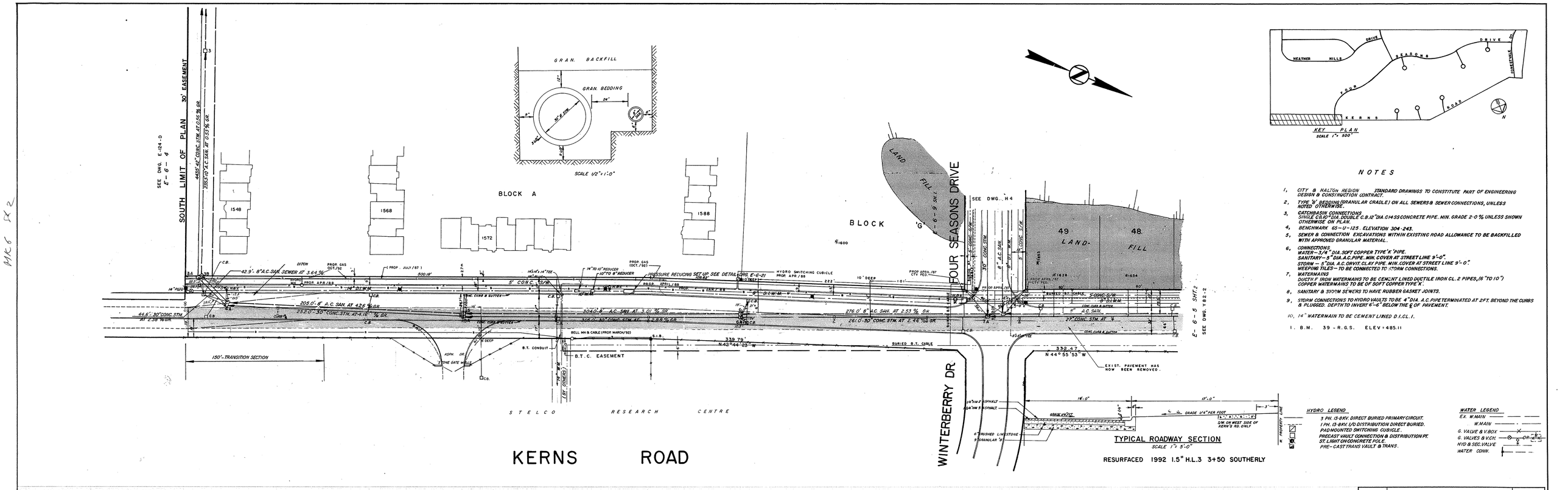
KWA SITE DEVELOPMENT CONSULTING INC.
1400 North Service Rd. W.
Burlington, ON L7L 7A9

NORTH ARROW

METRIC SCALE
0 9 Metres
1:300

SITE SERVICING PLAN
RETIREMENT HOME DEVELOPMENT
1600 KERNS ROAD
BURLINGTON, ONTARIO
PROJECT No.20500 | DRAWN BY:BC2 | CHECKED BY:CDB

SSP



NOTES

1. CITY & HALTON REGION STANDARD DRAWINGS TO CONSTITUTE PART OF ENGINEERING DESIGN & CONSTRUCTION CONTRACT.
2. TYPE 'B' BEDDING (GRANULAR CRADLE) ON ALL SEWERS & SEWER CONNECTIONS, UNLESS NOTED OTHERWISE.
3. CATCHBASIN CONNECTIONS SINGLE C.B.TO 9"DIA. DOUBLE C.B.TO 12"DIA. C14 SS CONCRETE PIPE. MIN. GRADE 2.0 % UNLESS SHOWN OTHERWISE ON PLAN.
4. BENCHMARK 65-U-125. ELEVATION 304.243.
5. SEWER & CONNECTION EXCAVATIONS WITHIN EXISTING ROAD ALLOWANCE TO BE BACKFILLED WITH APPROVED GRANULAR MATERIAL.
6. CONNECTIONS WATER - 3/4" DIA. SOFT COPPER TYPE 'K' PIPE. SANITARY - 5" DIA. A.C. PIPE. MIN. COVER AT STREET LINE 9'-0". STORM - 6" DIA. A.C. OR VIT. CLAY PIPE. MIN. COVER AT STREET LINE 9'-0". WEEPING TILES - TO BE CONNECTED TO STORM CONNECTIONS.
7. WATERMAINS DUCTILE IRON WATERMAINS TO BE CEMENT LINED DUCTILE IRON CL. 2 PIPES, (6" TO 10") COPPER WATERMAINS TO BE OF SOFT COPPER TYPE 'K'.
8. SANITARY & STORM SEWERS TO HAVE RUBBER GASKET JOINTS.
9. STORM CONNECTIONS TO HYDRO VAULTS TO BE 4" DIA. A.C. PIPE TERMINATED AT 2FT. BEYOND THE CURBS & PLUGGED. DEPTH TO INVERT 5'-6" BELOW THE ℄ OF PAVEMENT.
10. 14" WATERMAIN TO BE CEMENT LINED D.I.C.L.I.

I. B.M. 39 - R.G.S. ELEV = 485.11

TYPICAL ROADWAY SECTION

SCALE 1" = 5'-0"

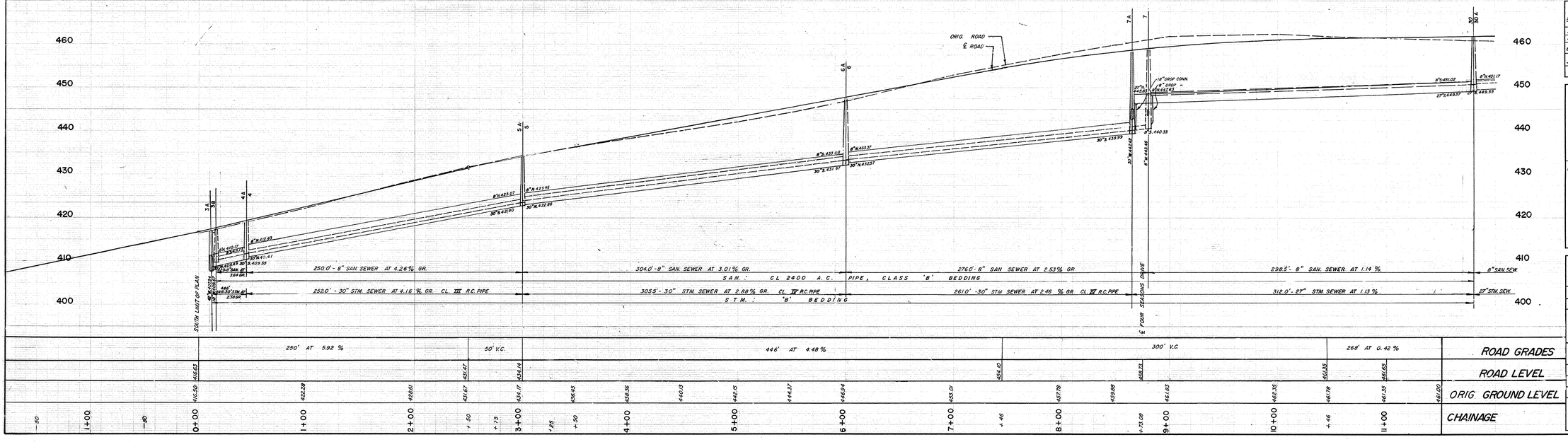
RESURFACED 1992 1.5" H.L.3 3+50 SOUTHERLY

HYDRO LEGEND

5 PH. 13.8 KV. DIRECT BURIED PRIMARY CIRCUIT.
1 PH. 13.8 KV. I/O DISTRIBUTION DIRECT BURIED.
PADMOUNTED SWITCHING CUBICLE.
PRECAST VAULT CONNECTION & DISTRIBUTION PT.
ST. LIGHT ON CONCRETE POLE.
PRE-CAST TRANS VAULT & TRANS.

WATER LEGEND

EX. W.MAIN
W.MAIN
G. VALVE & V.BOX
G. VALVES & V.C.H.
HYD & SEC. VALVE
WATER CONN.



SEPT. 1993	"AS CONSTRUCTED" RESURFACING 92-11	
AUG 1975	LINEN FINALIZED	
AUG. 3, 1973	ROAD GRADES REVISED	
APR 29/73	BLOCK 'G' ADDED.	
SEPT 2 1972.	PAVEMENT ETC. FROM FOUR SEASON DR. TO S. LIMIT OF PLAN REVISED	
JULY 1972	SEWERS IN DOWNSTREUCH APPROVED BY TOWN LETTER DATED JULY 5, 1972	
DATE	REVISION	APPROVED

APPROVALS

P.U.C. WATER H. E. Sears DATE 16-2-72

HYDRO DATE 16-2-72

DEPT. OF PUBLIC WORKS

DIRECTOR OF PUBLIC WORKS

DATE 16-2-72

250' AT 5.92 %	50' V.C.	446' AT 4.48 %
300' V.C.	268' AT 0.42 %	
ROAD GRADES		
ROAD LEVEL	416.50	416.63
ORIG. GROUND LEVEL	431.67	431.67
CHAINAGE	434.17	434.14
	456.90	458.72
	459.88	461.83
	461.78	461.85
	461.00	
	0+00	1+00
	2+00	3+00
	4+00	5+00
	6+00	7+00
	8+00	9+00
	10+00	11+00

CITY OF BURLINGTON

CHALFONTE INVESTMENTS LTD.

FORESTVALE HEIGHTS

PLAN AND PROFILE OF

KERNS ROAD

FROM SOUTH LIMIT TO 10+00

SCALE	HORIZ. 1" = 40'	DRG. NO. E-6-5
	VERT. 1" = 10'	
DATE	NOV. 1971	
DRAWN	P.K.	
CHECKED		SHEET 1 OF 4

URBAN ENGINEERING CONSULTANTS
CONSULTING ENGINEERS—TOWN PLANNERS
TORONTO

Y 82-1