



Kaitlin Corporation

Block 15 – Fields of Wellington Site Servicing & Stormwater Management Report

Prepared for:

Kaitlin Corporation

315 – 220 Duncan Mill Road
North York, ON M3B 3J5

Prepared by:

FOREFRONT Engineering Inc.

1329 Gardiners Road, Suite 210
Kingston, ON, Canada K7P 0L8

613.634.9009 tel

Date: August 2023

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August 31, 2023

Rodrick Sutherland
Land Development Manager
Kaitlin Corporation
315 -220 Duncan Mill Road
North York, ON M3B 3J5

**Regarding: Block 15 – Fields of Wellington
 Site Servicing and Stormwater Management Report**

Dear Mr. Sutherland,

The enclosed report details the infrastructure upgrades required and recommendations for the proposed development.

Two mid-rise residential buildings are proposed, the West Building and the East Building. The proposed West Building consists of a three-storey residential building with approximately 48 units and the East Building consists of a four-storey residential building with approximately 56 units for a total of 104 units. The development includes one and two-bedroom units, amenity space, and surface level parking with access from West Street.

A 150mm water service connection is proposed to West Street.

A 200mm sanitary sewer connection is proposed to the extension of West Street.

An onsite dry pond stormwater management facility including oil grit separator is proposed to provide quantity and quality control outletting south towards the Millennium Trail enhanced swale. Post-development flows are proposed to be attenuated to pre-development levels for all storm events up to the 100 year event.

Electrical, communication and natural gas connections are proposed on West Street.

The Report demonstrates that adequate servicing is available for the proposed development. If you have any enquiries or wish to discuss further, please contact this office.

Sincerely,

FOREFRONT Engineering Inc.



Jeff Homer, P.Eng.

FOREFRONT Signatures



Report Prepared By:

Jeff Homer, P. Eng.

Table of Contents

Statement of Qualifications and Limitations

Letter of Transmittal

	Page
1. Introduction	1
2. Water Supply	2
3. Sanitary Sewer System	4
4. Drainage and Stormwater Management	6
4.1 Water Quantity	7
4.2 Water Quality	10
4.3 Maintenance	11
4.4 Quality Control (Short Term)	13
5. Utilities	13
6. Conclusions	13

Appendices

Appendix A

- Architect Site Plan
- C1 - Grading Plan
- C2 - Servicing Plan
- C3 - Details
- C4 - Details

Appendix B.

- Water Demand Calculations
- Fire Flow Calculations
- MSP Water Modelling Figures A-1, A-2, A-3
- Sanitary Sewer Design Sheet

Appendix C.

- MTO IDF Curve
- Figure 2: Pre-development Catchment Areas
- Figure 3: Post-development Catchment Areas (Storm Sewers)
- Figure 4: Post-development Composite Runoff Coeff. Calculations
- Composite Runoff Coefficient Calculations
- Storm Sewer Design Sheet
- Modified Rational Method Calculations
- Outlet Stage-Storage Discharge Calculations
- CDS Estimated Net Annual Solids Load Reduction
- Outlet Swale Capacity Calculations

1. Introduction

Forefront has assembled relevant supporting information for the two mid-rise residential buildings at Block 15, located in the Fields of Wellington Subdivision in the Village of Wellington, Prince Edward County, Ontario. The property is bordered by the extension of West Street to the east, existing residential to the west, proposed multi-unit residential to the north, and open space and the Millennium Trail and to the south. Refer to Figure 1: Location Plan for the site location.



Figure 1: Location Plan

The subject area is currently zoned R3-70 – Urban Residential and is located in the Wellington Urban Centre Secondary Plan. The proposed development is approximately 1.22 hectares in area. Block 15 is proposed as part of Phase 1B of Fields of Wellington Subdivision currently under review for Final Subdivision Approval. The subject site in its current form is vacant and is formerly agricultural lands.

Two mid-rise residential buildings are proposed, the West Building and the East Building. The proposed West Building consists of a three-storey residential building with approximately 48 units and the East Building consists of a four-storey residential building with approximately 56 units for a total of 104 units. The development includes one and two-bedroom units, amenity space, and surface level parking with access from West Street.

Development of Block 15 will result in an increase in municipal water consumption, sanitary sewer discharge and stormwater runoff. This report proposes to address water, sanitary and stormwater management requirements for the proposed development.

Refer to Appendix A: **Site Plan** for the proposed development plan.

2. Water Supply

Existing Water Infrastructure

The existing subject site is currently vacant agricultural fields and is not serviced by any water supply infrastructure.

Block 15 is proposed as part of Phase 1B of Fields of Wellington Subdivision. Local infrastructure proposed as part of the Fields of Wellington Subdivision include a 200mm diameter watermain along West Street and a 200mm diameter watermain along proposed Raynor Road (Street B).

A 150mm water service is proposed to be provided from West Street extending to the property limit to serve the subject site as part of the Subdivision works.

Forefront Engineering has reviewed the “*Wellington Master Servicing Plan for Water, Wastewater and Stormwater*” (MSP), dated May 6, 2021 prepared by R.V. Anderson Associated Limited in association with the County of Prince Edward (PEC). The subject development is within the MSP study area and this Report has been prepared with consideration to the findings and recommendations of the MSP.

The MSP notes that the existing water supply distribution system prior to any upgrades currently experiences inadequate pressures in the northern parts of the existing urban area.

With consideration to the findings of the MSP, Prince Edward County (PEC) is undergoing proposed upgrades to the water supply infrastructure. Water supply upgrades to be completed by the County include the construction of a new water treatment plant (WTP) to be constructed in phases, a new elevated storage tank (EST), a 500mm diameter watermain to be constructed in 2023-2024 along the Millennium Trail and Main Street and a 400mm diameter watermain on Belleville Street.

The MSP completed a hydraulic water model of the full buildout of the Wellington Urban Centre including for the proposed Fields of Wellington Subdivision. Refer to Appendix B for further details regarding the **MSP Water Modelling**.

Proposed Development

PEC and the Ministry of the Environment, Conservation and Parks (MECP) require the system to be assessed at the critical locations including for peak hour flow, maximum day, and maximum day plus fire flow demand. MECP requires the system to maintain an operating pressure of 280-700 kPa under peak flow conditions and 138 kPa under maximum day plus fire flow conditions.

In accordance with the MSP, MECP guidelines and the Ontario Building Code (OBC), the following design criteria have been used.

Design Flow Parameters

Average Day Flow Rate – 320 L/cap/day (MSP, 2021)

Average Day Flow Rate – 350 L/day/person (PEC Request)

Peak Day Factor - 2.75

Peak Hour Factor - 4.25

Population

Multi Unit Population – 1.79 Persons/unit (MSP, 2021)

(Refer to *Wellington Master Servicing Plan – Technical Memorandum 1* for population and demand requirements)

Residential Dwelling Units

West Building – 56 Units

East Building - 48 Units

Currently local hydrant flow data is unavailable for the site.

Both mid-rise residential buildings are proposed to be equipped with a sprinkler system meeting NFPA standards. Wood frame construction is proposed for both buildings.

An onsite fire hydrant is proposed. Fire department connections are proposed approximately 45m from the onsite fire hydrant.

A single 150mm diameter water service is proposed to connect to the 200mm watermain on West Street to service the proposed development. A single service connection to each building is proposed and are to be 100mm in diameter.

It is assumed water modelling completed as part of the MSP assigned appropriate demands and fire flows to the proposed distribution systems. Refer to the **Water Supply Demand Calculations** and **Fire Flow Calculations** in Appendix B for the site demands and calculated fire flows. Fire flow demands are based on the “Water Supply for Public Fire Protection”, 2020 by Fire Underwriters Survey (FUS). Demand and fire flow calculations for the subject site are below.

The Maximum Daily Flow demand for this development is 2.07 L/s based on a peak day factor of 2.75 and the Peak Hour Flow demand is 3.21 L/s assuming a peak hour factor of 4.25. The calculated minimum recommended fire flow is 217 L/s.

Under Maximum Daily flow plus Fire Flow conditions, 219.1 L/s is the minimum recommended flow. From the MSP hydraulic modelling flow results, available fire flow during max day demand is anticipated to be in the range of 314 to 366 L/s at the minimum operating pressure of 138 kPa. Given that this is greater than the 219.1 L/s required, fire flow available to the site is considered adequate.

The mechanical engineer will be required to undertake a hydrant flow test meeting NFPA requirements to confirm available fire flow for the onsite fire suppression and sprinkler system design prior to construction.

MECP require the systems to maintain an operating pressure of 280-700 kPa (40-100 psi) under peak hour flow conditions. Based on the MSP hydraulic modelling, static and residual pressures range from 448–455 kPa (65–66 psi) which are greater than the required 280kPa (40psi).

Based on the infrastructure review and future upgrades to be completed by the County there will be no negative impact on the local water distribution system. There is adequate water pressure and flow available for the proposed development.

Refer to **Drawing C2** – Servicing Plan by Forefront Engineering for details regarding the proposed water servicing.

3. Sanitary Sewer System

Existing Sanitary Infrastructure

The existing subject site is currently vacant agricultural fields and is not serviced by any sanitary sewer infrastructure.

Block 15 is proposed as part of Phase 1B of Fields of Wellington Subdivision. Local infrastructure proposed as part of the Fields of Wellington Subdivision include a 200mm sanitary sewer along the extension of West Street and a 200mm diameter sanitary sewer along the proposed Raynor Road.

A 200mm sanitary service is to be provided within the West Street municipal right-of-way extending to the property limit to serve the subject site as part of the Subdivision works.

Wellington Urban Area Sanitary Sewer Upgrades

R.V. Anderson Associates Limited completed a “*Wellington Master Servicing Plan for Water, Wastewater and Stormwater*” (MSP), dated May 6, 2021, in association with the County of Prince Edward for the Wellington Urban Area. The MSP was undertaken as part of a Schedule “B” Class Environmental Assessment (EA). The subject development is within the MSP study area and this Report has been prepared with consideration to the findings and recommendations of the MSP.

The MSP notes that the existing Wastewater Treatment Plant (WWTP) requires several upgrades as the system capacity is near surpassing the County’s threshold of 80% of the rated capacity. Inflow and infiltration from precipitation and groundwater sources have played a major role in pushing wastewater flows above the capacity of the existing WWTP. Immediate upgrades to the WWTP are recommended per the MSP to address the current peak flow.

With consideration to the findings of the MSP, PEC is undergoing proposed upgrades to the sanitary sewer collection system. As noted above, the WWTP is proposed to be replaced in the near future. A 525mm sanitary sewer is to be completed in 2023-2024 by the County along Millennium Trail from Belleville Street to the Main Street WWTP to service future developments to the north. A new sewage pumping station on the east end and associated forcemain is proposed to service future development to the east.

Proposed Development

In accordance with the MSP, MECP guidelines and the Ontario Building Code, the following design criteria have been used.

Design Flow Parameters

Average Day Flow Rate – 320 L/day/person (MSP, 2021)
 Average Day Flow Rate – 350 L/day/person (PEC Request)
 Peak Residential Factor - Harmon Formula
 Infiltration Allowance - 0.28 L/s/ha

Population

Unit Population – 1.79 Persons/unit (MSP, 2021)
 (Refer to *Wellington Master Servicing Plan – Technical Memorandum 1* for population and demand requirements)

Residential Dwelling Units

West Building – 48 Units
 East Building - 56 Units

A 200mm diameter sanitary service is proposed to service the subject site, connecting to the 200mm sanitary sewer on West Street.

The proposed development is expected to generate a peak sanitary sewage flow in the order of 3.4 L/s. Refer to the **Sanitary Sewer Design Sheet** in Appendix B for further details.

Note that the proposed peak sanitary sewage flow is captured in the *Fields of Wellington Subdivision Servicing Report*, the Fields of Wellington Subdivision sanitary sewer design and the MSP. It is understood that with the upgrades to be completed by the County there is adequate downstream sanitary sewer capacity for the proposed development.

Refer to **Drawing C2** – Servicing Plan by Forefront Engineering for further details regarding the proposed sanitary servicing.

4. Drainage and Stormwater Management

Existing Conditions

The existing subject site is currently vacant agricultural fields and is not serviced by any stormwater infrastructure.

Block 15 is proposed as part of Phase 1B of Fields of Wellington Subdivision. Local infrastructure proposed as part of the Fields of Wellington Subdivision include a 375mm diameter to 450mm diameter storm sewer along West Street. The storm sewer is proposed to outlet to a proposed enhanced swale located north along the Millennium Trail as part of the Subdivision works.

General topography of the land drains south towards the Millennium Trail.

Refer to the *Fields of Wellington Subdivision Stormwater Management Report* dated May 2023, prepared by Forefront Engineering Inc for further details regarding the existing conditions for the subject site and adjacent land.

As per the findings and recommendations of the *Fields of Wellington Subdivision Stormwater Management Report* (2023), existing drainage from Block 15 is directed southwest to **Outlet 1**, the Wellington area storm sewer system and to Lake Ontario.

Refer to **Figure 2** – Pre-development Catchment Areas in Appendix C for further details regarding the existing site conditions.

Proposed Development

The development includes an approximate roof top area of 2,900m², an impervious hardscaped surface area of 4,600m² and approximately 4,700m² of pervious landscaping. The total lot area is approximately 1.23ha.

Pitched roofs are proposed for both the west and east buildings with disconnected roof drains directed to the onsite storm sewer and onsite swales.

The proposed Block 15 drainage plan is consistent with the recommendations of the *Fields of Wellington Subdivision Stormwater Management Report* (2023).

Quantity control and quality control is proposed onsite as per the recommendations from the *Fields of Wellington Subdivision Stormwater Management Report*. Post-development peak flows outletting from the site are to be equal to pre-development peak flows for the minor storm event up to the major storm event.

Onsite quantity storage is to be provided by the proposed dry pond stormwater management facility and storm sewer system. Peak flows from the site are to be restricted by an outlet control structure prior to outletting offsite.

Quality control is proposed onsite with the incorporation of an oil grit separator (OGS) provided at the outlet of the onsite storm sewer system prior to site runoff discharging into the dry storm pond. Enhanced protection corresponding to 80% TSS removal is provided for the subject site via the oil grit separator.

The onsite storm sewer is to be sized to convey the minor event. The major event is to be conveyed by the overland flow path and directed to the dry pond.

Drainage outletting from the subject site is to be directed to the west property line swale south to the enhanced swale proposed along the Millennium Trail eventually to **Outlet 1**, the Wellington area storm sewer system and to Lake Ontario.

A maximum of 250mm of surface ponding depth under blocked outlet conditions is proposed.

Refer to **Drawing C2** – Servicing Plan by Forefront Engineering in Appendix A for storm sewer servicing details and Figure 3: Post-Development Catchment Areas in Appendix C for the post-development catchment areas.

4.1 Water Quantity

4.1.1. Storm Sewers

Major Event

The major event shall be the 100 year event. Major flows greater than the minor event are to be conveyed via overland flow routes directed to the onsite dry pond where they will be controlled to pre-development levels and eventually outlet to the Millennium Trail enhanced swale (**Outlet 1**).

Minor Event

In accordance with general stormwater management practices, the onsite storm sewer is to be sized for up to the minor event, which is the 5 year design event based on the MTO IDF curve and Manning's equation.

The stormwater collection system shall be designed to accommodate runoff as per the Rational Method formula:

$$Q = 2.78AIR$$

where Q = Design flow in L/s,
A = area in hectares
I = rainfall intensity in mm/hr, and
R = runoff coefficient.

In the absence of local municipal guidelines, a minimum tc (time of concentration) of 15 minutes is to be used in conformance with general stormwater management practices.

The minor and major design storm events are based on IDF rainfall statistics that describe the frequency of rainfall depths over a specified duration. Rainfall intensities with various durations and return periods for the site were obtained from the **MTO IDF Curve** in Appendix C.

Refer to the **Storm Sewer Design Sheet** in Appendix C for storm sewer design details.

Runoff Coefficients

The runoff coefficient (R) is a dimensionless coefficient relating the amount of runoff to the amount of precipitation received. It is a larger value for areas with low infiltration and high runoff (pavement, steep gradient), and lower for permeable, well vegetated areas (forest, flat land). Coefficients were assigned based on surface cover and soil conditions as follows.

MECP Runoff Coefficients	
Land Use & Topography	Runoff Coefficients
Asphalt, concrete, roof area	0.90 - 1.00
Grassed area, parkland	0.15 - 0.35
Brick Roads	0.70 - 0.85
Sandy Soils	0.05 - 0.25
Playgrounds	0.20 - 0.35

Gravel	0.60 – 0.70
Forest and dense wooded areas	0.10 – 0.25
Permeable Pavements	0.15 – 0.25
Note: Values are taken from MECP “Design Criteria for Sanitary, Storm Sewer and Forcemain for Alterations Authorized under Environmental Compliance Approval”, 2022	

A runoff coefficient of 0.25 is to be used for grassed and soft landscape surfaces, and 0.9 is proposed for concrete, asphalt and rooftops. Refer to **Figure 4** Post-Development Runoff Coefficient and **Composite Runoff Coefficient Calculations** in Appendix C for further details.

The proposed development has an approximate composite coefficient of 0.65.

4.1.2. Analysis

Urbanization leads to an increase in impermeable surfaces (rooftops and parking areas). The resultant increased peak flows increase the risk to life, environment, and property damage. Water quantity control is generally required when there may be downstream quantity impacts.

Where the development proposes to outlet to the Wellington Main Street storm sewer system (**Outlet 1**) quantity control is required.

Minor storm drainage systems will be designed for the minor design event in accordance with general stormwater management practices. Overland flows paths are to convey the major storm event.

Pre-Development Flows

Analysis of the pre-development conditions and existing peak flows to **Outlet 1** are available in *The Fields of Wellington Subdivision Stormwater Management Report (2023)*. For the development blocks outletting to **Outlet 1**, in order meet pre-development peak flows *The Fields of Wellington Subdivision Stormwater Management Report* proposes an allowable minor event controlled peak outflow of 0.035 m³/s per hectare and a peak flow of 0.058 m³/s per hectare for the major event.

The subject site is 1.22 hectares in area, which equates to an allowable controlled peak flow of 0.043 m³/s for the minor event, and 0.071 m³/s for the major event directed to **Outlet 1**.

Post-Development Flows

The proposed development will increase the imperviousness of the site and hence the runoff. Results shown in Table 4-1 quantify the uncontrolled peak rate of surface runoff that has been routed through the collection system and directed to the onsite storage system.

Table 4-1 Uncontrolled Peak Flows in Post-Development Conditions

Uncontrolled Peak Flows in Post-Development Conditions (m ³ /s)				
Outlet	Area (ha)	Composite Coeff.	1:5 Year Storm	1:100 Year Storm
Directed to Dry Pond	1.22	0.65	0.147	0.255

Uncontrolled post-development peak flows result in an increase in peak flows from pre-development levels. The minor uncontrolled peak flow is 0.166 m³/s, requiring a reduction of 0.123 m³/s to reduce peak outflows to 0.043 m³/s, and

the 100 year uncontrolled peak flow is 0.275 m³/s, requiring a reduction of 0.204 m³/s to reduce peak outflows to 0.071 m³/s.

Onsite storage via a dry pond and storm sewer system is proposed to restrict post-development peak flows to pre-development levels.

Modified Rational Method

When developing the Rational Method the runoff volume was not considered, and the Rational Method alone was not meant for detention basin design. However, the Modified Rational Method, actually an extension of the conventional Rational Method, has been used in the past for sizing of detention basins. This method should generally be restricted to drainage areas less than 20 acres, and this method is expected to be appropriate for the subject site. The Modified Rational Method uses the peak flow calculations paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. This approach assumes the stormwater runoff hydrograph (detention basin inflow hydrograph) for the design storm is trapezoidal in shape. The peak runoff rate is calculated using the Rational Method formula and it is assumed that the peak of the outflow hydrograph falls on the recession limb of the inflow hydrograph and the rising limb of the outflow hydrograph can be approximated by a straight line. The storage volume is approximated with the following equation:

$$S_d = Q_p t_d - Q_d ((t_d + t_c) / 2)$$

*Storage Formula (Aron and Kibler, 1990)

Where:

- Q=Peak runoff rate (m³/s)
- C=Composite runoff coefficient
- I=Rainfall intensity (mm/hr)
- A=Drainage area (ha)
- tc= Time of Concentration (min)
- td = Duration of Storm (min)
- Qp = Peak Flow (m³/s)
- Qd = Discharge Rate (m³/s)
- Sd = Required Storage Volume (m³)

The design storm duration is that duration that maximizes the detention storage volume, S_d, for a given return period. An allowable target outflow is set based on pre-development conditions. The storm duration is t_d, and is varied until the storage volume is maximized.

Dry Pond Type Stormwater Management Facility

Post-development peak flows are to be controlled to pre-development levels to **Outlet 1** for the minor event (5 year) up to the major event (100 year), outletting to the Millennium Trail enhanced swale and Main Street storm sewer system and as summarized in Table 4-2.

Table 4-2 Controlled Peak Flows in Post-Development Conditions

Controlled Peak Flows in Post-Development Conditions						
Outlet	Area (ha)	Composite Coeff.	1:5 Year Peak Flow (m ³ /s)	1:5 Year Storage (m ³)	1:100 Year Peak Flow (m ³ /s)	1:100 Year Storage (m ³)
Outletting to Outlet 1	1.22	0.65	0.043	133	0.071	219

Storage is to be provided by the onsite storm sewer system and dry pond stormwater management facility located at the southwest of the site prior to outletting to the western swale. An outlet structure with a combination of an orifice plate fixed to the outlet control structure and interior weir/orifice will restrict flows for the minor and major storm events to pre-development levels.

Refer to the **Modified Rational Method Calculations** in Appendix C for the 5 year and 100 storage requirement calculations.

A total of 219 m³ of storage is required to maintain post-development peak flows to pre-development levels. The dry pond stormwater management facility and enhanced swale provide capacity for up to 206 m³ of storage and the onsite storm sewers provide an additional 14 m³ of storage for a total of 219 m³ of storage at the 84.50 m elevation (high water line). To include freeboard, the top of the proposed dry pond facility is to be 84.70 m. There is more than sufficient volume available for this development based on the proposed dry pond characteristics listed in Table 4-3. Refer to the **Stage-Storage Discharge Calculations** in Appendix C for detailed storage and discharge calculations.

Table 4-3 Dry Pond Stormwater Management Facility - Stage – Storage Relationship

Stage - Storage Relationship							m ³ /s		
Elevation (m)	Surface Area (m ²)	Incremental Depth (m)	Incremental Volume (m ³)	Total Depth (m)	Total Volume (m ³)	Comment	160mm Orifice	195mm Orifice	Total
							83.72	0	0.00
84.36	450	0.64	136	0.64	136	5 year Event	0.044	-	0.044
84.50	630	0.14	75	0.78	211	100yr Event (HWL)	-	0.071	0.071
84.70	895	0.20	N/A	0.98	N/A	Freeboard (0.2m)	-	-	-

Post-development flows are proposed to be attenuated to pre-development levels for all storm events up to the major event (100 year event) to **Outlet 1**.

An outlet structure located at the west end of the dry pond facility with an interior concrete weir insert and orifice sized for the minor event and an orifice plate at the structure outlet sized for the major event is provided to restrict peak flows to pre-development levels. The interior weir and orifice controls for the minor event while flows greater than the minor event overtop the interior concrete weir and are further restricted by the outlet orifice. Refer to Appendix A, **Drawing C4** - Details for further structure and stormwater management facility details.

Should an inlet grate become blocked or an event exceed the major design storm, surface ponding shall occur within the parking area around the catchbasins up to a maximum ponding depth of up to 250mm prior to flowing over the high points and spill over into the dry pond. An emergency bypass is provided at the outlet of the dry pond facility in the case that the outlet orifice is blocked or an event exceed the major design storm. Refer to Appendix A, **Drawing C1** – Grading Plan prepared by Forefront Engineering for further details regarding site grading.

The west property limit outlet swale has an available capacity of approximately 0.32 m³/s which is more than adequate to convey up to the controlled major storm event of 0.07 m³/s and includes capacity for the upstream development. Refer to the **Outlet Swale Capacity Calculations** in Appendix C for calculation details.

4.2 Water Quality

The *Stormwater Management Planning and Design Manual* (2003) by the Ministry of the Environment, Conservation and Parks (MECP) describes various levels of protection of water quality, based on a general relationship between the end-of-pipe stormwater management facilities long-term suspended solids removal and the lethal and chronic effects of suspended solids on aquatic life.

Based on the characteristics of the receiving watercourse and recommendations from the *Fields of Wellington Subdivision Stormwater Management Report (2023)*, Enhanced protection (corresponding to the end-of-pipe storage volumes required for the long-term removal of 80% of suspended solids) is required.

Oil Grit Separator

An oil grit separator is the preferred choice for a stormwater management facility for this site for quality control given the gradient and area available onsite.

The *Stormwater Management Planning and Design Manual (2003)* recommends that for Enhanced protection, oil grit separators be sized to capture and treat a minimum of 90% of the total runoff volume that occurs on a long-term average basis and remove 80% of suspended solids as required by Enhanced protection.

A Contech CDS 2020_5 unit has been specified as the OGS to provide 80% suspended solids removal.

The proposed onsite storm sewer network will outlet to an oil grit separator. The proposed oil grit separator will provide in excess of 83.9% suspended solids removal, capturing an annual 98.9% of the total runoff volume over a long-term average basis. Therefore the site achieves Enhanced protection quality treatment targets.

Below is a summary of the design criteria and OGS unit specifications.

Area:	0.90 hectares
Runoff Coefficient (C):	0.70
Treatment Capacity:	31 L/s (Rational Method)
Predicted Net Annual Load Removal Efficiency:	83.9 %
Predicted % Annual Rainfall Treated:	98.9 %
Sump Storage Capacity:	1.5 m ³
Oil Storage Capacity:	376 Litres

Refer to Appendix C, **CDS Estimated Net Annual Solids Load Reduction** for further details.

4.3 Maintenance

Dry Pond, Oil Grit Separator and Storm Sewer System

Sediment accumulation will occur rapidly during the construction period. Once a catchment area is developed and vegetation established, sediment accumulation diminishes and the frequency of removal within the stormwater management facilities will decrease.

Dry pond sediment buildup is considered minimal as most of the sediment is collected in the oil grit separator. The dry pond is designed for quantity control, so minimal sediment is collected. Sediment cleanout of the dry pond is recommended to be every 15 to 30 years or as necessary.

The oil grit separator will separate the oils and sediment from runoff onsite and will require annual maintenance and pumper truck access.

Periodic maintenance inspection of the oil grit separator should be undertaken and semi-annual maintenance reports (spring and fall) should be completed. Oil grit separators should be inspected regularly to ensure optimum performance. Pollutants collect in these systems at highly varying rates depending on the activities and conditions

taking place on the roadways and parking areas. For instance heavy winter sanding will cause the grit chamber to fill more quickly but regular sweeping of paved surfaces will slow accumulation.

Inspection should include and comment on the following:

- Working order of the system components including any defects and damage, these deficiencies should be repaired as soon as possible.
- Quantify the accumulations of hydrocarbons, trash, and sediment in the system. This can be done with a calibrated dipstick, tape measure or other measuring instrument.
- Levels of discoloration of absorbent materials used in some systems should be recorded.
- Hydraulic operation of the oil grit separator (detention time, evidence or occurrence of overflows).
- Condition of structure.
- Inlet obstructions if any.
- Evidence of parking lot spills.
- Maintenance and operational control undertaken during the year.
- Recommendations for inspection and maintenance program for the coming year.

The system should be cleaned when the level of sediment has reached 75% of capacity in the isolated sump or when an appreciable level of hydrocarbons and trash has accumulated. If absorbent material is used, it should be replaced when significant discoloration has occurred. Performance will not be impacted until 100% of the sump capacity is exceeded, however it is recommended that the system be cleaned prior to that for easier removal of sediment.

The level of sediment is easily determined by measuring from finished grade down to the top of the sediment pile. To avoid underestimating the level of sediment in the chamber, the measuring device must be lowered to the top of the sediment pile carefully. Particles at the top of the pile typically offer less resistance to the end of the rod than consolidated particles toward the bottom of the pile. Once this measurement is recorded, it should be compared to the as-built drawing for the unit to determine whether the height of the sediment pile from the bottom of the sump floor exceeds 75% of the total height of isolated sump.

The following are instructions and best practices for cleaning the oil grit separator unit:

- Work should take place during dry weather and no flow conditions.
- Vacuum trucks are the most effective means of sediment and pollutant removal.
- Completely drain the system and fully excavate the sediment from the sump.
- The system requires cleaning immediately following a large hydrocarbon spill event.
- Hydrocarbon accumulation should be removed using absorbent pads.
- Trash and debris can be separated from the pollutants.
- Screens should be power washed.
- Confined space entry procedures should be followed.
- Pollutants should be disposed of and handles as per MECP guidelines.

As a first approximation, and based on an 80% TSS capture efficiency, OGS catchment area of 0.90 ha, runoff coefficient of 0.70, average annual rainfall of 900 mm, estimated influent grit concentration of 150 mg/L (STEP database for residential) and compacted grit density of 1600 mg/L it can be expected that the OGS will accumulate an estimated 425 L of grit annually, requiring cleanout of the OGS every 30 months on average. The OGS manufacturer typically recommends cleanout at a minimum of every 2 – 3 years to avoid sediment agglomeration which makes cleanout difficult.

All inspection and maintenance onsite are the responsibility of the Owner.

4.4 Quality Control (Short Term)

Silt fencing is to be provided at all side slopes and down gradient locations to ensure sediment and erosion control during construction. Other control devices such as straw bale check dams will also be provided where drainage is concentrated. A clear stone mud mat is to be provided at the entrance to the site.

The timeframe for land to remain exposed before it is stabilized with sod, mulch, or hydroseeding is to be minimized.

Inspection of the sediment control works should be undertaken before and after all rainfall (and snowmelt) events. Maintenance is to be undertaken as required to ensure the proper operation of all sediment and erosion controls.

5. Utilities

The proposed development will be serviced by electrical, gas, telephone and cable television services.

Final gas and electrical service sizes required will be confirmed with the mechanical and electrical engineers prior to Building Permit Approval.

Refer to **Drawing C2** – Servicing Plan by Forefront Engineering detailing the proposed servicing.

6. Conclusions

Detailed calculations find that the proposed infrastructure is capable of and will effectively service the proposed demand created by this development.

A 150mm water service connection is proposed to West Street.

A 200mm sanitary sewer connection is proposed to the extension of West Street.

An onsite storm sewer system outletting to the onsite oil grit separator and dry pond stormwater management facility discharging to the proposed Fields of Wellington enhanced swale located along the Millennium Trail is proposed. An onsite dry pond is proposed to provide quantity control and an oil grit separator is proposed to provide enhanced quality control. Post-development flows are proposed to be attenuated to pre-development levels for all storm events up to the major event (100 year event) to Outlet 1.

Electrical, communication and natural gas connections are proposed on West Street.

Appendix A

Architect Site Plan

C1 – Grading Plan

C2 – Servicing Plan

C3 – Details

C4 – Details

SITE WORKS

- Refer to Geotechnical Report by Malroz Engineering Inc. dated November 5, 2018.
- Road cuts to be reinstated as per The County of Prince Edward requirements.
- Provide construction fencing and signage as per The County of Prince Edward requirements.
- Hot mix, hot laid asphalt concrete as per OPSS 1150.
- Mix designs shall contain a minimum of 5.4% asphalt cement with a performance grade of PH58-28 and 3.5% air voids. Parking lot cross sections shall be:
Heavy Duty
40mm HL3
50mm HL8
150mm Granular 'A'
300mm Granular 'B' Type II
- Concrete curb shall be barrier type OPSD 600.110 unless indicated otherwise.
- Concrete shall be 32Mpa 28 day strength, 19mm coarse aggregate nominal maximum size, 6.0% to 8.0% air content. Normal portland cement type as per OPSS 1301. Concrete materials and production as per OPSS 350, 351, 352, 353, 1301, 1302, and 1350.
- Boulevards and grass areas to be finished with 100mm of topsoil and seed.
- Paint lines for standard parking spaces to be CAN/CSG-1.74-2001, ALKYD Traffic Paint. Pavement surface to be free from ponded water, frost, ice, dust, oil, grease, and other foreign materials prior to painting. Paint lines to be uniform colour and density with sharp edges. Protect pavement markings until dry.
- All construction dewatering during building construction up to occupancy is the Contractor's responsibility.

GRADING

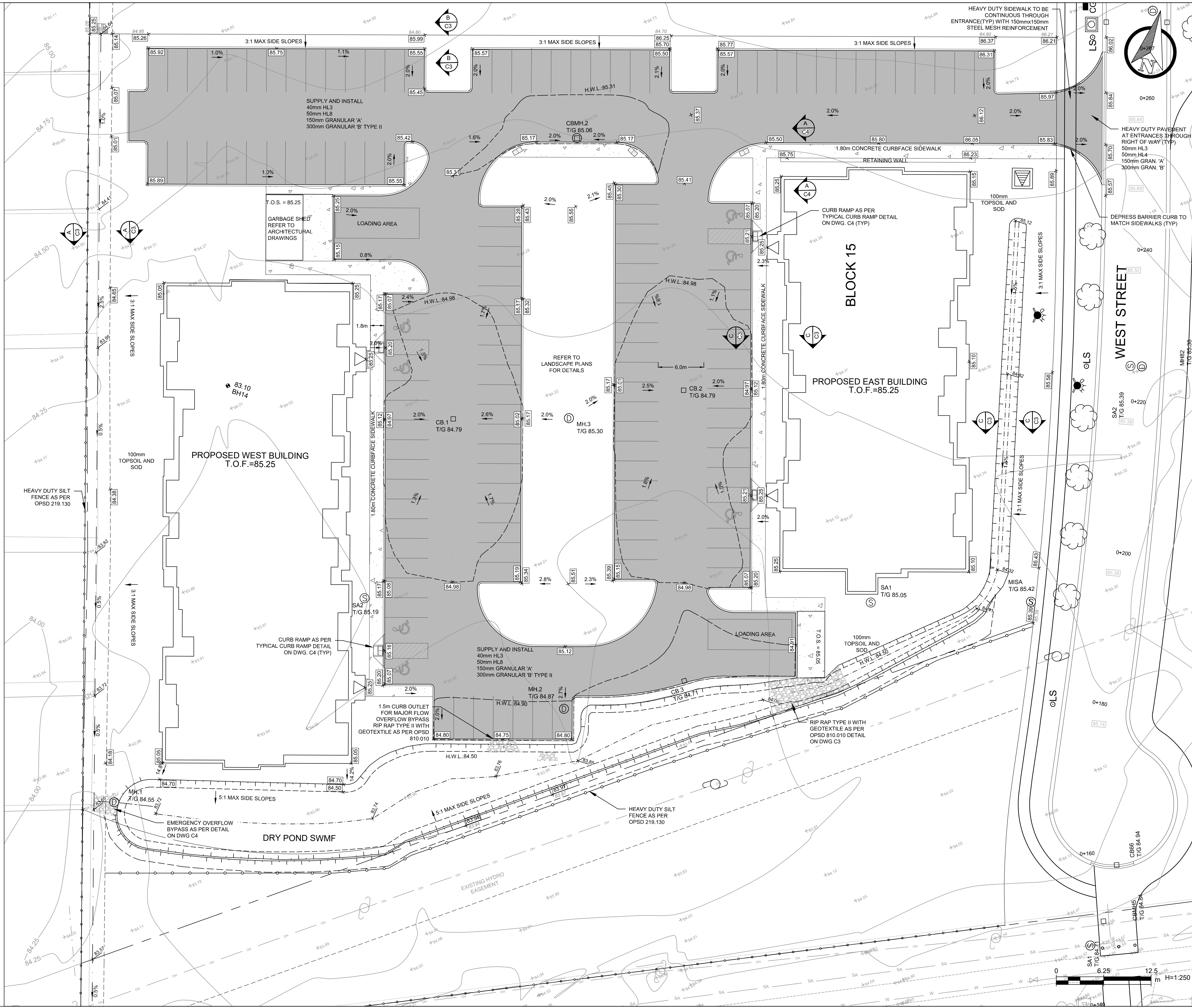
- Existing elevations as per Forefront Survey 2022.
- Positive drainage away from the building shall be achieved. Finished surfaces shall be at a minimum grade of 2% unless otherwise noted.
- 150mm of granular 'A' shall be supplied and installed under all steps, decks, and porches unless otherwise specified.
- Side slopes shall be a maximum slope of 3:1 unless otherwise noted.
- All existing elevations and grades are to be verified by the contractor prior to grading.
- Utilities are to be located prior to construction.
- All ground surfaces shall be graded to prevent ponding and without low areas except where approved swale or catchbasin outlets are provided.
- Maximum hard surface grades are to be 8.0%.
- Staked straw bale filters and flow checks to be installed as per OPSD 219.100 and OPSD 219.180 around catchbasins and sewer inlets.
- Conflicts between the architectural drawings and grading shall be brought to the attention of the contract administrator.
- The contractor is responsible for reviewing proposed grades with conflicts regarding the proposed structures.
- Sub-drain and outlet elevations are to be confirmed prior to the construction of any structures.
- Sub-grade shall be graded at a minimum of 2% until a lower ditch is encountered or the existing surface drains away from the sloped sub-grade.
- Grades are to match the adjacent properties unless otherwise noted.
- All driveway curbing within 0.6m of a county sidewalk is to be depressed to the elevation of the sidewalk.
- F.F.E. shall mean First Floor Elevation.
T.O.F. shall mean Top Of Foundation.
T.O.S. shall mean Top Of Slab.

SEDIMENT AND EROSION CONTROL NOTES

- All erosion and sediment controls shall be installed prior to construction and monitored and maintained by the Contractor throughout the construction process, until all disturbed areas have been revegetated, then the temporary sediment and erosion control measures must be removed once the site has been stabilized and the site works are complete.
- Staked straw bale filters and flow checks to be installed as per OPSD 219.100 and OPSD 219.180 around catchbasins and sewer inlets.
- All erosion and sediment control measures shall be inspected after each rainfall to the satisfaction of Prince Edward County and QCA.
- Any disturbed area not scheduled for further construction within forty-five (45) days will be provided with a suitable temporary mulch and seed cover within seven (7) days of completion of that particular phase of construction.
- Regardless of site specific items detailed on the plans, the Contractor shall install erosion control measures to suit the proposed work methods controlling sediment runoff from discharging offsite prior to any disturbance.
- Following construction, disturbed areas, as well as proposed grassed and vegetated surfaces, shall be reinstated as soon as practical.
- The placement of surplus fill to be placed and/or temporarily stored in accordance with Prince Edward County requirements or O Reg 148/06 where applicable. Temporary fill sites will require the approval of the County. Fill locations, side slopes, elevations as per the Approved Drawings. Restoration to be completed during the final phase of construction.
- All roads used to access the site shall be kept clean to the satisfaction of the Director of Engineering Services.

CLOSEOUT

- Request Final Inspection and SPA securities release from the Civil Engineering Consultant.
- Maintain as-built drawings throughout the durations of the project. Submit as-built drawings for the project record.
- Complete and submit a final site survey certificate (completed by OLS or Engineer), certifying that elevations and locations of completed Works are in conformance, or non-conformance with Contract Documents.
- Prior to final review remove surplus products, tools, construction machinery and equipment. Remove waste products and debris including that caused by Owner or other Contractors. Broom clean and wash exterior walks, steps and surfaces, rake clean other surfaces of grounds. Remove dirt and other disfiguration from exterior surfaces. Sweep and wash clean paved areas.



LEGEND

- HEAVY DUTY ASPHALT
- CONCRETE
- PROPOSED STORMWATER
- PROPOSED SANITARY
- PROPOSED WATERMAIN
- PROPOSED SWALE
- EXISTING SANITARY
- EXISTING STORM
- EXISTING WATER
- EXISTING GAS
- EXISTING BELL CABLE
- EXISTING UND.G. HYDRO
- EXISTING FENCE
- EXISTING CB, DCB
- EXISTING STORM MH, CBMH
- EXISTING SANITARY MH
- EXISTING HYDRANT, VALVE
- EXISTING DECIDUOUS, CONIFEROUS TREE
- EXISTING LIGHT STANDARD
- BENCHMARK
- BOREHOLE
- WELL
- EXISTING GRADE
- PROPOSED GRADE
- SLOPE ARROW
- PROPOSED SWALE GRADE

Benchmark
 1. BENCHMARK BEING THE SPIKE IN CONCRETE SIDEWALK, NORTHING: 4885386.064 EASTING: 31181.400, HAVING AN ELEVATION OF 84.38m.
 2. BM2 BEING NAIL IN HYDRO POLE, NORTHING: 4885386.270 EASTING: 311425.870, HAVING AN ELEVATION OF 85.15.

R#	Revision/Issue	Date

Forefront Engineering Inc.

1329 Gardiners Road, Suite 210
 Kingston, ON, Canada K7P 0L8
 613.634.9009 tel.
 1.888.884.9392 fax.

Client: **KAITLIN CORPORATION**

Project: **FIELDS OF WELLINGTON SUBDIVISION BLOCK 15**

Drawing: **GRADING PLAN**

Drawn by: GSD	Checked by: JH	Project No.:
Designed by: KMN	Approved by: KMN	Drawing No.:
Date: JUNE 2023	Scale: 1:250	



GENERAL

- All works to be installed in accordance with current Prince Edward County Development Guidelines, Ontario Building Code and, Ontario Provincial Standard Specifications and Drawings unless specified otherwise.
- The Contractor shall acquire all permits as required for all works within the Municipal Right of Way. Cut permits are required for all offsite works.
- Refer to the Architectural and Landscape Architectural Drawings and coordinate.
- Prior to construction, Contractor to verify all dimensions, sewer inverts and utility locates and identify possible conflicts.
- Refer to Geotechnical Report by Malroz Engineering dated Nov. 27, 2019.
- In the event that human remains are encountered during construction, the Contractor shall immediately contact the Ministry of Tourism Culture and Sport (416-314-7132), the Registrar of the Cemeteries Regulation Unit from the Ministry of Consumer Services, at (416-326-8393), Ontario Provincial Police (613-476-2151).
- In the event that deeply buried or previously undiscovered archaeological deposits are discovered in the course of development or site alteration, the Contractor shall immediately contact the Ministry of Tourism Culture and Sport (416-314-7132).

WATERMANS

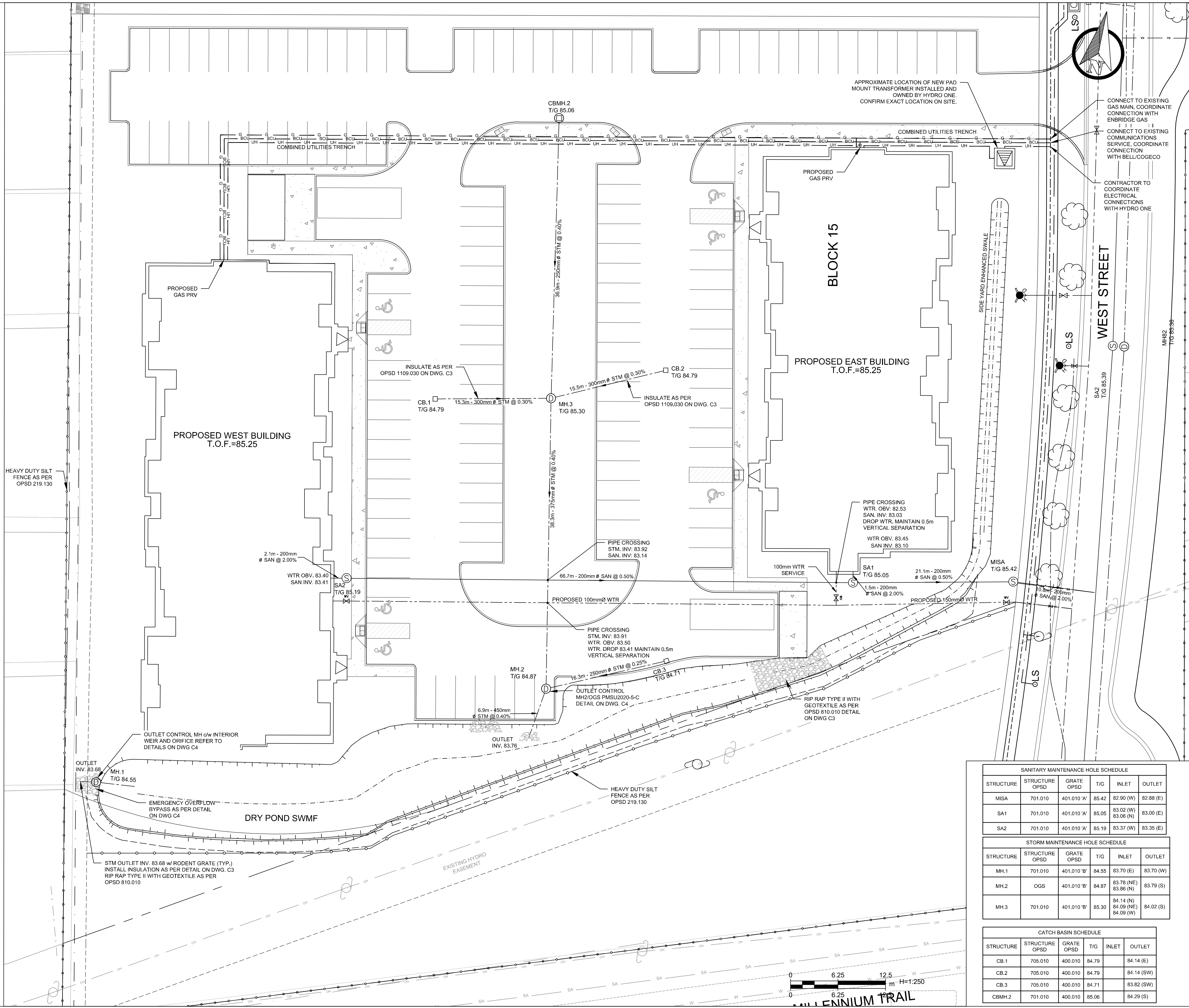
- Watermains shall be AWWA C900-Poly (Vinyl Chloride) (PVC) Specification, SDR18, Pressure Class (PC) of 235 or (b) AWWA C900-Molecularly Oriented Polyvinyl Chloride (PVC) Specification Pressure Class(PC) of 235. Other pipe materials may be accepted by the County at the sole discretion of the Director of Engineering Services based on the acceptance of technical specification for that material.
- Pipe bedding and cover shall conform to OPSP 802 series. Bedding and cover shall be Granular "A".
- The minimum depth of cover measured from the top of a main or a service connection gosseneck shall not be less than 1.7m.
- Watermain to have a minimum separation of 2.5m from parallel ditches and sewers, crossing sewers shall have a minimum vertical separation.
- Watermain vertical separation of 0.5m and insulated as directed by the Engineer.
- Service connections to PVC mains shall be by stainless steel saddle within municipal right of ways and molded tee fittings within the buildings property, or approved equivalent.
- Valve boxes shall be adjusted to finish grade. Valve boxes shall be of cast iron as manufactured by Bibby Ste Croix or Star Pipe Products, 112mm and shall be of sliding type, complete with grommet hole for tracer wire, which cannot carry any surface load down to the pipe. The covers shall be of a design which prevents unauthorized entry and marked "Water", length to be 140mm to 240mm.
- Gate valves for pipe shall be flow or equal, cast iron body, resilient seated mechanical joint pattern conforming to ANSI/AWWA C509, designed for a working pressure of not less than 150 PSI. Valves shall open when turned in a clockwise direction, shall be fitted with a compound operating nut.
- Water services a minimum of 5.5m in length shall be tested and disinfected to County Standards, AWWA and Building Code Standards.
- Water services 100mm in diameter or larger shall be hydrostatically tested and disinfected to County, AWWA and Building Code Standards. Fire supply lines shall be tested to NFPA 24.
- Hydrant shall be the colour for private hydrants. Chrome yellow or red. Contractor to confirm with building department prior to ordering.
- Prior to testing and disinfection of the watermain and large services, Prince Edward County requires the contractor submit for review a testing, Disinfection and Final Connection Plan.

SEWERS

- Existing pipe inverts are approximate and must be confirmed by the Contractor in the field.
- Pipe bedding and cover shall conform to OPSP 802 series. Bedding and cover shall be Granular "A".
- Storm sewer materials as per:
 - Reinforced concrete pipe per OPSS 1820 according to CSA A257.2 Class 65-D with rubber gaskets or
 - Type PSM polyvinyl chloride (PVC) pipe with elastomeric gasketed bell and spigot type joints as per OPSS 1841 and CSA B182.2.
 - Mains - DR 35 pipe
 - Storm Laterals - DR 35 pipe
 - Profile Polyvinyl Chloride (PVC) pipe with elastomeric gasketed bell and spigot type joints meeting OPSS 1841 and CSA B182.4 and having a minimum stiffness of 320 kPa may be used for storm sewer applications.
 - ADS N-12 ST/WT sizes 100mm to 900mm corrugated dual-wall High Density Polyethylene (HDPE) pipe with elastomeric gasketed bell and spigot type joints meeting CSA 182.8 and OPSS 1840, and having a minimum stiffness of 320 kPa.
- All manhole and catchbasin grate elevations are approximate and are subject to adjustment in the field.
- Catchbasin leads to be 200mm unless noted otherwise.
- Water and sewer services and foundation drains may be laid in the same trench subject to the provisions of the Ontario Building Code and MOECC.
- Sanitary laterals shall be PVC, CSA certified and conform to OPSS.
- Laterals are to be sized to meet the Ontario Building Code as amended and shall be minimum 125mm. The colour shall be green.
- Saddles and fittings for services are to be approved by Prince Edward County.
- Backwater Valve to be installed on all storm services to the satisfaction of Prince Edward County.
- Sanitary sewers shall be tested for deflection as per OPSS 410.
- Where a clearance between pipe or conduit crossings of 300mm or less cannot be avoided, there shall be concrete encasement or non-shrink backfill of the crossing extending one metre in each direction of each pipe.
- Sanitary maintenance hole cover to OPSS 401.01 Type A. Storm maintenance hole cover to OPSS 401.01 Type B, unless noted otherwise. Catch basin cover to be 400.010.
- All maintenance holes and catch basins located in the pavement are to have the frame set at base course elevation. These frames are to be subjected to final adjustment at such time as the surface course is applied.
- Geotextiles must be placed under the lids of all existing manholes and catch basins in the working area for the duration of construction. Upon completion of construction, geotextile is to be removed along with any accumulated sediment.
- Basement sanitary sewers and foundation subdrains are to be pumped to the proposed service connections, foundation drain shall include backwater valve. Refer to Mechanical Engineering Drawings.

CLOSEOUT

- Request Final Inspection and SPA securities release from the Civil Engineering Consultant.
- Contractor to maintain as-built drawings throughout the durations of the project. Submit as-built drawings for the project record.
- Complete and submit a final site survey certificate (completed by OLS or Engineer), certifying that elevations and locations of completed Works are in conformance, or non-conformance with Contract Documents.
- Prior to final review remove surplus products, tools, construction machinery and equipment. Remove waste products and debris including that caused by Owner or other Contractors. Broom clean and wash exterior walks, steps and surfaces; rake clean other surfaces of grounds. Remove dirt and other disfiguration from exterior surfaces. Sweep and wash clean paved areas.
- Clean, flush and CCTV all sanitary and storm sewer's prior to expiration of the Contractor's warranty for Contract Administrators approval.
- Contractor to Mandrel all sewers under Contract Administrators supervision.
- Contractor to Leak test sanitary sewer system under Contract Administrators supervision.



LEGEND

- PROPOSED STORMWATER
- PROPOSED SANITARY
- PROPOSED WATERMAIN
- PROPOSED SWALE
- EXISTING SANITARY
- EXISTING STORM
- EXISTING WATER
- EXISTING GAS
- EXISTING BELL CABLE
- EXISTING UND.G. HYDRO
- EXISTING FENCE
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- EXISTING STORM MH, CBMH
- EXISTING SANITARY MH
- EXISTING HYDRANT, VALVE
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- BENCHMARK
- BOREHOLE
- WELL
- EXISTING GRADE
- PROPOSED GRADE

Benchmark
 1. BENCHMARK BEING THE SPIKE IN CONCRETE SIDEWALK, NORTHING: 4869386.064 EASTING: 31181.400, HAVING AN ELEVATION OF 84.38m.
 2. BM2 BEING NAIL IN HYDRO POLE, NORTHING: 4869386.270 EASTING: 311425.870, HAVING AN ELEVATION OF 85.15.

Revised	Revision/Issue	Date



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Client
KAITLIN CORPORATION

Project
**FIELDS OF WELLINGTON SUBDIVISION
 BLOCK 15**

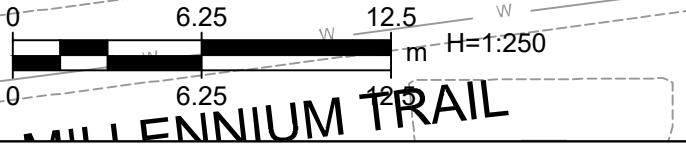
Drawing
SERVICING PLAN

Drawn by: GSD	Checked by: JH	Project No.:
Designed by: KMN	Approved by: KMN	Drawing No.:
Date: JUNE 2023	Scale: 1:250	

STRUCTURE	STRUCTURE OPSD	GRATE OPSD	T/G	INLET	OUTLET
MISA	701.010	401.010 'A'	85.42	82.90 (W)	82.88 (E)
SA1	701.010	401.010 'A'	85.05	83.02 (W)	83.06 (E)
SA2	701.010	401.010 'A'	85.19	83.37 (W)	83.35 (E)

STRUCTURE	STRUCTURE OPSD	GRATE OPSD	T/G	INLET	OUTLET
MH.1	701.010	401.010 'B'	84.55	83.70 (E)	83.70 (W)
MH.2	OGS	401.010 'B'	84.87	83.78 (NE)	83.79 (S)
MH.3	701.010	401.010 'B'	85.30	84.14 (N)	84.09 (NE)

STRUCTURE	STRUCTURE OPSD	GRATE OPSD	T/G	INLET	OUTLET
CB.1	705.010	400.010	84.79	84.14 (E)	
CB.2	705.010	400.010	84.79	84.14 (SW)	
CB.3	705.010	400.010	84.71	83.82 (SW)	
CBMH.2	701.010	400.010	85.06	84.29 (S)	



Appendix B

Water Demand Calculations

Fire Flow Calculations

MSP Water Model Figures

Sanitary Sewer Design Sheet

Water Supply Demand Calculations

Block 15 - Fields of Wellington Subdivision

DESIGN NOTES:

q=average daily per capita flow (L/cap. d) 350 L/cap. D (PEC Request, MSP = 320 L/cap.D)
 -Multi- unit residential 1.79 ppl/unit (MSP, 2021)

Category	# of Residential Units	Population	Demand	L/day	L/s	Peak Flow Demands		Fire Flow Required
						Peak Hour L/s	Maximum Day L/s	L/s
Residential	104	186	350 L/cap. D	65156	0.75	3.21	2.07	
	Total	186		65156	0.8	3.2	2.1	217

TOTAL AVERAGE DAY FLOW 0.8 L/s
 PEAK DAY FACTOR - 2.75
 MAXIMUM DAY FLOW 124 L/min 2.07 L/s

 PEAK HOUR FACTOR - 4.25
 PEAK HOUR FLOW 192 L/min 3.21 L/s

 MD+Fire Flow 13144 L/min 219.1 L/s

FUS Fire Flow Calculation

Project Name: Block 15 Fields of Wellington
 Date: August 2023
 Data Input by: Erik Persson
 Review by: Jeff Homer

Calculations based on "Water Supply for Public Fire Protection"
 by Fire Underwriters' Survey, 2020

Fire Flow Calculation #: 1
 Building Description: 4 Storey Wood Frame Apartment Complex
 East Building

Table A: Fire Underwriters Survey Determination of Required Fire Flow - Long Method									
Step	Task	Term	Options	Multiplier Associated with Option	Choose:	Value Used	Units	Total Fire Flow (L/min)	
1	Choose Frame used for Construction of Unit	Coefficient related to type of construction (C)	Framing Material			Wood Frame	1.5		
			Type V Wood Frame Construction	1.5					
			Type IV-A Mass Timber Construction	0.8					
			Type IV-B Mass Timber Construction	0.9					
			Type IV-C Mass Timber Construction	1.0					
			Type IV-D Mass Timber Construction	1.5					
			Type III Ordinary Construction	1.0					
			Type II Non-Combustible Construction	0.8					
Type I Fire resistive construction (>2 hrs)	0.6								
2	Choose Type of Housing (if TH Enter Number of Units Per TH Block)	Type of Housing	Floor Space Area			Mid Rise Residential	56	Units	
			Single Family						
			Townhouse - Indicate # of units						
			Other (Comm, Ind, Apt, etc.)						
2.1	# of Storeys	Number of Floors/Storeys in the Unit (do not include basement)				4	Storeys		
3	Enter Ground Floor Area of One Unit	Building Area Calculation (Total Effective Area)				4564	m ²		
		For Wood Frame and Ordinary Construction 100% of all floor areas are considered in determining the Total Effective Area.							
		Total Floor Area (√A)				68	√m ²		
4	Obtain Required Fire Flow without Reductions	Required Fire Flow (without reductions or increases per FUS) (F=220°C*√A)						22294	
5	Apply Factors Affecting Burning	Reductions/Increases Due to Factors Affecting Burning							
5.1	Choose Combustibility of Building Contents	Occupancy content hazard reduction or surcharge	Non-Combustible	-0.25	Limited Combustible	-0.15	N/A	-3344	
			Limited Combustible	-0.15					
			Combustible	0					
			Free Burning	0.15					
			Rapid Burning	0.25					
5.2	Choose Reduction Due to Presence of Sprinklers	Sprinkler Reduction	Adequate Sprinkler Conforms to NFPA13	-0.3	Sprinkler System Conforming to NFPA 13	-0.3	N/A	-5684.95	
			None	0					
		Water Supply Credit	Water supply is standard for sprinkler and fire dept. hose line	-0.1	Water Supply is standard	-0.1	N/A	-1894.98	
			Water supply is not standard or N/A	0					
5.3	Choose Separation Distance Between Units	Exposure Distance Between Units	N. Side	20.1 to 30m	0.1	0	m	1895	
			E. Side	Greater than 45m					
			S. Side	Greater than 45m					
			W. Side	Greater than 45m					
				Greater than 45m					
6	Obtain Required Fire Flow, Duration & Volume	Total Required Fire Flow, rounded to nearest 1000 L/min, with max/min limits applied:						13000	
		Total Required Fire Flow (above) in L/s						217	
		Required Duration of Fire Flow (hrs)						3.0	
		Required Volume of Fire Flow (m ³)						2340	

Notes:

Building Area Calculation (Approximate Floor Areas)

Story	Area (m ²)	Adjustment Factor	Adjusted Areas (m ²)
Ground	1141	100%	1141
2	1141	100%	1141
3	1141	100%	1141
4	1141	100%	1141

(2) Separation Charge

The Total percentage shall be the sum of the percentages for all sides, but shall not exceed 75%

Distance to exposure	Exposure Charge
0 to 3 m	25%
3.1 to 10 m	20%
10.1 to 20 m	15%
20.1 to 30 m	10%
30.1 to 45 m	0%

Firewall- Using a 10% exposure charge for fire wall (FUS, Note E)

FUS Fire Flow Calculation

Project Name: Block 15 Fields of Wellington
 Date: August 2023
 Data Input by: Erik Persson
 Review by: Jeff Homer

Calculations based on "Water Supply for Public Fire Protection"
 by Fire Underwriters' Survey, 2020

Fire Flow Calculation #: 2
 Building Description: 3 Storey Wood Frame Apartment Complex
West Building

Table A: Fire Underwriters Survey Determination of Required Fire Flow - Long Method									
Step	Task	Term	Options	Multiplier Associated with Option	Choose:	Value Used	Units	Total Fire Flow (L/min)	
1	Choose Frame used for Construction of Unit	Coefficient related to type of construction (C)	Framing Material			Wood Frame	1.5		
			Type V Wood Frame Construction	1.5					
			Type IV-A Mass Timber Construction	0.8					
			Type IV-B Mass Timber Construction	0.9					
			Type IV-C Mass Timber Construction	1.0					
			Type IV-D Mass Timber Construction	1.5					
			Type III Ordinary Construction	1.0					
			Type II Non-Combustible Construction	0.8					
Type I Fire resistive construction (>2 hrs)	0.6								
2	Choose Type of Housing (if TH Enter Number of Units Per TH Block)	Type of Housing	Floor Space Area			Mid Rise Residential	48	Units	
			Single Family						
			Townhouse - Indicate # of units						
			Other (Comm, Ind, Apt, etc.)						
2.1	# of Storeys	Number of Floors/Storeys in the Unit (do not include basement)				3	Storeys		
3	Enter Ground Floor Area of One Unit	Building Area Calculation (Total Effective Area)				4107	m ²		
		For Wood Frame and Ordinary Construction 100% of all floor areas are considered in determining the Total Effective Area.							
		Total Floor Area (√A)				64	√m ²		
4	Obtain Required Fire Flow without Reductions	Required Fire Flow (without reductions or increases per FUS) (F=220°C*√A)						21148	
5	Apply Factors Affecting Burning	Reductions/Increases Due to Factors Affecting Burning							
5.1	Choose Combustibility of Building Contents	Occupancy content hazard reduction or surcharge	Non-Combustible	-0.25	Limited Combustible	-0.15	N/A	-3172	
			Limited Combustible	-0.15					
			Combustible	0					
			Free Burning	0.15					
			Rapid Burning	0.25					
5.2	Choose Reduction Due to Presence of Sprinklers	Sprinkler Reduction	Adequate Sprinkler Conforms to NFPA13	-0.3	Sprinkler System Conforming to NFPA 13	-0.3	N/A	-5392.83	
			None	0					
		Water Supply Credit	Water supply is standard for sprinkler and fire dept. hose line	-0.1	Water Supply is standard	-0.1	N/A	-1797.61	
			Water supply is not standard or N/A	0					
5.3	Choose Separation Distance Between Units	Exposure Distance Between Units	Sprinkler Supervision		N/A	0	N/A	0	
			Sprinkler system is fully supervised	-0.1					
			Sprinkler not fully supervised or N/A	0					
			N. Side	20.1 to 30m					0.1
			E. Side	Greater than 45m					0
S. Side	Greater than 45m	0							
W. Side	Greater than 45m	0							
6	Obtain Required Fire Flow, Duration & Volume	Total Required Fire Flow, rounded to nearest 1000 L/min, with max/min limits applied:						13000	
		Total Required Fire Flow (above) in L/s						217	
		Required Duration of Fire Flow (hrs)						3.0	
		Required Volume of Fire Flow (m³)						2340	

Notes:

Building Area Calculation (Approximate Floor Areas)

Story	Area (m ²)	Adjustment Factor	Adjusted Areas (m ²)
Ground	1369	100%	1369
2	1369	100%	1369
3	1369	100%	1369

(2) Separation Charge

The Total percentage shall be the sum of the percentages for all sides, but shall not exceed 75%

Distance to exposure	Exposure Charge
0 to 3 m	25%
3.1 to 10 m	20%
10.1 to 20 m	15%
20.1 to 30 m	10%
30.1 to 45 m	0%

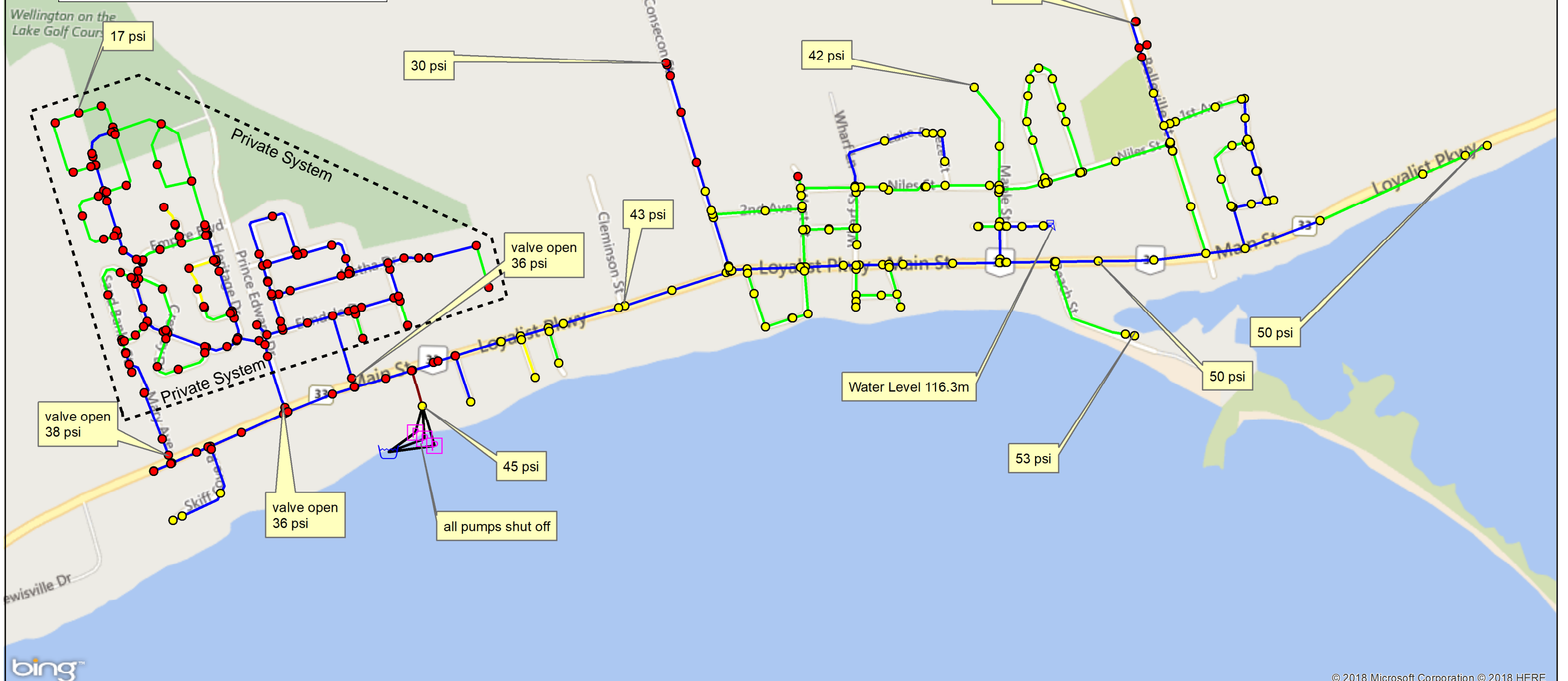
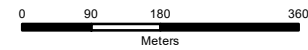
Firewall- Using a 10% exposure charge for fire wall (FUS, Note E)

APPENDIX 5

Wellington Water Model Figures

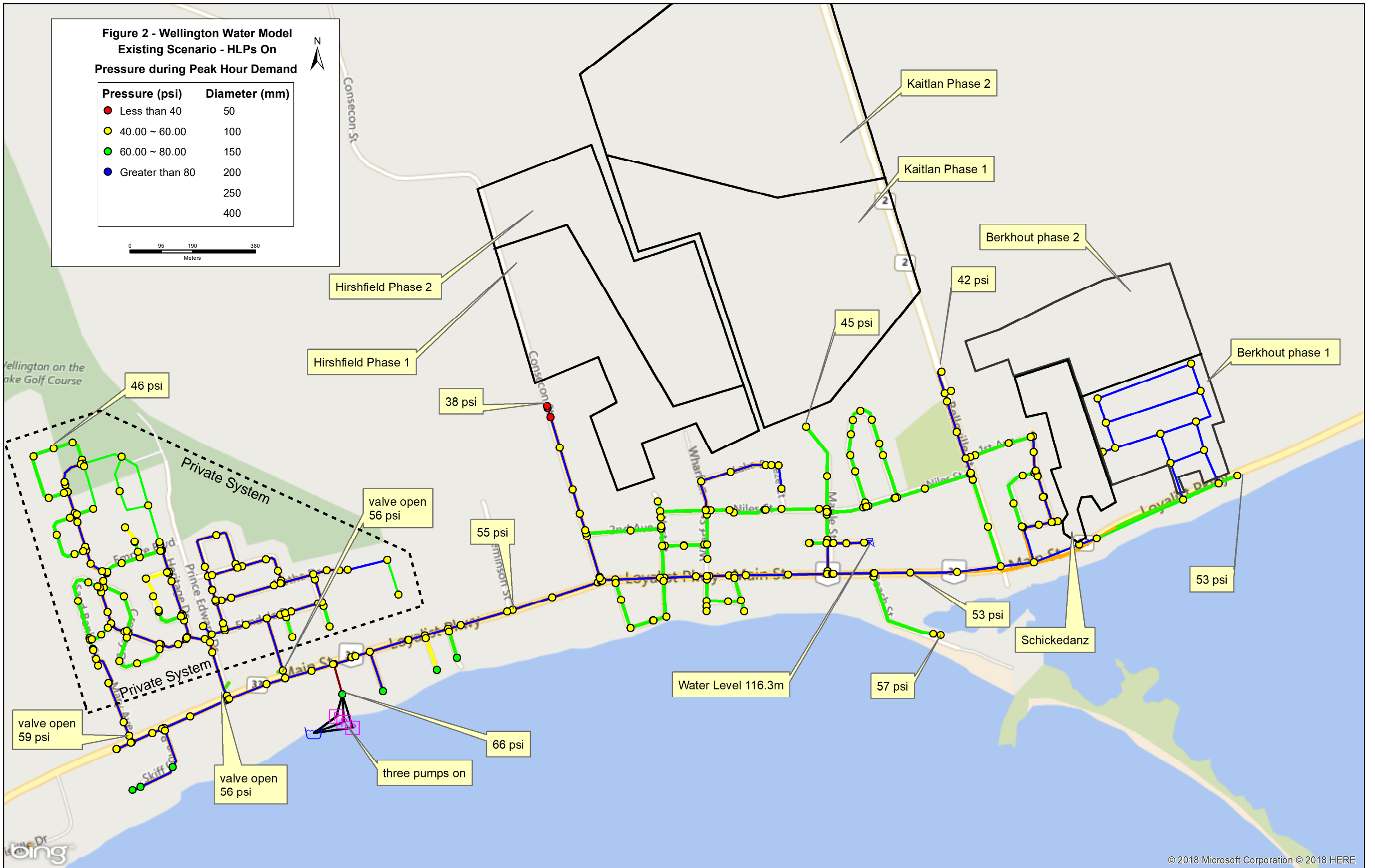
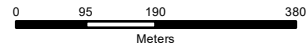
Figure 1 - Wellington Water Model
Existing Scenario - HLPs off
Pressure during Peak Hour Demand

Pressure (psi)	Diameter (mm)
Less than 40	50
40.00 ~ 60.00	100
60.00 ~ 80.00	150
Greater than 80	200
	250
	400



**Figure 2 - Wellington Water Model
Existing Scenario - HLPs On
Pressure during Peak Hour Demand**

Pressure (psi)	Diameter (mm)
Less than 40	50
40.00 ~ 60.00	100
60.00 ~ 80.00	150
Greater than 80	200
	250
	400



**Figure 3 - Wellington Water Model
Existing Scenario - HLPs On
Available Flow during Max Day Demand**

Available Flow (L/s)	Diameter (mm)
Less than 50	50
50.00 ~ 75.00	100
75.00 ~ 100.00	150
100.00 ~ 150.00	200
Greater than 150	250
	400

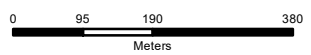


Figure A-1: Wellington Water Model

New ET, New Millennium Trail Watermain; HLPs Off

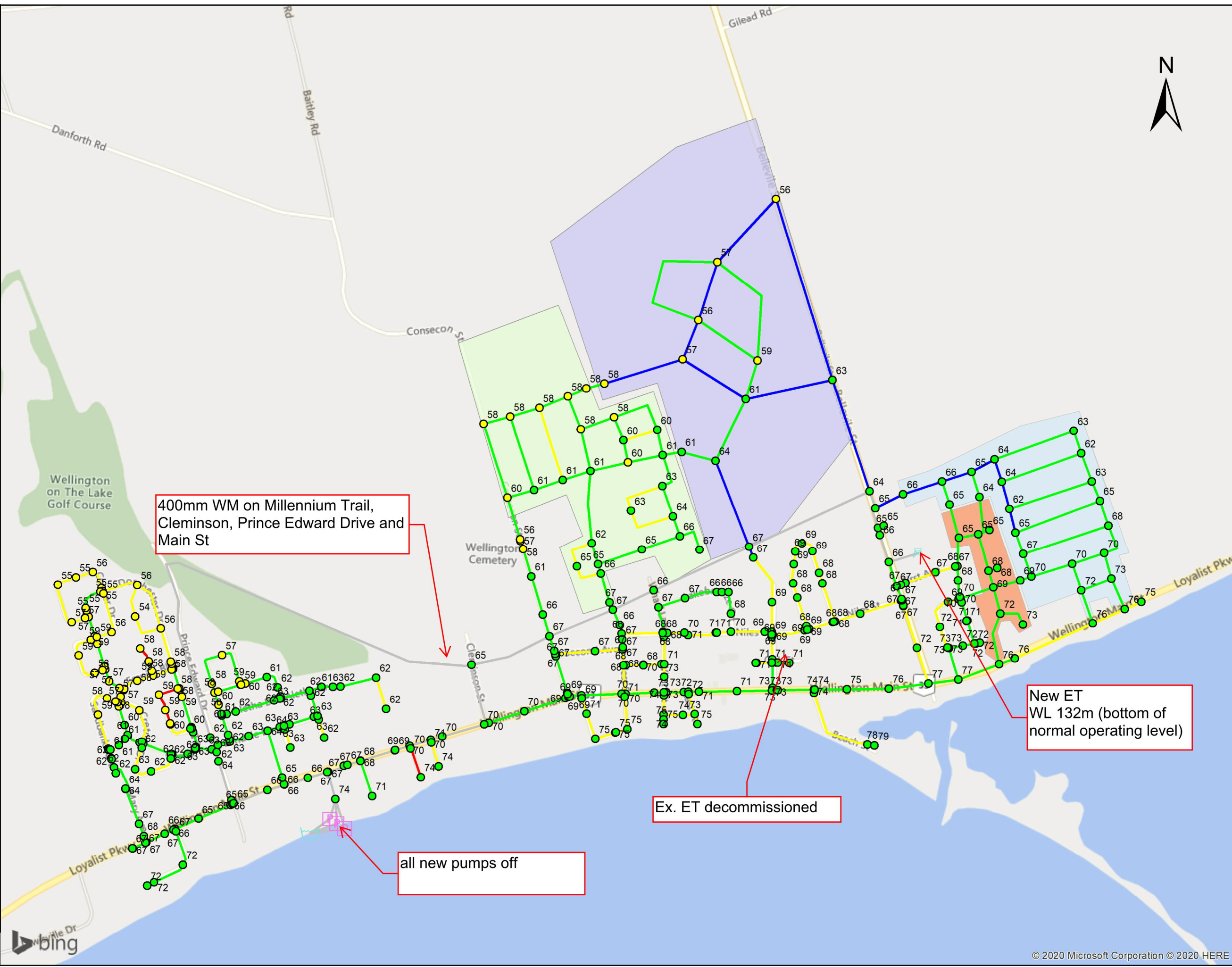
Pressure during Peak Hour Demand

Legend

Node Pressure (psi)	Pipe Diameter (mm)
● < 40	— <=100
● 40 ~ 60	— 150
● 60 ~ 80	— 200
● > 80	— 250
	— 300
	— 400

Developments

- Fields of Wellington
- Country Club Estates
- Lakeside Estates
- Wellington Bay Estates



400mm WM on Millennium Trail, Cleminson, Prince Edward Drive and Main St

New ET WL 132m (bottom of normal operating level)

Ex. ET decommissioned

all new pumps off



RVA Project Number: 183934
Date: June 2020

Figure A-2: Wellington Water Model

New ET, New Millennium Trail Watermain; HLPs Off

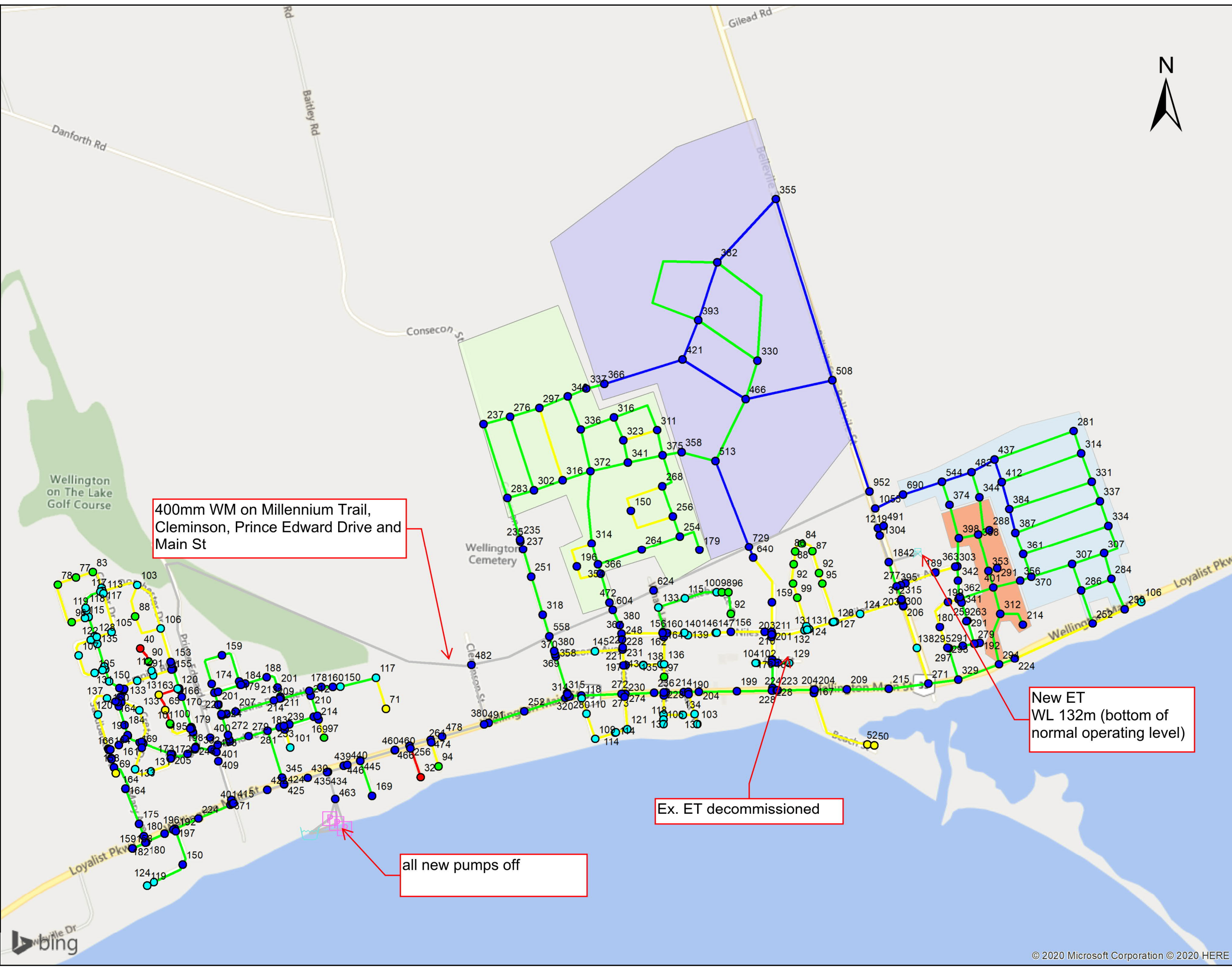
Available Fire Flows during Max Day Demand

Legend

Node FF (L/s)	Pipe Diameter (mm)
● < 50	— <=100
● 50 ~ 75	— 150
● 75 ~ 100	— 200
● 100 ~ 150	— 250
● > 150	— 300
	— 400

Developments

■	Fields of Wellington
■	Country Club Estates
■	Lakeside Estates
■	Wellington Bay Estates



RVA Project Number: 183934
Date: June 2020

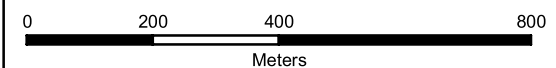


Figure A-2: Wellington Water Model

New ET, New Millennium Trail Watermain; HLPs Off

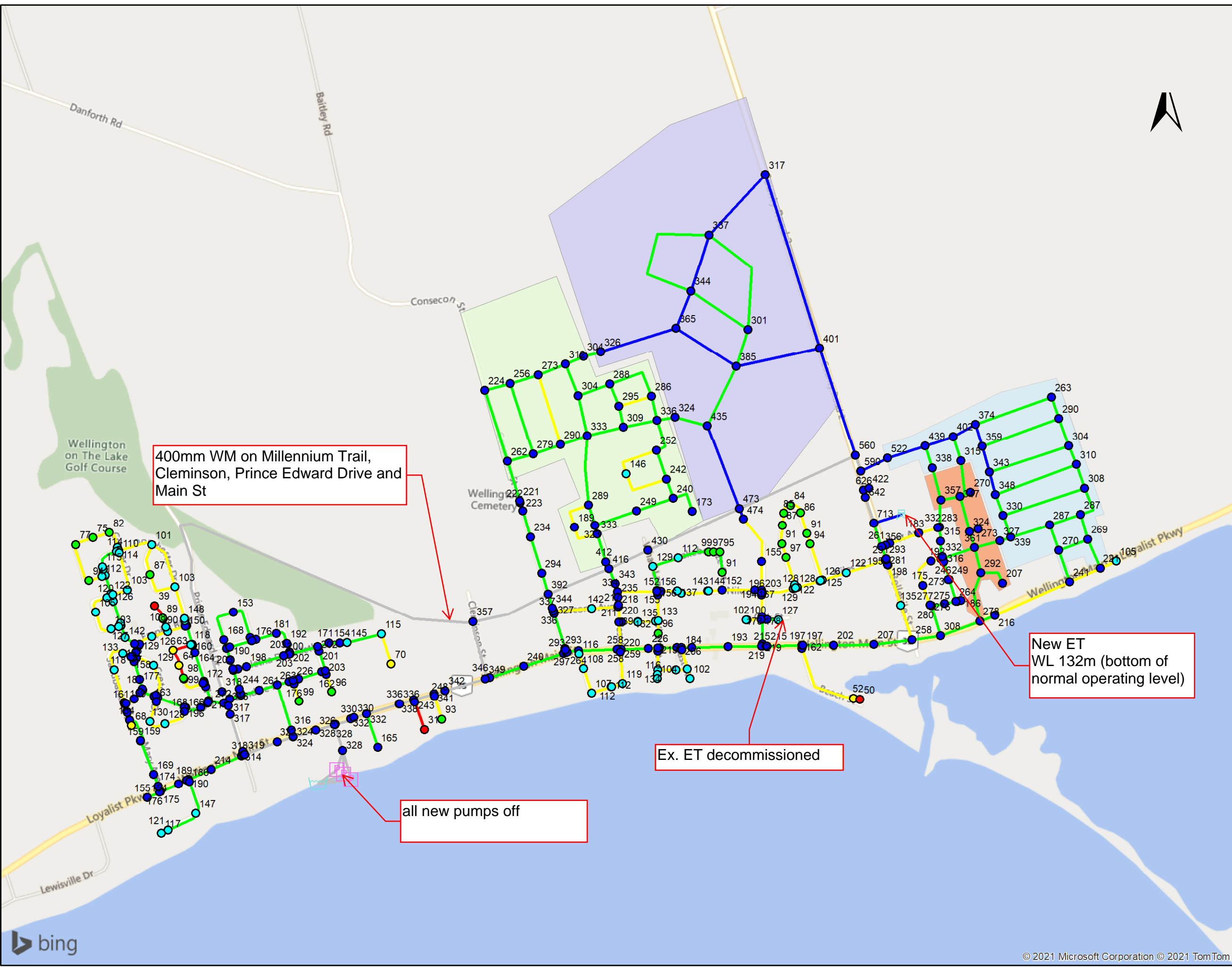
Available Fire Flows during Max Day Demand

Legend

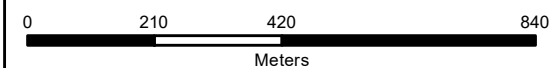
Node FF (L/s)	Pipe Diameter (mm)
● < 50	— <=100
● 50 ~ 75	— 150
● 75 ~ 100	— 200
● 100 ~ 150	— 250
● > 150	— 300
	— 400

Developments

- Fields of Wellington
- Country Club Estates
- Lakeside Estates
- Wellington Bay Estates



RVA Project Number: 183934
Date: June 2020



Appendix C

MTO IDF Curve

Figure 2: Pre-Development Catchment Areas

Figure 3: Post-Development Catchment Areas (Storm Sewers)

Figure 4: Post-Development Runoff Coefficients

Composite Runoff Coefficient Calculations

Storm Sewer Design Sheet

Modified Rational Method Calculations

Outlet Stage-Storage Discharge Calculations

Outlet Swale Capacity Calculations

CDS Estimated Net Annual Solids Load Reduction

Active coordinate

43° 56' 44" N, 77° 23' 15" W (43.945833,-77.387500)

Retrieved: Mon, 06 Jun 2022 20:39:30 GMT



Location summary

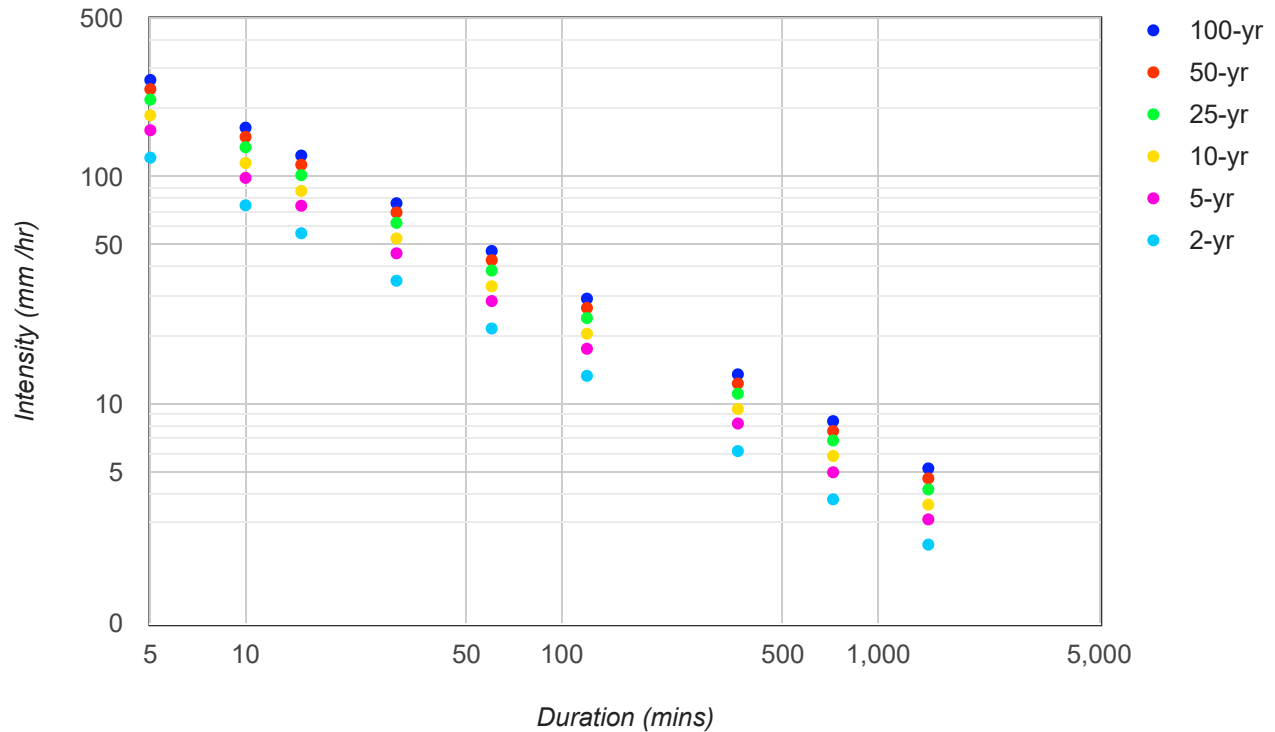
These are the locations in the selection.

IDF Curve: 43° 56' 44" N, 77° 23' 15" W (43.945833,-77.387500)

Results

An IDF curve was found.

Coordinate: 43.945833, -77.387500
IDF curve year: 2022



Coefficient summary**IDF Curve:** 43° 56' 44" N, 77° 23' 15" W (43.945833,-77.387500)

Retrieved: Mon, 06 Jun 2022 20:39:30 GMT

Data year: 2010**IDF curve year:** 2022**Statistics****Rainfall intensity (mm hr⁻¹)**

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	121.6	75.0	56.5	34.9	21.5	13.3	6.2	3.8	2.4
5-yr	160.7	99.1	74.7	46.1	28.4	17.5	8.2	5.0	3.1
10-yr	186.9	115.2	86.8	53.6	33.0	20.4	9.5	5.9	3.6
25-yr	219.3	135.2	101.9	62.8	38.7	23.9	11.1	6.9	4.2
50-yr	243.7	150.2	113.2	69.8	43.0	26.5	12.3	7.6	4.7
100-yr	267.5	164.9	124.3	76.6	47.2	29.1	13.5	8.4	5.2

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	10.1	12.5	14.1	17.4	21.5	26.6	37.2	45.6	57.6
5-yr	13.4	16.5	18.7	23.1	28.4	35.0	49.2	60.0	74.4
10-yr	15.6	19.2	21.7	26.8	33.0	40.8	57.0	70.8	86.4
25-yr	18.3	22.5	25.5	31.4	38.7	47.8	66.6	82.8	100.8
50-yr	20.3	25.0	28.3	34.9	43.0	53.0	73.8	91.2	112.8
100-yr	22.3	27.5	31.1	38.3	47.2	58.2	81.0	100.8	124.8

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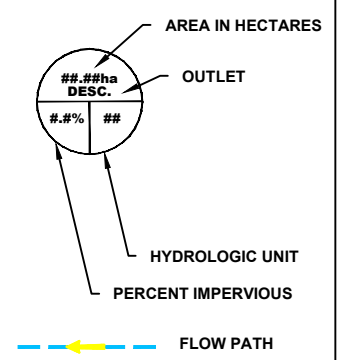
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Last Modified: September 2016



ADJACENT LANDS OWNED BY DEVELOPER COUNTRY CLUB ESTATES



Benchmark

No.	Revision/Issue	Date

Forefront Engineering Inc
 1329 Gardiners Road, Suite 210
 Kingston, ON, Canada K7P 0L8
 613.634.9009 tel.
 1.866.884.9392 fax.

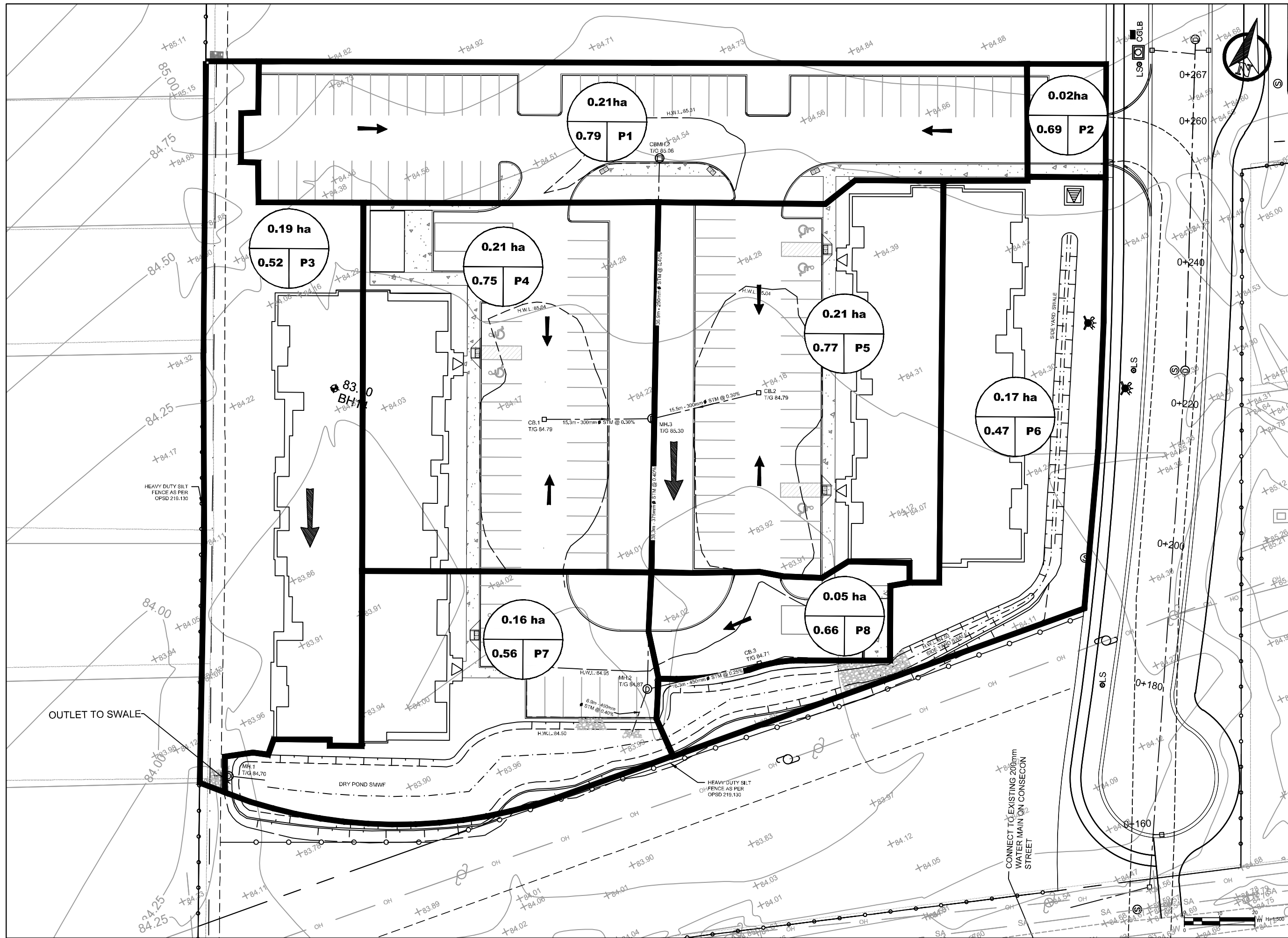
Client
KAITLIN CORPORATION


Project
THE FIELDS OF WELLINGTON

Drawing
PRE-DEVELOPMENT CATCHMENT AREAS

Drawn by: JH	Checked by: JH	Project No.:
Designed by: KMN	Approved by: KMN	Drawing No.:
Date: JUNE 2022	Date:	
Scale: 1:6000	Date:	

FIG.2





AREA IN HECTARES

RUNOFF COEFFICIENT

CATCHMENT AREA ID

MINOR FLOW PATH

MAJOR FLOW PATH

No.	Revision/Issue	Date

Benchmark

1. BENCHMARK BEING THE SPIKE IN CONCRETE SIDEWALK, NORTHING: 486936.964 EASTING: 311081.430, HAVING AN ELEVATION OF 84.39m.

2. BM2 BEING NAIL IN HYDRO POLE, NORTHING: 486936.270 EASTING: 311425.870, HAVING AN ELEVATION OF 85.15

Client

KAITLIN CORPORATION

Project

FIELDS OF WELLINGTON BLOCK 15

Drawing

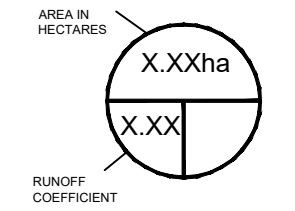
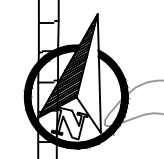
POST DEVELOPMENT AREAS (STORM SEWERS)

Drawn by: CGD	Checked by: KMN	Project No.:
Designed by: KMN	Approved by: KMN	Drawing No.:

Date: MAY 2023

Scale: 1:500 (11x17)

FIG. 3



- ROOFTOP = 0.29 ha
- SIDEWALK, CURB, PARKING LOT = 0.47 ha
- LANDSCAPING = 0.47 ha

Benchmark
 1. BENCH MARK BEING THE SPIKE IN CONCRETE SIDEWALK, NORTHING: 4869392.964 EASTING: 311081.429, HAVING AN ELEVATION OF 84.38m.
 2. BM2 BEING NAIL IN HYDRO POLE, NORTHING: 4869392.270 EASTING: 311425.870, HAVING AN ELEVATION OF 85.15

No.	Revision/Issue	Date



1329 Gardiners Road, Suite 210
 Kingston, ON, Canada K7P 0L8
 613.634.9009 tel.
 1.866.884.9392 fax.

Client
KAITLIN CORPORATION

Project
**FIELDS OF WELLINGTON
 BLOCK 15**

Drawing
**POST DEVELOPMENT
 RUNOFF COEFFICIENT**

Drawn by: CGD	Checked by: JH	Project No.
Designed by: KMN	Approved by: KMN	Drawing No.
Date: AUG 2023		FIG. 4
Scale: 1:500 (1/4"=1')		

	Impervious Areas (ha)	Pervious Areas (ha)	Run-off Coefficient	C x A
Rooftops	0.29		0.90	0.26
Sidewalk, Curb, Asphalt	0.47		0.90	0.42
Landscaping		0.47	0.25	0.12
			Weighted Run-off =	0.65

Composite Runoff Coefficient P1			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.174	0.9	
Pervious Areas	0.035	0.25	
Total	0.21	0.79	

Composite Runoff Coefficient P2			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.014	0.9	
Pervious Areas	0.006	0.25	
Total	0.02	0.69	

Composite Runoff Coefficient P3			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.077	0.9	
Pervious Areas	0.111	0.25	
Total	0.19	0.52	

Composite Runoff Coefficient P4			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.161	0.9	
Pervious Areas	0.049	0.25	
Total	0.21	0.75	

Composite Runoff Coefficient P5			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.174	0.9	
Pervious Areas	0.042	0.25	
Total	0.22	0.77	

Composite Runoff Coefficient P6			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.06	0.9	
Pervious Areas	0.116	0.25	
Total	0.18	0.47	

Composite Runoff Coefficient P7			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.076	0.9	
Pervious Areas	0.084	0.25	
Total	0.16	0.56	

Composite Runoff Coefficient P8			
Wellington Lowrise Block 15			
Hydrologic Units - Post Conditions			
Drainage Area No.	Total Area (ha)	Runoff Coefficient -C	Description
Impervious Areas	0.031	0.9	
Pervious Areas	0.018	0.25	
Total	0.05	0.66	

Total		0.65	
-------	--	------	--

STORM SEWER DESIGN SHEET - BLOCK 15 - MINOR EVENT

CLIENT: KAITLIN CORPORATION
 PROJECT NAME: Wellington Block 15
 DATE: August 2023

Min. V = 0.75 m/s
 Max. V = 6 m/s

DESIGN FREQUENCY: MTO Look up
 RAINFALL STATIONS: 0.013
 DESIGNED 'n':

LOCATION:				STORM SEWER CATCHMENT AREA = 1.22 ha								RUNOFF 3			PIPE SELECTION													
Area (ha)	Inlet Description	FROM	TO	R =	R =	R =	R =	R =	R =	High Density	Minor Event		Time of Conc. (min)	Minor Event Intensity I (mm/hr)	Peak Flow Q (L/S)	Type of Pipe	Required Pipe Diameter D (m)	Nominal Diameter D (mm)	Pipe Length (m)	Grade S	Full Capacity (L/S)	Full Flow Velocity V (m/s)	Time of Flow (min)	Capacity Used Q/Q(f)	Actual Velocity (m/s)	Normal Depth (mm)	Free Outfall D/S HGL (m)	Fall in Sewer (m)
				0.25 ha	0.45 ha	0.50 ha	0.65 ha	0.75 ha	0.85 ha	R = 0.90 ha	Indiv. 2.78AC ha	Accum. 2.78AC ha																
0.21	P1	CBMH2	MH3					0.210			0.438	0.438	15.0	75	33	HDPE	0.237	250	37	0.40%	37.6	0.77	0.80	0.87	0.86	180	0.18	0.148
0.21	P4	CB1	MH3					0.210			0.438	0.438	15.0	75	33	HDPE	0.250	300	15	0.30%	53.0	0.75	0.34	0.62	0.79	170	0.17	0.046
0.21	P5	CB2	MH3					0.210			0.438	0.438	15.0	75	33	HDPE	0.250	300	16	0.30%	53.0	0.75	0.34	0.62	0.79	170	0.17	0.047
			MH3								0.000	1.313	15.8	72	95	HDPE	0.354	375	38	0.40%	110.9	1.00	0.64	0.85	1.13	265	0.27	0.153
0.05	P8	CB3	MH2				0.050				0.090	0.090	15.0	75	7	HDPE	0.143	450	16	0.25%	142.6	0.90	0.30	0.05	0.46	66	0.07	0.041
0.16	P7	MH2	Site Pond			0.160					0.222	1.625	16.4	70	114	HDPE	0.379	450	7	0.40%	180.3	1.13	0.10	0.63	1.20	258	0.26	0.028
0.19	West Swale (P3)	West Swale	Swale Outlet			0.190					0.264	0.264	15.0	75	20													
0.02	P2	P2	Sub. SWMF				0.020				0.036	0.036	15.0	75	3													
0.17	East Swale (P6)	East Swale	Site Pond			0.170					0.236	0.536	15.0	75	40													

Notes:
 1. Intensity Calculation from MTO Look up Curve provided below

$$i = \frac{A}{(t_d + B)^c}$$

Example

15min	74.8 mm/hr
20min	61.3 mm/hr
25min	52.5 mm/hr
30min	46.2 mm/hr

l = rainfall intensity, mm/h A = 499.354
 t_d = duration, min B = 0.10
 A, B, C are constants c = 0.69917

DRY POND STORMWATER MANAGEMENT FACILITY
 MODIFIED RATIONAL METHOD CALCULATIONS &
 STORAGE VOLUMES FOR SMALL SITES

Project: Fields of Wellington Block 15
 Date: Aug 2023

5 Year Return Period

Pre-development Runoff	
Q (m ³ /s)	0.043

Post-development Characteristics			
Description	C	Area (ha)	CxA
Total	0.65	1.22	0.79

Post-development Peak Flow	
C	0.65
t _c (min)	15.00
Area (ha)	1.22
Intensity (mm/hr)	74.83
Q (m ³ /s)	0.166

Duration (td- min)	Intensity (mm/hr)	CxA	Q _p - Uncontrolled Runoff Rate (m ³ /s)	Q _d - Allowable Outflow (m ³ /s)	Peak Storage Rate (m ³ /s)	Storage Volume (m ³)	Comments
15	74.83	0.79	0.166	0.043	0.123	111.1	
20	61.48	0.79	0.137	0.043	0.094	119.0	
25	52.60	0.79	0.117	0.043	0.074	124.0	
30	46.31	0.79	0.103	0.043	0.060	127.4	
35	41.58	0.79	0.092	0.043	0.050	129.8	
40	37.87	0.79	0.084	0.043	0.041	131.4	
45	34.88	0.79	0.077	0.043	0.035	132.2	
50	32.40	0.79	0.072	0.043	0.029	132.6	Maximum Storage Volume
55	30.31	0.79	0.067	0.043	0.025	132.4	
60	28.52	0.79	0.063	0.043	0.021	131.9	
65	27.23	0.79	0.060	0.043	0.018	133.4	
70	25.84	0.79	0.057	0.043	0.015	132.1	
75	24.61	0.79	0.055	0.043	0.012	130.6	
80	23.51	0.79	0.052	0.043	0.010	128.9	

Outlet	Orifice / Pipe Diameter (m)	Water Surface Elevation (m)	Invert of Orifice (m)	Centreline of Orifice Elevation (m)	Head (m)	Release Rate (m ³ /s)	Required Release (m ³ /s)	Velocity m/s	Comments
Orifice 1	0.160	84.34	83.700	83.780	0.560	0.043	0.043	2.15	5 Year Orifice

Note: Refer to Stage - Storage Discharge Calculations in Appendix C for further details

Formulas:

I= MTO IDF Curve

$$Q = 0.0028 * C * I * A$$

$$S_d = Q_p t_d - Q_d ((t_d + t_c) / 2)$$

*Storage Formula (Aron and Kibler, 1990)

Where:

Q=Peak runoff rate (m³/s)

t_d = Duration of Storm (min)

C=Composite runoff coefficient

Q_p = Peak Flow (m³/s)

I=Rainfall intensity (mm/hr)

Q_d = Discharge Rate (m³/s)

A=Drainage area (ha)

S_d = Required Storage Volume (m³)

t_c= Time of Concentration (min)

Orifice Equation

$$Q = 0.65 A (2gH)^{1/2}$$

where A = orifice area; g = gravity; and H=head above centre of orifice (m)

DRY POND STORMWATER MANAGEMENT FACILITY
 MODIFIED RATIONAL METHOD CALCULATIONS &
 STORAGE VOLUMES FOR SMALL SITES

Project: Fields of Wellington Block 15
 Date: Aug 2023

100 Year Return Period

Pre-development Runoff	
Q (m ³ /s)	0.071

Post-development Characteristics			
Description	C	Area (ha)	CxA
Total	0.65	1.22	0.79

Post-development Peak Flow	
C	0.65
t _c (min)	15.00
Area (ha)	1.22
Intensity (mm/hr)	123.69
Q (m ³ /s)	0.275

Duration (td- min)	Intensity (mm/hr)	CxA	Q _p - Uncontrolled Runoff Rate (m ³ /s)	Q _d - Allowable Outflow (m ³ /s)	Peak Storage Rate (m ³ /s)	Storage Volume (m ³)	Comments
15	123.69	0.79	0.275	0.071	0.204	183.5	
20	101.65	0.79	0.226	0.071	0.155	196.5	
25	86.98	0.79	0.193	0.071	0.122	204.7	
30	76.59	0.79	0.170	0.071	0.099	210.5	
35	68.77	0.79	0.153	0.071	0.082	214.5	
40	62.65	0.79	0.139	0.071	0.068	217.0	
45	57.70	0.79	0.128	0.071	0.057	218.5	
50	53.61	0.79	0.119	0.071	0.048	219.0	Maximum Storage Volume
55	50.16	0.79	0.111	0.071	0.041	218.8	
60	47.20	0.79	0.105	0.071	0.034	218.0	
65	45.07	0.79	0.100	0.071	0.029	220.4	
70	42.77	0.79	0.095	0.071	0.024	218.3	
75	40.73	0.79	0.090	0.071	0.020	215.8	
80	38.92	0.79	0.086	0.071	0.016	213.0	

Outlet	Orifice / Pipe Diameter (m)	Water Surface Elevation (m)	Invert of Orifice (m)	Centreline of Orifice Elevation (m)	Head (m)	Release Rate (m ³ /s)	Required Release (m ³ /s)	Velocity m/s	Comments
Orifice 2	0.195	84.48	83.700	83.798	0.683	0.071	0.071	2.38	100 Year Orifice

Note: Refer to Stage - Storage Discharge Calculations in Appendix C for further details

Formulas:

I= MTO IDF Curve

$$Q = 0.0028 * C * I * A$$

$$S_d = Q_p t_d - Q_d ((t_d + t_c) / 2)$$

*Storage Formula (Aron and Kibler, 1990)

Where:

Q=Peak runoff rate (m³/s)

C=Composite runoff coefficient

I=Rainfall intensity (mm/hr)

A=Drainage area (ha)

t_c= Time of Concentration (min)

t_d = Duration of Storm (min)

Q_p = Peak Flow (m³/s)

Q_d = Discharge Rate (m³/s)

S_d = Required Storage Volume (m³)

Orifice Equation

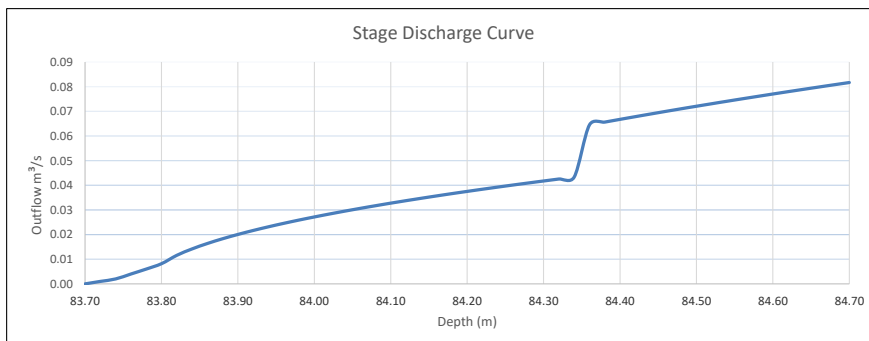
$$Q = 0.65 A (2gH)^{1/2}$$

where A = orifice area; g = gravity; and H=head above centre of orifice (m)

Block 15 - Outlet Stage - Storage Discharge Calculations

Orifice 1 Equation	$0.65 \cdot A \cdot (2gh)^{1/2}$
Orifice Invert	83.70 m
Orifice Size	0.160 m
Centreline	83.78 m
Orifice 2 Equation	$0.65 \cdot A \cdot (2gh)^{1/2}$
Orifice Invert	83.70 m
Orifice Size	0.195 m
Centreline	83.80 m

Height (m)	Water Surface Elevation (m)	Inc. Depth (m)	Dry Pond Volume (m ³)	Pipe Volume (m ³)	Total Volume (m ³)	Orifice 1 Release Rate (m ³ /s)	Orifice 2 Release Rate (m ³ /s)	Total Release Rate (m ³ /s)	Notes
0.000	83.70	0.000	1	0	1	0.000		0.000	
0.020	83.72	0.020	2	0	2	0.001		0.001	
0.040	83.74	0.020	2	0	2	0.002		0.002	
0.060	83.76	0.020	3	0	3	0.004		0.004	
0.080	83.78	0.020	4	0	5	0.006		0.006	
0.100	83.80	0.020	5	1	6	0.008		0.008	
0.120	83.82	0.020	7	1	8	0.012		0.012	
0.140	83.84	0.020	8	2	10	0.014		0.014	
0.160	83.86	0.020	9	2	12	0.016		0.016	
0.180	83.88	0.020	11	3	14	0.018		0.018	
0.200	83.90	0.020	13	3	16	0.020		0.020	
0.220	83.92	0.020	15	4	19	0.022		0.022	
0.240	83.94	0.020	17	4	21	0.023		0.023	
0.260	83.96	0.020	20	5	24	0.025		0.025	
0.280	83.98	0.020	22	5	27	0.026		0.026	
0.300	84.00	0.020	25	6	31	0.027		0.027	
0.320	84.02	0.020	28	6	34	0.028		0.028	
0.340	84.04	0.020	32	6	38	0.030		0.030	
0.360	84.06	0.020	35	7	42	0.031		0.031	
0.380	84.08	0.020	39	7	47	0.032		0.032	
0.400	84.10	0.020	44	8	52	0.033		0.033	
0.420	84.12	0.020	48	8	57	0.034		0.034	
0.440	84.14	0.020	53	9	62	0.035		0.035	
0.460	84.16	0.020	58	9	68	0.036		0.036	
0.480	84.18	0.020	64	10	74	0.037		0.037	
0.500	84.20	0.020	70	10	80	0.038		0.038	
0.520	84.22	0.020	76	11	87	0.038		0.038	
0.540	84.24	0.020	83	11	94	0.039		0.039	
0.560	84.26	0.020	90	12	101	0.040		0.040	
0.580	84.28	0.020	97	12	109	0.041		0.041	
0.600	84.30	0.020	105	12	118	0.042		0.042	
0.620	84.32	0.020	113	13	126	0.043		0.043	
0.640	84.34	0.020	122	13	136	0.043		0.043	5 Year Event
0.660	84.36	0.020	131	14	145		0.064	0.064	Top of Inner
0.680	84.38	0.010	141	14	155		0.066	0.066	Conc. Insert
0.700	84.40	0.020	151	14	165		0.067	0.067	
0.720	84.42	0.020	162	14	176		0.068	0.068	
0.740	84.44	0.020	173	14	187		0.069	0.069	
0.760	84.46	0.020	185	14	199		0.070	0.070	
0.780	84.48	0.020	197	14	211		0.071	0.071	100 Year Event
0.800	84.50	0.020	210	14	224		0.072	0.072	Freeboard
0.820	84.52	0.020	223	14	237		0.073	0.073	
0.840	84.54	0.020	237	14	251		0.074	0.074	
0.860	84.56	0.020	251	14	265		0.075	0.075	
0.880	84.58	0.020	266	14	280		0.076	0.076	
0.900	84.60	0.020	282	14	296		0.077	0.077	
0.920	84.62	0.020	298	14	312		0.078	0.078	
0.940	84.64	0.020	315	14	329		0.079	0.079	
0.960	84.66	0.020	333	14	347		0.080	0.080	
0.980	84.68	0.020	351	14	365		0.081	0.081	
1.000	84.70	0.020	370	14	384		0.082	0.082	Freeboard



Channel Report

Outlet Swale Calculations

Triangular

Side Slopes (z:1) = 3.0000, 3.0000
Total Depth (m) = 0.4000

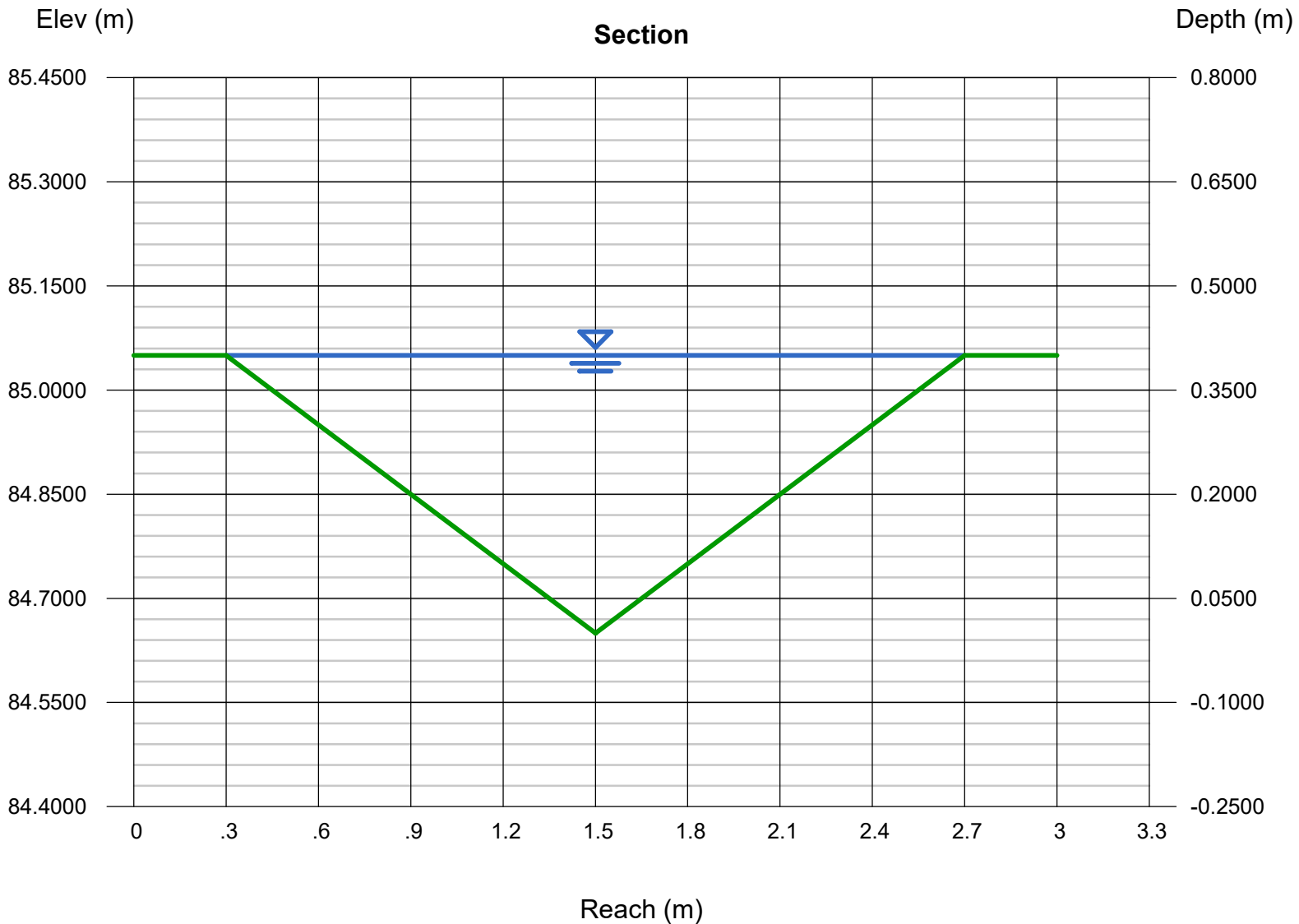
Invert Elev (m) = 84.6500
Slope (%) = 0.5000
N-Value = 0.035

Calculations

Compute by: Q vs Depth
No. Increments = 10

Highlighted

Depth (m) = 0.4000
Q (cms) = 0.3202
Area (sqm) = 0.4800
Velocity (m/s) = 0.6670
Wetted Perim (m) = 2.5298
Crit Depth, Yc (m) = 0.2987
Top Width (m) = 2.4000
EGL (m) = 0.4227





**CDS ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON THE RATIONAL RAINFALL METHOD
BASED ON A FINE PARTICLE SIZE DISTRIBUTION**



Project Name: Fields of Wellington, Block 15	Engineer: Forefront Engineering Inc.
Location: Wellington	Contact: Chris DeCecco
OGS #: OGS 1	Report Date: 30-Aug-23

Area 0.9 ha	Rainfall Station # 214	
Weighted C 0.7	Particle Size Distribution FINE	
CDS Model 2020	CDS Treatment Capacity 31 l/s	

<u>Rainfall Intensity¹</u> (mm/hr)	<u>Percent Rainfall Volume¹</u>	<u>Cumulative Rainfall Volume</u>	<u>Total Flowrate (l/s)</u>	<u>Treated Flowrate (l/s)</u>	<u>Operating Rate (%)</u>	<u>Removal Efficiency (%)</u>	<u>Incremental Removal (%)</u>
0.5	9.9%	9.9%	0.9	0.9	2.8	98.1	9.7
1.0	10.8%	20.7%	1.8	1.8	5.6	97.2	10.5
1.5	10.1%	30.8%	2.6	2.6	8.4	96.4	9.8
2.0	9.1%	39.9%	3.5	3.5	11.2	95.6	8.7
2.5	7.0%	46.9%	4.4	4.4	14.1	94.8	6.7
3.0	6.9%	53.9%	5.3	5.3	16.9	94.0	6.5
3.5	4.5%	58.4%	6.1	6.1	19.7	93.2	4.2
4.0	4.5%	62.9%	7.0	7.0	22.5	92.4	4.2
4.5	4.1%	67.0%	7.9	7.9	25.3	91.6	3.7
5.0	3.8%	70.8%	8.8	8.8	28.1	90.8	3.5
6.0	5.7%	76.5%	10.5	10.5	33.7	89.2	5.0
7.0	4.5%	81.0%	12.3	12.3	39.4	87.6	4.0
8.0	3.6%	84.5%	14.0	14.0	45.0	86.0	3.1
9.0	2.3%	86.8%	15.8	15.8	50.6	84.4	1.9
10.0	1.9%	88.7%	17.5	17.5	56.2	82.7	1.6
15.0	6.1%	94.8%	26.3	26.3	84.3	74.7	4.6
20.0	2.6%	97.5%	35.0	31.2	100.0	62.4	1.6
25.0	2.0%	99.4%	43.8	31.2	100.0	49.9	1.0
30.0	0.4%	99.9%	52.5	31.2	100.0	41.6	0.2
35.0	0.1%	100.0%	61.3	31.2	100.0	35.7	0.0
40.0	0.0%	100.0%	70.1	31.2	100.0	31.2	0.0
45.0	0.0%	100.0%	78.8	31.2	100.0	27.7	0.0
50.0	0.0%	100.0%	87.6	31.2	100.0	25.0	0.0

90.4

Removal Efficiency Adjustment² = 6.5%
Predicted Net Annual Load Removal Efficiency = 83.9%
Predicted % Annual Rainfall Treated = 98.9%

1 - Based on 44 years of hourly rainfall data from Canadian Station 6104175, Kingston ON
 2 - Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.
 3 - CDS Efficiency based on testing conducted at the University of Central Florida
 4 - CDS design flowrate and scaling based on standard manufacturer model & product specifications