FUNCTIONAL SERVICING REPORT
DRAKE DEVONSHIRE AND MIDTOWN SITE PLANS

July 16, 2021

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1 Background

Jewell Engineering Inc. (Jewell) was retained by Drake Hotel Properties (Drake) to assist with the planning applications to provide additional hotel accommodations to both the Drake Devonshire Hotel and the Drake Motor Inn. The sites are both located on Wharf Street in Wellington, Ontario. The Devonshire is located south of Main Street while the Motor Inn is at the Midtown site north of Main Street (Figure 1-1). The properties are both serviced with municipal water and sanitary sewer. This report was submitted to support the proposed hotel modifications.

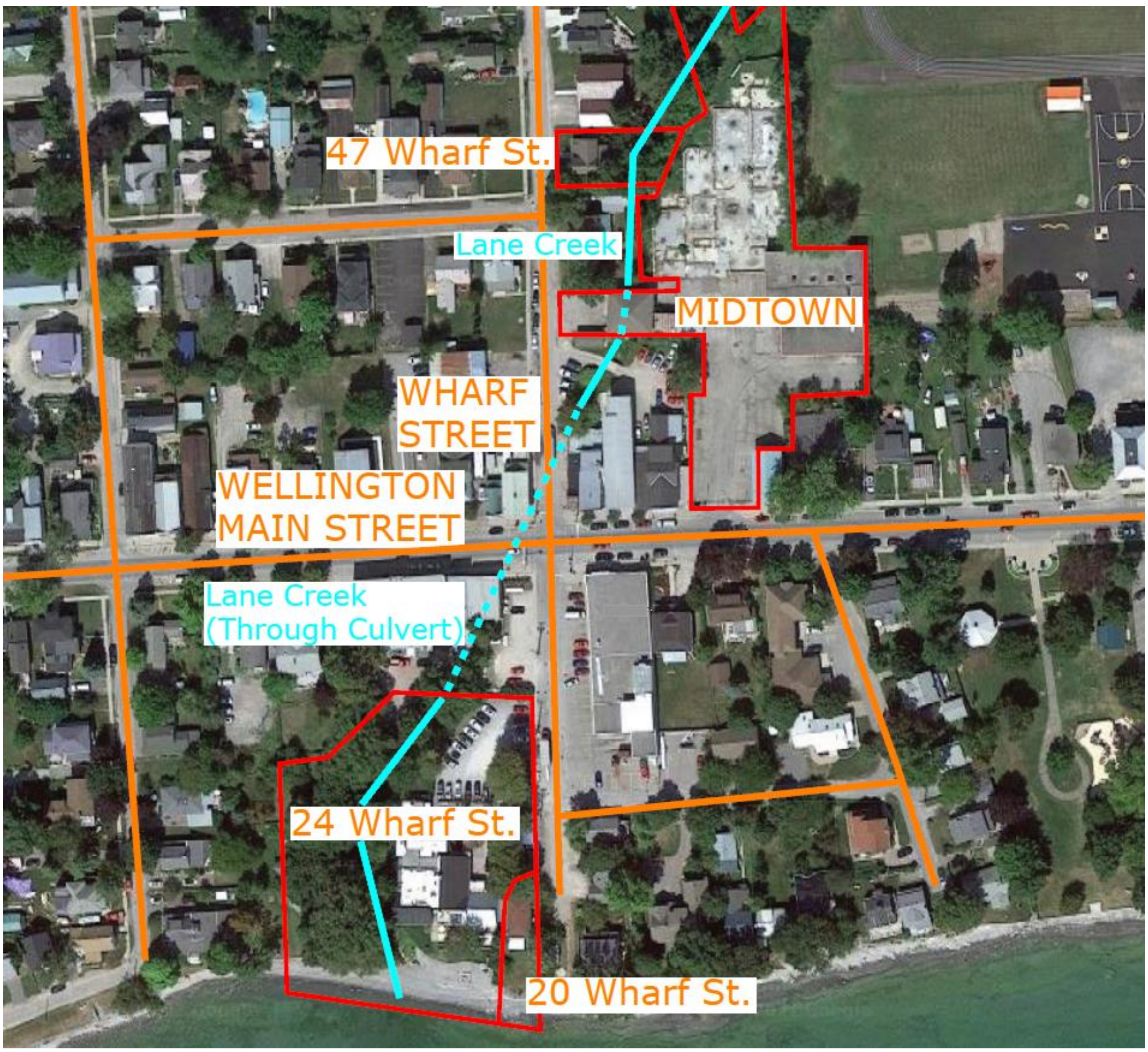

Figure 1-1: Site Location

The following services have been reviewed as part of this functional servicing report:

- Water Distribution
- Sanitary Servicing
- Stormwater Management (separate cover)
- Traffic (separate cover)

1.1 Site Description

Drake owns and operates two sites on Wharf Street to provide short term hotel accommodations.

The Drake Devonshire:

The Devonshire is located at 24 Wharf Street and includes a 12-unit inn and restaurant. The Devonshire has 12 rooms and a restaurant that operates throughout the year.

The Drake Motor Inn:

The Motor Inn is part of the Midtown Site and includes two multi-unit short term hotel buildings and is located at 43 and 45 Wharf St, north of Main Street. The Midtown site includes the Motor Inn, the Midtown Brewery and additional parking for the Devonshire.

Lane Creek flows through both site and constrains the two developments.

1.2 Proposed Site Development

The Drake Devonshire:

The Drake is proposing to expand the Devonshire Inn with a building addition that would replace the single-family dwelling located at 20 Wharf Street. The proposed addition will increase the number of units from 12 to 27 rooms and add guest capacity to the restaurant and event space equivalent to 50 additional seats.

The Drake Motor Inn:

The Motor Inn site operates at 45 Wharf Street and is also proposed to be expanded. The existing single-family dwelling on 47 Wharf Street and accessory building at 45 Wharf Street will be demolished and replaced with a 6-unit inn.

It is assumed the existing service connection will be maintained for the new structure. The new 6-unit inn will form part of the Drake Motor Inn.

The review of the available servicing was completed using the specifications outlined by the following:

- Ministry of Environment, Conservation, and Parks (MECP)
    - Design Guidelines for Drinking-Water Systems, 2008
    - Design Guidelines for Sewage Works, 2008
- Ontario Building Code, O Reg 332/12
    - Table 8.2.1.3.A.

2 Water Distribution System

A 150mm ductile iron watermain services Wharf Street. The single-family dwellings have a 19mm service connection while the Devonshire Inn is serviced with a 38mm connection. The 19mm connection currently servicing 20 Wharf Street will be abandoned during the demolition of the single-family dwelling.

Jewell requested flow data from PEC staff for the main along Wharf Street. Hydrant flow test data from June 2019 was provided for hydrant 49#2038 located just outside the Drake Hotel on Wharf Street.

The flow test concluded the hydrant has a flow rate of 671 US gpm (42.2 L/s) at 20 psi. The flow test data did not include static pressures or any information on the pitot hydrant.

Static pressures of 55 psi under Peak Hour conditions were estimated by Dillon in a recent modelling investigation in 2019. Figure 2-1 shows the results of their modelling.

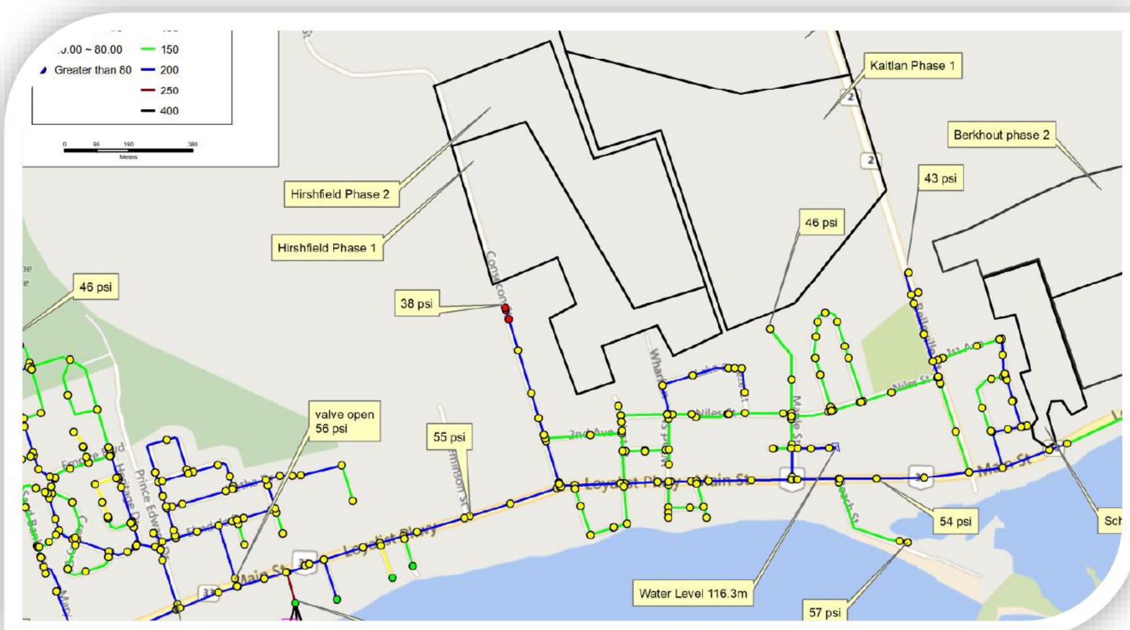

Figure 2-1: Existing Water System Static Pressures, Peak Hour Conditions (From Dillon, 2019)

Dillon also found the Wellington system has a Maximum Day Peak Factor of 2.36 for residential and 2.5 for commercial. Peak Hour Factor is 3.59 for both. See Table 2-1 below.

Table 2-1: Peaking Factors – Residential and Commercial (from Dillon 2019)

TABLE 2: HYDRAULIC MODEL WATER DEMAND DESIGN BASIS

Demand Scenario	Residential	Commercial
ADD	350 L/c/d	1.15 L/s/ha
MDD peak factor (xADD)	2.36	2.50
PHD peak factor (xADD)	3.59	3.59

2.1 Design Criteria

Jewell reviewed the water usage as per the 2008 MOE Drinking Water System Guidelines and the Ontario Building Code:

- Average Daily Residential Domestic Flows: 350 L/d*cap
- Population per Household 3.5 persons
- Average Daily Use for Hotels: 250 L/d*room
- Average Daily Use for Restaurants: 125 L/d*seat

MOE, 2008 provides methodologies to estimate water usage of restaurants based on seating capacity. However, the Drake Devonshire has an assembly of different indoor and outdoor assembly areas with a theoretical seating capacity that is difficult to enumerate. Drake provided capacities as below, but felt actual seating would be approximately **50%** of the total tally. This would convert to about 209 seating capacity.

- Indoor Room Capacity 168
- Outdoor Room Capacity 250

Historical water usage data is preferred for commercial and institutional water demands as per Section 3.4.3 of the 2008 MOE guidelines. Jewell reviewed the historical water usage for the Devonshire Inn and Restaurant year-round to develop an average day water demand for the establishment from 2017-2019. The review of these records concluded the highest monthly average demand (851.5 m^3) was recorded in June. All water usage records were dated before the COVID-19 pandemic to accurately represent the expected demand of the Devonshire.

The data was converted to a daily usage in the table below. The hotel portion was assumed using the 250L/d per room and 12 rooms and the restaurant usage was calculated as the difference between the total usage and the hotel usage.

Table 2-2: Hotel Water Usage Data and Approximate Usages

Monthly Usage (m^3/month)	Total Usage (m^3/day)	Hotel Usage(m^3/day)	Restaurant (m^3/day)
851.5	28.4	3.0	25.4

A peak daily average flow rate 28.4 m^3/day for the restaurant was calculated by using the water data shown in Table 2-3. A usage of 25.4m^3 (25,400L) divided by a per seat usage of 125L/day would correspond to 203 seats. This is slightly less than the 209 person seating estimated by Drake Devonshire.

The review of the historical water usage data clearly outlines a seasonal peak in water usage Monthly usage is seen to increase as much as 8 times from March to June in the same year. Jewell selected the month with the highest monthly average to calculate baseflows. This is a conservative design approach as the baseflows will be inflated to represent peak occupancy with no discount for off peak usage.

2.2 Fire Flow Requirement

The capacity of the Wellington system to provide fire flow is known by PEC staff to be constrained. The available fire flow is 42.2L/s (see Section 2).

The original water treatment facility was designed to provide a fire flow of 37.5L/s (Wellington Water Pressures Staff Report to Committee of the Whole, June 28, 2018). The system currently meets that fire flow requirement.

2.2.1 Devonshire Site

Existing Conditions

The fire flow requirement was calculated for the existing site and for the proposed site improvements using the Ontario Building Code. The Drake is currently serviced with a sprinkler system for fire protection that will be extended into the building addition. The existing single-family dwelling does not have any fire protection controls in place. The tables and figures used to calculate the fire flows can be seen in Appendix A. The existing fire flow condition was selected by taking the maximum of the two minimum required fire flows for the Drake and the single-family dwelling.

Table 2-3: Existing Fire Flow Conditions per O.B.C. – Drake Devonshire

Procedure

1	Building to be Assessed		Existing - Hotel + Restaurant	Existing - Single Family	
2	Building Classification		E	C	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	1007.2	100	m2
		2 - Number of Storeys	3	2	
		3 - Building Height	10.48	9	m
		4 - Firewall Separation?	No	No	
		5 - Sprinkler System?	Yes	No	
		6 - Construction Type	Ordinary	Ordinary	
4	Minimum Supply of Water	K	31	18	Table 1
		V	10555	900	m3
		S_{tot}	2.0	2.0	
		Q	654,438	32,400	L
5	**Minimum Fire Flow**	**Table 2**	**9000**	**2700**	L/min
			150	**45**	L/s

The minimum fire flow requirement for the two sites under existing conditions is the greater of the two structures – 150L/s.

Proposed Conditions

Under proposed conditions, fire flow requirements do not change. The total floor area of the hotel is increased and the total volume of water required to fight the fire is increased, but the fire flow rate is maintained at 150L/s.

Table 2-4: Proposed Fire Flow Conditions per O.B.C.– Drake Devonshire

Procedure

1	Building to be Assessed		Proposed - Hotel + Restaurant	
2	Building Classification		E	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	2163	m2
		2 - Number of Storeys	3	
		3 - Building Height	12	m
		4 - Firewall Separation?	No	
		5 - Sprinkler System?	Yes	
		6 - Construction Type	Ordinary	
4	Minimum Supply of Water	K	31	Table 1
		V	25956	m3
		S_{tot}	2.0	
		Q	1,609,272	L
5	**Minimum Fire Flow**	**Table 2**	**9000**	L/min
			150	L/s

2.2.2 Motor Inn Site

The proposed site modifications include the demolition of a single-family dwelling at 47 Wharf Street and the construction of a 6-unit hotel. The fire flow calculations demonstrate a minor increase in the required fire flow for the site modifications. The calculations are shown below:

Table 2-5: Existing and Proposed Fire Flow Conditions per O.B.C. - 47 Wharf Street

Procedure					
1	Building to be Assessed		Existing - Single Family	Proposed - 6-unit Hotel	
2	Building Classification		C	E	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	100	185	m2
		2 - Number of Storeys	2	3	
		3 - Building Height	9	10	m
		4 - Firewall Separation?	No	No	
		5 - Sprinkler System?	No	No	
		6 - Construction Type	Ordinary	Ordinary	
4	Minimum Supply of Water	K	18	31	Table 1
		V	900	1850	m3
		S_{tot}	2.0	2.0	
		Q	32,400	114,700	L
5	**Minimum Fire Flow**	**Table 2**	**2700**	**3600**	L/min
			45	**60**	L/s

The proposed development will result in an increase in the minimum fire flow requirement. As outlined previously in Section 2.2, the Wellington system is designed for a fire flow of 37.5 L/s and presently there is 42.2 L/s available in the system for fire flow. O.B.C. fire flow calculations lead to a minimum fire flow requirement of 60 L/s. However, given that the Wellington system design fire flow is 37.5 L/s is available in the hydrant, fire flow is satisfied. It is known that Wellington plans to upgrade the water distribution system that will significantly improve the systems fire flow.

2.3 Hydraulic Data

The maximum day demand (MDD) and peak hour demand (PHD) for the hotel and restaurant on the existing and proposed sites were calculated (Appendix B) and summarized in Sections 2.3.1 and 2.3.2. Peaking factors from Table 2-1 were applied to the base demands to calculate the demand during Max Day + Fire Flow and Peak Hour scenarios.

2.3.1 Water Demand – Drake Devonshire

Water demands have been calculated before and after development for the Drake Devonshire Inn. The proposed building addition includes the addition of 15 hotel units and 50 restaurant seats. The calculations are summarized in the Table below:

Table 2-6: Water Demand Summary – Drake Devonshire (L/s)

Water Usage Condition	Existing	Proposed	Difference
Base Demand	0.34	0.30	0.10
Max Day Demand	0.85	1.11	0.26
Peak Hour Demand	1.23	1.59	0.36

The peak hour flow is expected to increase by approximately 0.36 L/s. This increase is considered to be minor and will have no negative impacts on the Wellington Water Distribution System.

2.3.2 Water Demand – Drake Motor Inn

The Drake Motor Inn is proposing the addition of a new 6-unit hotel unit in place of an existing single-family dwelling on 47 Wharf Street. Water demand calculations were completed to support the hotel flows. The calculations are summarized in the Table below:

Table 2-7: Water Demand Summary - Drake Motor Inn (L/s)

Water Usage Condition	Existing	Proposed	Difference
Base Demand	0.01	0.02	0.01
Max Day Demand	0.03	0.04	0.01
Peak Hour Demand	0.05	0.06	0.01

The increase in peak hour flow is minimal 0.01 L/s and will have a negligible effect on the Wellington Water Distribution System.

2.4 Summary - Water

As demonstrated in Section 2.3.1 and 2.3.2, there is expected to be a 0.37 L/s increase in the peak hour demand on Wharf Street. Dillon found the static pressure in the main during peak hour to be 55 psi. The Wellington system is capable of supplying this very minor increase in demand.

The minimum required fire flow remains unchanged with the proposed building addition to the Devonshire Inn. Fire flows are expected to increase for the proposed 6-unit hotel at the Drake

Motor Inn. Upgrades to the Wellington Water Distribution system are proposed to be constructed in the coming years and are expected to significantly improve the fire flow available in the system. Jewell believes with these improvements the system will be able to meet the fire flow requirements of both properties.

There will be no negative impacts on Wellington's Water Distribution System as a result of the proposed site modifications.

3 Sanitary Sewer System

3.1 Existing Conditions

A 200mm asbestos concrete (AC) sanitary sewer services Wharf Street north of Main Street while a 300mm AC sewer services the southern end of Wharf Street to the pump station. The station outlets through a 250mm PVC forcemain flowing north towards Main Street on its way to the treatment plant. The properties at 20 and 24 Wharf Street are expected to be serviced with 125mm sanitary laterals connecting to the sewer on Wharf Street.

3.2 Design Criteria

The sanitary design criteria used is based on 2008 MOE Sewage Design Guidelines and the Ontario Building Code as summarized below:

- Average Daily Residential Domestic Flows: 350 L/d*cap
- Average Daily Use for Hotels: 250 L/d*room
- Average Daily Use for Restaurants: 125 L/d*seat
- Extraneous Flow Allowance: 0.28 L/s*ha
- Population Factors:
    - Hotel Unit: 2 persons per room

3.3 Hydraulic Design

Jewell used MOE 2008 guidelines to calculate the anticipated sewage demand of the proposed site modifications. The sanitary flows for each site have been calculated separately as seen below:

Table 3-1: Sanitary Baseflow Rates – Drake Devonshire

Condition	Hotel (L/day)	Restaurant (L/day)	Single Family (L/day)	Total Baseflow (L/day)	Total Baseflow (L/s)
Pre	3,000	25,000	1,225	29,225	0.34
Post	6,750	31,250	N/A	38,000	0.44

Table 3-2: Sanitary Baseflow Rates - Drake Motor Inn

Condition	Single Family (L/day)	Hotel (L/day)	Total Baseflow (L/day)	Total Baseflow (L/s)
Pre	1,225	N/A	1,225	0.01
Post	N/A	1,500	1,500	0.02

The Harmon Peaking Factor is used to calculate the projected maximum day sanitary flow rate. A sample calculation is shown below.

Equation 3-1: Harmon Peaking Factor

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

Where:

M = Harmon Peaking Factor

P = Population (in thousands)

$$M = 1 + \frac{14}{4 + 0.227^{0.5}} = 4.13$$

Sewage demand is calculated by multiplying the baseflow by the Harmon peaking factor and adding the extraneous flow. The sanitary flows before and after the proposed developments can be seen below:

Table 3-3: Sanitary Max Day Flow Rates – Drake Devonshire

Condition	Area	Population	Baseflow (L/s)	Harmon Peaking Factor	Extraneous Flow (L/s)	Total Flow (L/s)
Pre	0.53	227.5	0.34	4.13	0.15	1.54
Post		304	0.44	4.08		1.94

Table 3-4: Sanitary Max Day Flow Rates - Drake Motor Inn

Condition	Area (ha)	Population	Baseflow (L/s)	Harmon Peaking Factor	Extraneous Flow (L/s)	Peak Flow (L/s)
Pre	0.05	3.5	0.01	4.45	0.01	0.08
Post		12	0.02	4.41		0.09

3.4 Wharf Street Pump Station & Wellington WWTP

Several residential developments are proposed in the near future in Wellington that will require upgrades to the Wharf Street pumping station. In a report completed by R.V. Anderson in 2018, it was noted that the Wharf Street pump station has a rated capacity of 34.1 L/s with one pump operating. A standby pump of equal size is present. With both pumps running, the pumping station could handle a flow of 68.2 L/s in dual-pump operation. R.V. Anderson also noted that during a 25-Yr flood event, a peak flow of 39 L/s would be expected and would require the operation of both pumps. It was noted by R.V. Anderson, "During normal (average) flow conditions, sufficient capacity exists at the Wharf Street pumping station." It is also stated in the R.V. Anderson report that Prince Edward County intends to upgrade this pump station to accommodate increased flows from the proposed residential development.

Jewell reviewed bypass event records outlined in the Prince Edward County Annual reports from 2017 and 2018 and concluded that all naturally occurring bypass events were due to precipitation in April and early May (Appendix E). It is anticipated these bypass events are directly related to spring runoff increasing infiltration rates into the sanitary system. The Drake water usage data concludes that peak usage from their site occurs from June through August (851.5 m^3/month) and does not coincide with spring runoff. As previously noted from the R.V. Anderson report, this would mean during the Drake's peak usage, there would be capacity in the Wharf Street pumping station.

Sanitary flows from both sites are directed south down Wharf Street to the Wharf Street pumping station. To review the impacts on the pumping station, the flows were combined and summarized in the Table below:

Table 3-5: Existing vs. Proposed Sanitary Peak Summer Flows (L/s)

Description	Existing	Proposed	Difference
Devonshire	1.54	1.94	0.40
Motor Inn	0.08	0.09	0.01
Total	1.62	2.03	0.41

Water usage data from the Drake shows a significant decrease in usage during the spring months. An average water usage of 269.8 m^3/month was calculated for the spring months of

March through April using the 2018 and 2019 water usage data. This is approximately 31.7% of the peak usage. A comparison of the expected sanitary flows during the spring thaw are shown in the Table below:

Table 3-6: Existing vs. Proposed Sanitary Peak Spring Flows (L/s)

Description	Existing	Proposed	Difference
Devonshire	0.49	0.62	0.13
Motor Inn	0.02	0.03	0.01
Total	0.51	0.65	0.14

The discounted flow during the period of concern (spring thaw) is approximately equal to 0.14 L/s.

As outlined in the R.V. Anderson report, the Wellington Wastewater Treatment Plant has a rated capacity of 52.7 L/s. R.V. Anderson also concluded that the WWTP has sufficient capacity to handle the Wellington peak flows. The plant is susceptible to bypass events during the spring thaw. The Drake Devonshire has low sanitary usage during the time of spring runoff and the increase in peak demand would not contribute to effects of the WWTP during a spring bypass event. Jewell does not believe there would be any negative impacts as a result of the increased flows.

3.5 Summary - Sanitary

Sanitary flow rates are expected to increase by 0.41 L/s during the summer peak. This increase in flows is considered to be minimal.

The WWTP can treat the additional 0.41 L/s during normal flow conditions. However, during the 25-Yr event, the capacity of the WWTP is exceeded due to inflow and infiltration. Bypass event data concludes that bypasses in the WWTP occur during major spring runoff. The Drake Devonshire sanitary flows during the spring months is approximately 31.7% of its peak summer flow. The increase during the spring is approximately 0.14 L/s and presents minimal affect to the Wharf Street pumping station.

Since this is an infill development, the minor increase in flow is negligible, and the municipality could permit the site modifications. Jewell believes there is sufficient capacity in the pump station and WWTP based on the reported findings in the RV Anderson report. This minimal increase in sanitary flows will have no appreciable impacts to the Wharf Street pump station or the Wellington WWTP.

4 Conclusion

Jewell studied the existing site and reviewed the implications of the proposed modifications on the Town of Wellington's municipal systems and the conclusions are as follows:

Water Distribution System

The increase in peak hour demand as a result of the development is 0.37 L/s. Fire flow conditions will be unchanged on the Devonshire site and will see an increase on 47 Wharf Street. Wellington's water distribution system has a design flow of 37L/s and a recent test performed by PEC staff found 42.2L/s was available. The static pressure during peak hour is 55 psi, and the small increase in demand can be supplied by the system. There will be no negative impacts to the Wellington Water Distribution system as a result of the proposed site modifications.

Sanitary Sewer System

The increase in the max day sanitary flows is 0.41 L/s so the impact on the Wharf Street pumping station is minor. R.V. Anderson states in their report that "there is sufficient capacity in the pumping station under normal flows". The Drake Devonshire water records show during the spring, the Drake only uses 31.7% of the peak summer usage. This significantly discounts the flow directed to the Wharf Street pumping station during the spring thaw.

Bypass data from the WWTP conclude that previous bypass events as a result of precipitation only occur during the spring thaw months. The WWTP is constrained only by inflow and infiltration during the 25-yr event. The flow condition was prorated at the pump station by RVA with no flow data to support this conclusion.

In conclusion, the proposed site modifications will not contribute significantly to peak flows at the Wharf Street pumping station during spring runoff season.

We believe the existing sanitary sewer system in Wellington will be adequate to receive and treat the small increase in sewage flow.

There is sufficient water and sewer capacity to support the proposed development.

Prepared by:	Approved by:
Nick Gliddon, E.I.T.	Bryon Keene, P. Eng
Jewell Engineering Inc.	Jewell Engineering Inc.

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5 References

The information used to prepare this report is based on the following documents and information provided as noted below:

- Dillion Consulting
  - Proposed Country Club Estates Water Supply Review, Wellington Ontario
- Ontario Regulation 332/12
  - Ontario Building Code, 1992
- Ontario Ministry of Environment
  - Design Guidelines for Sewage Works, 2008
  - Design Guidelines for Drinking-Water Systems, 2008
- Prince Edward County
  - Annual Performance Report, 2017
  - Annual Performance Report, 2018
- R.V. Anderson
  - Capacity Analysis for Proposed Development Draft, 2018

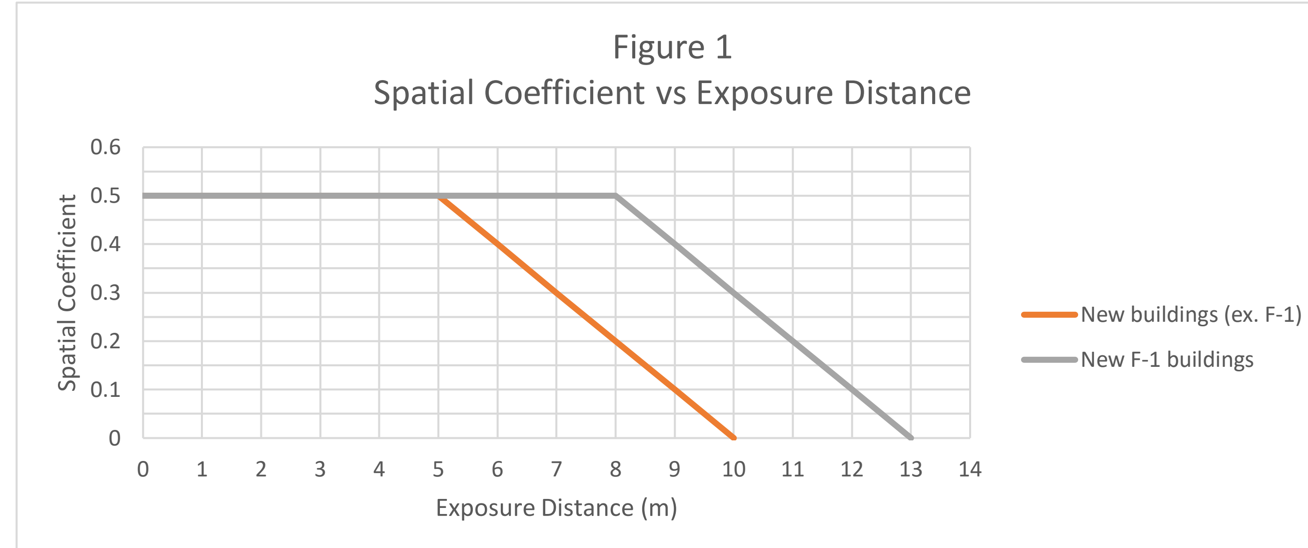
APPENDIX A:
ONTARIO BUILDING CODE FIRE FLOW TABLES AND FIGURES

Fire Flow Requirements as per the Ontario Building Code (OBC)

*A-3.2.5.7. Water Supply for Fire-Fighting

Procedure

			Existing - Hotel + Restaurant	Existing - Single Family	
1	Building to be Assessed		Existing - Hotel + Restaurant	Existing - Single Family	
2	Building Classification		E	C	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	1007.2	100	m2
		2 - Number of Storeys	3	2	
		3 - Building Height	10.48	9	m
		4 - Firewall Separation?	No	No	
		5 - Sprinkler System?	Yes	No	
		6 - Construction Type	Ordinary	Ordinary	
4	Minimum Supply of Water	K	31	18	Table 1
		V	10555	900	m3
		S_{tot}	2.0	2.0	
		Q	654,438	32,400	L
5	**Minimum Fire Flow**	**Table 2**	**9000**	**2700**	L/min
			150	**45**	L/s



lding Classifi	Table 1					
A-1	**Water Supply Coefficient - K**					
A-2	Type of Construction	Classification by Group or Division in Accordance with Table 3.1.2.1 of Building Code				
A-3						
A-4		A-2	A-4	A-1	E	F-1
B-1		B-1	F-3	A-3	F-2	
B-2		B-2				
B-3		B-3				
C		C				
D		D				
E	Fire-Resistive	10	12	14	17	23
F-1	Non-combustible	16	19	22	27	37
F-2	Ordinary	18	22	25	31	41
F-3	Wood Frame	23	28	32	39	53
irewall Sepa	**Exposure - Figure 1**					
Yes	North (Sback)	21	m	0.0		
No	East (Sside)	2.5	m	0.5		
Sprinkler Sy	South (Sfront)	13	m	0.0		
Yes	West (Sside)	10	m	0.5		
No	Total			2.0		

to property line or center of street
or midpoint between 2 buildings on same property

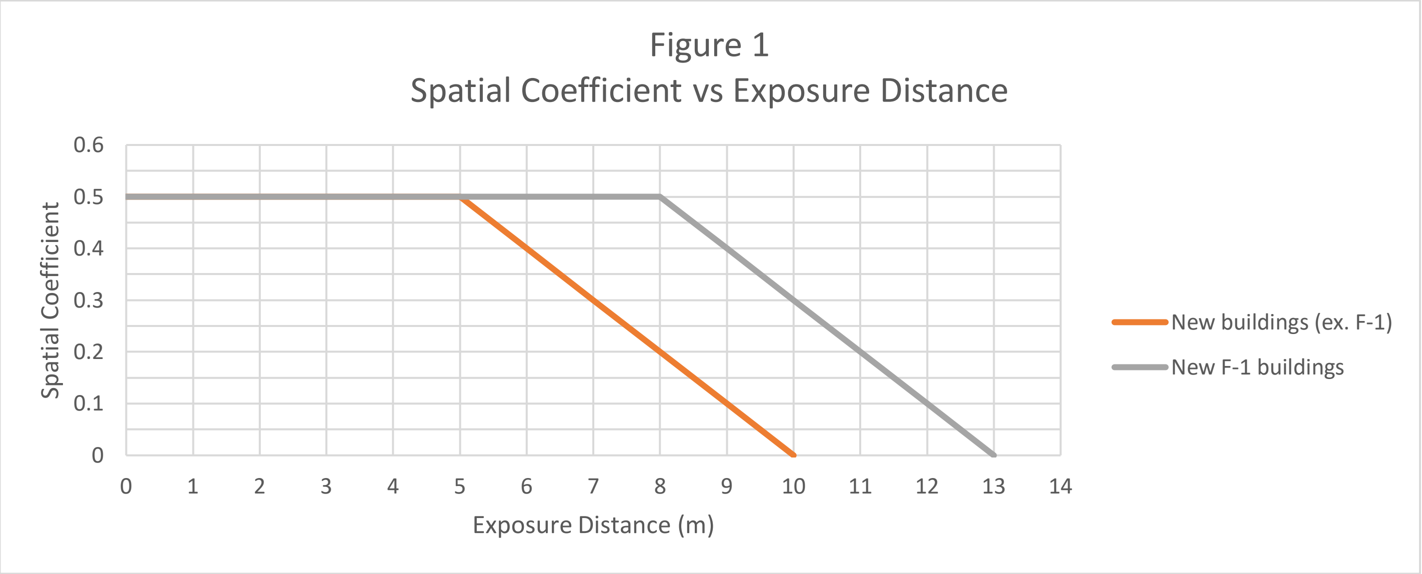
Table 2				
Fire Load			**mum Water Supply Flow**	
			(L/min)	**(L/s)**
	Q <	108000	2700	45
108000	< Q <	135000	3600	60
135000	< Q <	162000	4500	75
162000	< Q <	190000	5400	90
190000	< Q <	270000	6300	105
270000	< Q		9000	150

Fire Flow Requirements as per the Ontario Building Code (OBC)

*A-3.2.5.7. Water Supply for Fire-Fighting

Procedure

1	Building to be Assessed		Existing - Single Family	
2	Building Classification		C	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	100	m2
		2 - Number of Storeys	2	
		3 - Building Height	9	m
		4 - Firewall Separation?	No	
		5 - Sprinkler System?	No	
		6 - Construction Type	Ordinary	
4	Minimum Supply of Water	K	18	Table 1
		V	900	m3
		S_{tot}	2.0	
		Q	32,400	L
5	**Minimum Fire Flow**	**Table 2**	**2700**	L/min
			45	L/s



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A-1	**Water Supply Coefficient - K**					
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A-3						
A-4		A-2	A-4	A-1	E	F-1
B-1		B-1	F-3	A-3	F-2	
B-2		B-2				
B-3		B-3				
C		C				
D		D				
E	Fire-Resistive	10	12	14	17	23
F-1	Non-combustible	16	19	22	27	37
F-2	Ordinary	18	22	25	31	41
F-3	Wood Frame	23	28	32	39	53
irewall Sepa	**Exposure - Figure 1**					
Yes	North (Sback)	15	m	0.0		
No	East (Sside)	3	m	0.5		
Sprinkler Sy	South (Sfront)	13	m	0.0		
Yes	West (Sside)	2.5	m	0.5		
No	Total			2.0		

to property line or center of street

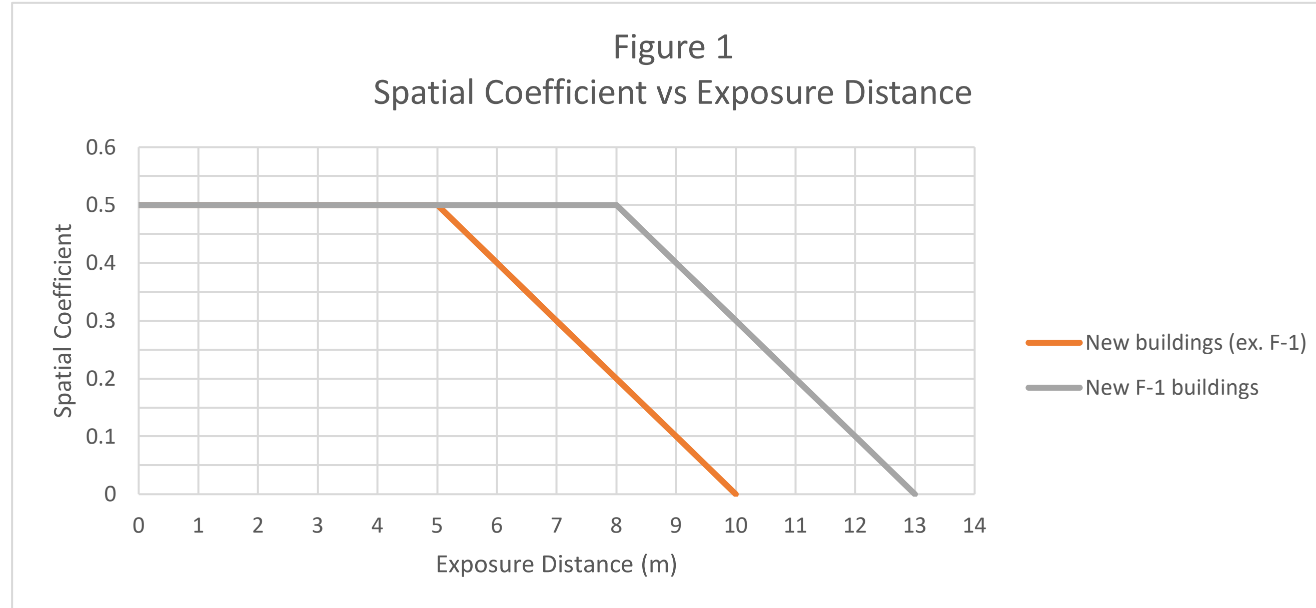
or midpoint between 2 buildings on same property

Table 2				
Fire Load			**mum Water Supply Flow**	
			(L/min)	**(L/s)**
	Q <	108000	2700	45
108000	< Q <	135000	3600	60
135000	< Q <	162000	4500	75
162000	< Q <	190000	5400	90
190000	< Q <	270000	6300	105
270000	< Q		9000	150

Fire Flow Requirements as per the Ontario Building Code (OBC)
*A-3.2.5.7. Water Supply for Fire-Fighting

Procedure

1	Building to be Assessed		Proposed - Hotel + Restaurant	
2	Building Classification		E	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	2163	m2
		2 - Number of Storeys	3	
		3 - Building Height	12	m
		4 - Firewall Separation?	No	
		5 - Sprinkler System?	Yes	
		6 - Construction Type	Ordinary	
4	Minimum Supply of Water	K	31	Table 1
		V	25956	m3
		S_{tot}	2.0	
		Q	1,609,272	L
5	**Minimum Fire Flow**	**Table 2**	**9000**	L/min
			150	L/s



ilding Classifi	Table 1					
A-1	Water Supply Coefficient - K					
A-2	Type of Construction	Classification by Group or Division in Accordance with Table 3.1.2.1 of Building Code				
A-3		A-2 B-1 B-2 B-3 C D	A-4 F-3	A-1 A-3	E F-2	F-1
A-4						
B-1						
B-2						
B-3						
C						
D						
E	Fire-Resistive	10	12	14	17	23
F-1	Non-combustible	16	19	22	27	37
F-2	Ordinary	18	22	25	31	41
F-3	Wood Frame	23	28	32	39	53

irewall Sepa	Exposure - Figure 1			
Yes	North (Sback)	21	m	0.0
No	East (Sside)	3	m	0.5
Sprinkler Sy	South (Sfront)	13	m	0.0
Yes	West (Sside)	10	m	0.5
No	Total			2.0

to property line or center of street
or midpoint between 2 buildings on same property

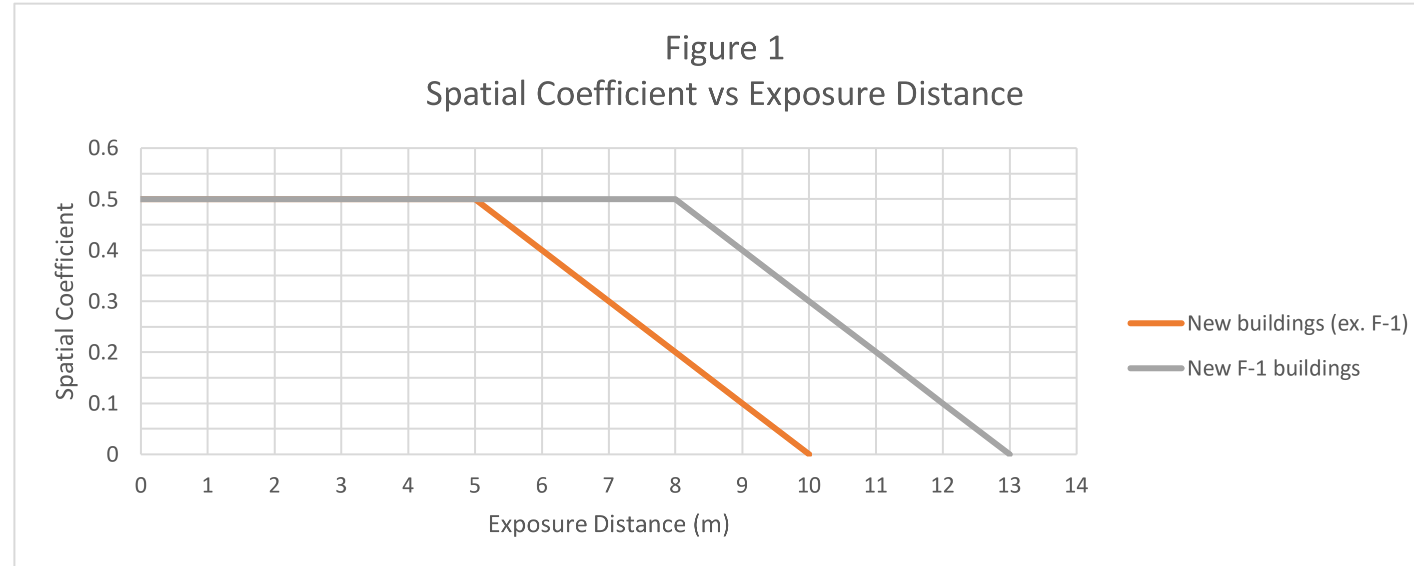
Table 2				
Fire Load			imum Water Supply Flow	
			(L/min)	(L/s)
	Q <	108000	2700	45
108000	< Q <	135000	3600	60
135000	< Q <	162000	4500	75
162000	< Q <	190000	5400	90
190000	< Q <	270000	6300	105
270000	< Q		9000	150

Fire Flow Requirements as per the Ontario Building Code (OBC)

*A-3.2.5.7. Water Supply for Fire-Fighting

Procedure

1	Building to be Assessed		Existing - 47 Wharf St	
2	Building Classification		C	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	100	m2
		2 - Number of Storeys	2	
		3 - Building Height	9	m
		4 - Firewall Separation?	No	
		5 - Sprinkler System?	No	
		6 - Construction Type	Ordinary	
4	Minimum Supply of Water	K	18	Table 1
		V	900	m3
		S_{tot}	2.0	
		Q	32,400	L
5	**Minimum Fire Flow**	**Table 2**	**2700**	L/min
			45	L/s



lding Classifi	Table 1					
A-1	**Water Supply Coefficient - K**					
A-2	Type of Construction	Classification by Group or Division in Accordance with Table 3.1.2.1 of Building Code				
A-3						
A-4		A-2	A-4	A-1	E	F-1
B-1		B-1	F-3	A-3	F-2	
B-2		B-2				
B-3		B-3				
C		C				
D		D				
E	Fire-Resistive	10	12	14	17	23
F-1	Non-combustible	16	19	22	27	37
F-2	Ordinary	18	22	25	31	41
F-3	Wood Frame	23	28	32	39	53

irewall Sepa	**Exposure - Figure 1**			
Yes	North (Sback)	15	m	0.0
No	East (Sside)	3	m	0.5
Sprinkler Sy	South (Sfront)	13	m	0.0
Yes	West (Sside)	2.5	m	0.5
No	Total			2.0

to property line or center of street
or midpoint between 2 buildings on same property

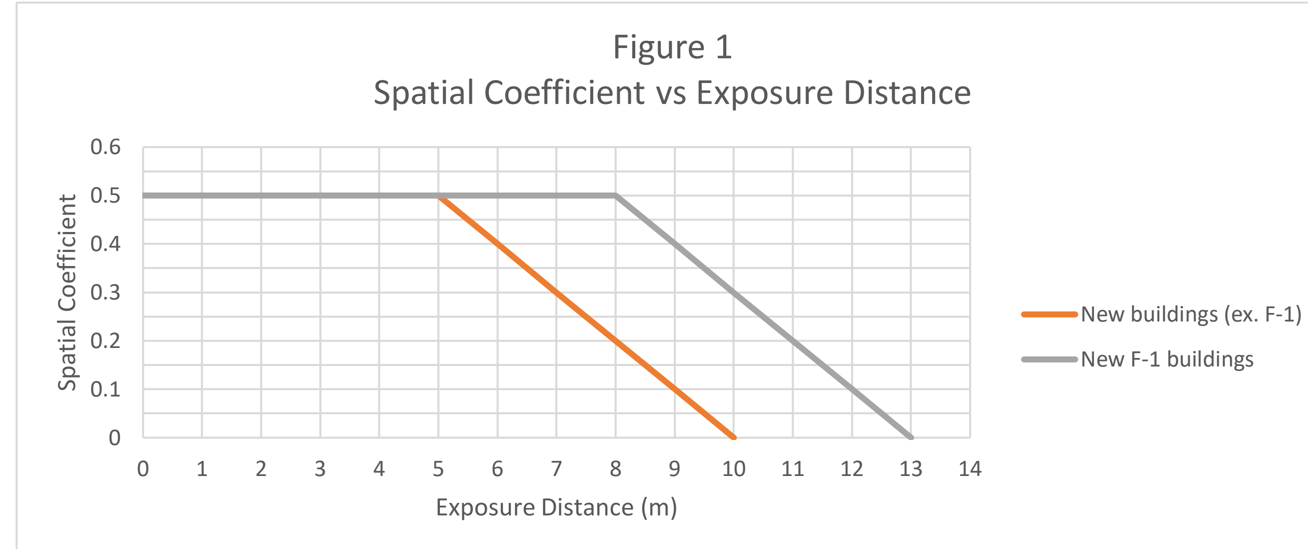
Table 2				
Fire Load			**mum Water Supply Flow**	
			(L/min)	**(L/s)**
	Q <	108000	2700	45
108000	< Q <	135000	3600	60
135000	< Q <	162000	4500	75
162000	< Q <	190000	5400	90
190000	< Q <	270000	6300	105
270000	< Q		9000	150

Fire Flow Requirements as per the Ontario Building Code (OBC)

*A-3.2.5.7. Water Supply for Fire-Fighting

Procedure

1	Building to be Assessed		Existing - Single Family	Proposed - 6-unit Hotel	
2	Building Classification		C	E	A-3.1.2.1.(1)
3	Building Specific Details	1 - Single Floor Area	100	185	m2
		2 - Number of Storeys	2	3	
		3 - Building Height	9	10	m
		4 - Firewall Separation?	No	No	
		5 - Sprinkler System?	No	No	
		6 - Construction Type	Ordinary	Ordinary	
4	Minimum Supply of Water	K	18	31	Table 1
		V	900	1850	m3
		S_{tot}	2.0	2.0	
		Q	32,400	114,700	L
5	**Minimum Fire Flow**	**Table 2**	**2700**	**3600**	L/min
			45	**60**	L/s



lding Classifi	**Table 1**					
A-1	**Water Supply Coefficient - K**					
A-2		Classification by Group or Division in Accordance with Table 3.1.2.1 of Building Code				
A-3						
A-4	Type of Construction	A-2	A-4	A-1	E	F-1
B-1		B-1	F-3	A-3	F-2	
B-2		B-2				
B-3		B-3				
C		C				
D		D				
E	Fire-Resistive	10	12	14	17	23
F-1	Non-combustible	16	19	22	27	37
F-2	Ordinary	18	22	25	31	41
F-3	Wood Frame	23	28	32	39	53
irewall Sepa	**Exposure - Figure 1**					
Yes	North (Sback)	4	m	0.5		
No	East (Sside)	12	m	0.1		
Sprinkler Sy	South (Sfront)	11.5	m	0.2		
Yes	West (Sside)	5	m	0.5		
No	Total			2.0		

to property line or center of street

or midpoint between 2 buildings on same property

Table 2				
Fire Load			**mum Water Supply Flow**	
			(L/min)	**(L/s)**
	Q <	108000	2700	45
108000	< Q <	135000	3600	60
135000	< Q <	162000	4500	75
162000	< Q <	190000	5400	90
190000	< Q <	270000	6300	105
270000	< Q		9000	150

APPENDIX B:
WATER USAGE CALCULATIONS

Drake Devonshire Inn:

Existing Conditions

Base Demand:

Type	Hotel	
Rooms	12	
Usage per Room	250	L/d*room
Base	**3,000**	L/d

Type	Restaurant	
Seating	203	
Usage per Seat	125	L/d*seat
Base	**25,375**	L/d

Type	Single Family	
Population	3.5	
Capita Usage	350	L/d*cap
Base	**1,225**	L/d

Total Baseflow = 28,375 L/d

Max Day and Peak Hour Demand:

Hotel	7,500	L/d
Restaurant	63,438	L/d
Single Family	2,891	L/d
Fire Flow	12,960,000	L/d
Max Day + Fire	**13.03**	ML/d
	150.85	L/s

Hotel	10,770	L/d
Restaurant	91,096.3	L/d
Single Family	4,397.8	L/d
Peak Hour	**106,264.0**	L/d
	1.23	L/s

Proposed Conditions

Base Demand:

Type	Hotel	
Rooms	27	
Capita Usage	250	L/d*room
Base Demand	**6,750**	L/d

Type	Restaurant	
Seating	253	
Capita Usage	125	L/d*seat
Base Demand	**31,625**	L/d

Total Baseflow = 38,375 L/d

Max Day and Peak Hour Demand:

Hotel	16,875	L/d
Restaurant	79,063	L/d
Fire Flow	12,960,000	L/d
Max Day + Fire	**13.1**	ML/d
	151.11	L/s

Hotel	24,232.5	L/d
Restaurant	113,533.8	L/d
Peak Hour	**137,766.3**	L/d
	1.59	L/s

Drake Motor Inn

Existing Conditions

Base Demand:

Type	**Single Family**	
Population	3.5	
Capita Usage	350	L/d*cap
Base	**1,225**	L/d

Max Day and Peak Hour Demand:

Single Family	2,891	L/d
Fire Flow	3,888,000	L/d
Max Day + Fire	**3.89**	ML/d
	45.03	L/s

Single Family	4,397.8	L/d
Peak Hour	**4,397.8**	L/d
	0.05	L/s

Proposed Conditions:

Base Demand:

Type	**Hotel**	
Rooms	6	
Capita Usage	250	L/d*room
Base	**1,500**	L/d

Max Day and Peak Hour Demand:

Hotel	3,750	L/d
Fire Flow	5,184,000	L/d
Max Day + Fire	**5.2**	ML/d

Hotel	5,385.0	L/d
Peak Hour	**5,385.0**	L/d
	0.06	L/s

APPENDIX C:
DILLON CONSULTING WELLINGTON WATER SUPPLY MEMO

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MEMO

DILLON CONSULTING

TO: Steve Harvey, P.Eng.
FROM: Matthew Murdock, P.Eng.
cc: Justin Doiron, P.Eng.
DATE: January 6, 2019
SUBJECT: DRAFT Proposed Country Club Estates Water Supply System Review, Wellington Ontario
OUR FILE:

Dillon Consulting Limited (Dillon) was retained to review the proposed Country Club Estates property development in Wellington Ontario by *Jewell Engineering Inc*. (Jewell). The background design data for the proposed development was provided by Jewell. The water system analysis included steady-state review of probable service pressures under several typical operating conditions. A proposed watermain layout is included in this analysis and forms the basis of the physical assumptions for the study area. The model results include projected service pressure and available fire flow within the margin of error in the hydraulic model and source data.

Background

A hydraulic model was developed as a computational numeric simulation using Bentley WaterGEMS V8i (SELECTSeries 6). This is an industry standard analysis platform commonly employed for the review of water distribution systems. The model consists primarily of a physical representation of the proposed development and a portion of the existing water supply system directly influencing the study area. All background data for the proposed development was provided to Dillon by Jewell with the exclusion of common regulations and guidelines, including the Ontario Ministry of Environment, Conservation and Parks (MOECP) design guidelines for water supply systems.

The hydraulic model was reviewed under steady-state operating conditions including typical demand conditions of average day demand (ADD), maximum day demand (MDD), and peak hour demand (PHD). The purpose of the analysis was to identify service pressures and available fire flow versus acceptable design criteria.

Model Development

A physical representation of the proposed development draft plan for the Country Club Estates in Wellington, Ontario was developed based on the data provided to Dillon. The existing water distribution system skeleton, including 200 mm and 150 mm watermains, directly influencing the study area was schematically represented in the model. The influencing system includes the existing water standpipe which characterizes the boundary condition for the present study.

Existing Water System

The location of the existing water standpipe, according to the third-party peak hour model results, is at the east terminus of Oak St, north of Main St. The existing system directly influencing the study area is summarized in **Figure 1** as provided through third party model raster image to Dillon.

FIGURE 1: EXISTING WATER SYSTEM STATIC PRESSURES UNDER PEAK HOUR CONDITIONS (ADAPTED FROM THRID-PARTY MODEL)

The distribution system is modelled "at grade" according to the detailed survey elevations of the proposed development or the topographical contour elevations provided by Jewell. Elevation data were supplemented from Google Earth for model junction points on Main St. located at the east and west extremes beyond the elevation data.

The model result image of the peak hour day conditions in **Figure 1** indicate that the standpipe water elevation was 116.3 m water level. The TSH design drawings (provided by Jewell) for the existing tower indicate a maximum water level 38.1 m above grade with a minimum water level 8.535 m below maximum, see **Figure 2**. The geodetic base of the existing water tower is not provided on the TSH drawing. Google Earth indicates an approximate grade elevation of 81 m at the tower location. Based on the drawings and the Google Earth data, an approximate working band for the existing water tower is between 110.565 m and 119.1 m water level. The third party water system evaluation indicated a water level of 116.3 m which is between the estimate minimum and maximum level of the existing water storage. The understood working hydraulic grade line of 116.3 m is used for the present analysis.

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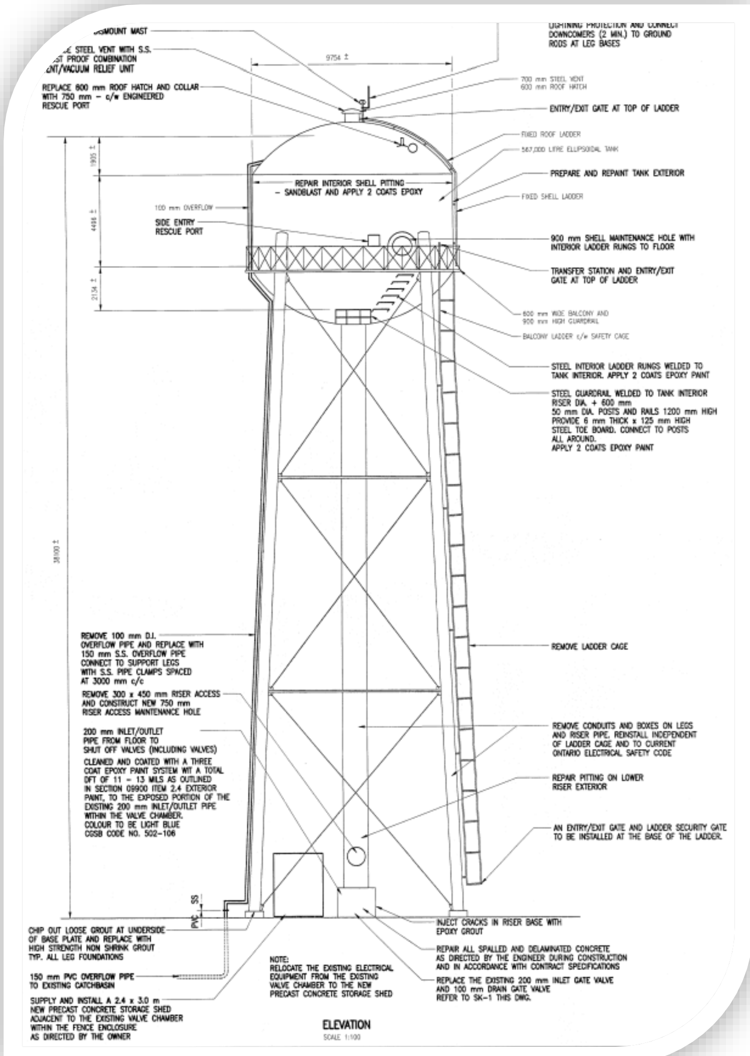

FIGURE 2: EXISTING WELLINGTON WATER STANDPIPE, EXCERPT (TSH 2006)

The understood design criteria for the existing water standpipe are as follows:

- Volume (per TSH): 576 m3
- Base (Grade, approx..): 81 m
- Minimum WL (approx..): 110.565 m
- Working Band WL (approx.): 112.699 m to 117.195 m
- Maximum WL (approx..): 119.100 m
- Given Boundary Condition (PHD): 116.3 m

The model representation of the existing water distribution system directly influencing the study area is summarized in **Figure 3**.

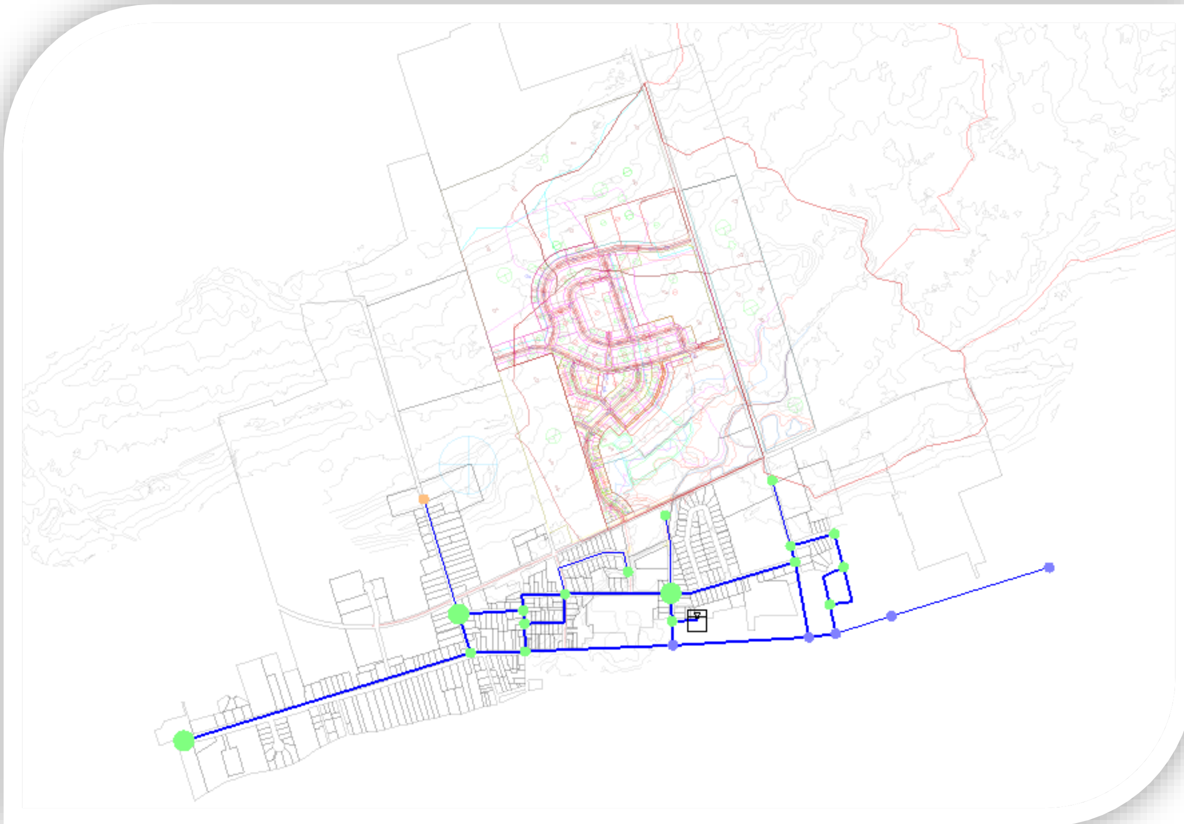

FIGURE 3: WATER MODEL REPRESENTATION OF EXISTING SYSTEM DIRECTLY INFLUENCING PROPOSED DEVELOPMENT

The estimated water demand for Wellington Ontario, including peaking factors for maximum day are adapted from the report "*Operation Reports 2017, Annual and Summary Reports: Wellington Water Treatment Plant & Water Distribution System*" (Prince Edward County, 2017). The treated water flows for the year 2017 are reproduced in part in the following summary table. Peaking factors are calculated for this analysis.

TABLE 1: EXISTING WATER SYSTEM DEMAND FOR WELLINGTON ONTARIO, 2017

Month	Total Flow	ADD	MDD	MDD P.F.	Month P.F.
	[m^3/mo]	[m^3/d]	[m^3/d]	[xADD]	[xADD]
January	12241.09	394.87	533.44	1.35	0.76
February	10683.44	381.55	450.23	1.18	0.73
March	13341.43	430.37	1047.31	2.43	0.82
April	17082.82	569.43	1233.29	2.17	1.09
May	15044.27	485.30	667.01	1.37	0.93

Month	Total Flow	ADD	MDD	MDD P.F.	Month P.F.
	[m^3/mo]	[m^3/d]	[m^3/d]	[xADD]	[xADD]
June	16396.69	546.56	739.32	1.35	1.05
July	19205.77	619.54	732.68	1.18	1.19
August	22922.59	739.44	978.42	1.32	1.42
September	19789.89	659.66	850.51	1.29	1.26
October	17034.94	549.51	833.88	1.52	1.05
November	13324.84	444.16	524.35	1.18	0.85
December	13540.69	436.80	556.97	1.28	0.84
Annual Summary	**190608**	**522.21**	**1233.29**	**2.36**	**1.00**

The available population for Wellington Ontario was adapted from 2011 and 2016 census populations of 1860 people and 1932 people respectively. A growth rate of 0.762 %/a was calculated from these census findings and a projected serviced population of 1947 people was estimated for the 2017 supply year. Based on these data, a blended per-capita water demand of 268.22 L/c/d was estimated (inclusive of residential and non-residential demands). The recorded water demand and calculated peaking factors are summarized in **Figure 4** below.

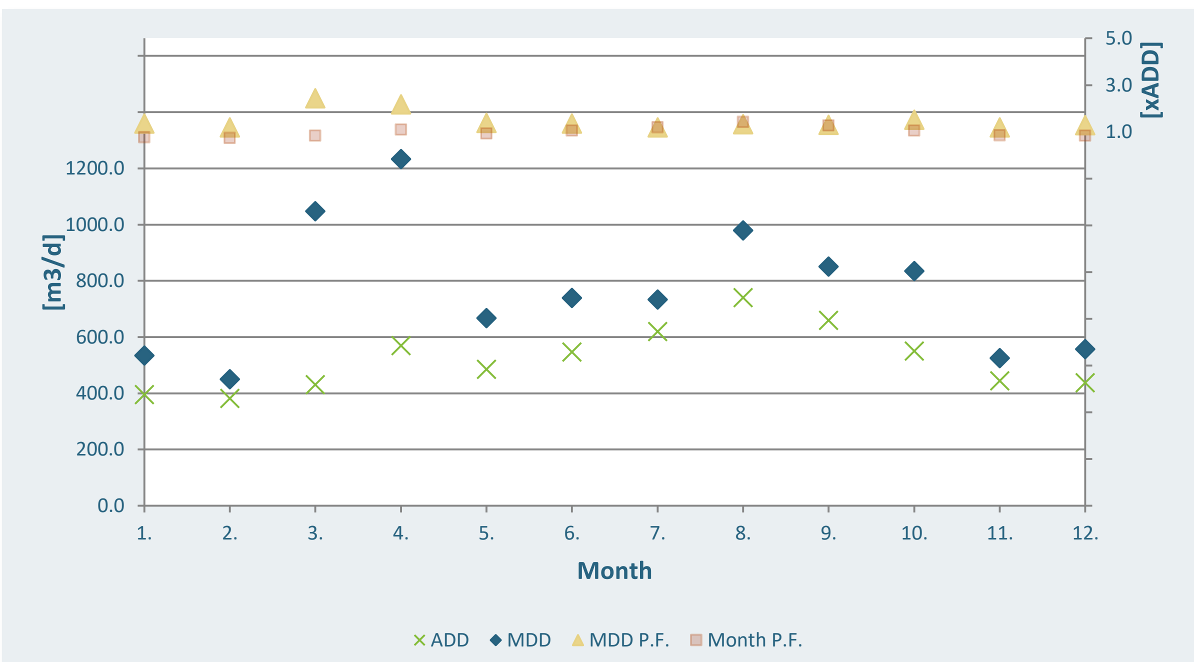

FIGURE 4: EXISTING WATER SYSTEM MONTHLY DEMAND AND PEAKING FACTORS, YEAR 2017

The sanitary design for the proposed development employed a per-capita residential demand of 350 L/c/d with a further commercial demand of 1.15 L/s/ha (average).

The MOECP design guideline for flow rate is documented in "*Design Guidelines for Drinking-Water Systems*" (Ontario Ministry of the Environment, 2008). The design range for residential flow has historically been 270 to 450 L/c/d. It is noted that increased conservation may result in design values toward the lower range. The historic water demand of 268.22 L/c/d for year 2017 supports a lower design range; however, the design flow of 350 L/c/d will be conservatively applied to account for potential uncertainty in the ultimate design density of the proposed development.

The historic maximum day peaking factor was 2.36 for year 2017. The peak hour demand factor, based on the Harmon formula for an estimated year 2017 population of 1947 people, is 3.59.

The analysis will employ the following design basis based on the above analysis:

TABLE 2: HYDRAULIC MODEL WATER DEMAND DESIGN BASIS

Demand Scenario	Residential	Commercial
ADD	350 L/c/d	1.15 L/s/ha
MDD peak factor (xADD)	2.36	2.50
PHD peak factor (xADD)	3.59	3.59

In addition to the above demands, the golf course irrigation demand allowance will be 5 cm per week in two events of 2.5 cm each per information from the USGA. This irrigation demand was evaluated as a second MDD demand scenario. An average demand of 0.019 L/s/ha was used for golf course average demands in accordance with typical annual averages for northern climates according to the USGA.

Proposed Development

The proposed development layout was provided by Jewell in AutoCAD format. The file is georeferenced and drawn in units of meters in the x-y plane. The proposed development parcel includes detailed survey points within the property boundary. These survey points are used to establish hydraulic model elevation at grade elevation for the proposed water distribution system with supplemental elevation data provided by topographical contours at one meter intervals. The proposed water main grid for the development consists of a 200 mm main interconnected to the existing water distribution system, and 150 mm supply mains. The 200 mm alignment follows proposed "Street A" northward to "Street D". The 200 mm main continues west on "Street D" then north on "Street E" to the terminus on Belleville Street. The water mains within the proposed development are modelled as PVC with a Hazen-Williams coefficient of friction (C-factor) of 140 which is considered conservative.

Two interconnections were considered for the proposed development. The primary interconnection extends the existing 200 mm water main on Belleville St. that presently terminates south of Millenium Trail. This interconnection would include a 200 mm length westward on Millenium Trail from Belleville St. to "Street A" and forms a southern boundary of the study area with future connections both east and west for development by others. A secondary interconnection was evaluated and consists of an extension of the existing 200 mm supply main on Lake Breeze Ct. due north on the east side of the existing residential property and intersecting the proposed development at the intersection of Millenium Trail and "Street A".

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The water model representation of the proposed development and including the existing system directly influencing the study area is summarized in **Figure 5**. This figure includes the primary interconnection and the 200 mm main on Millenium Trail as described above.

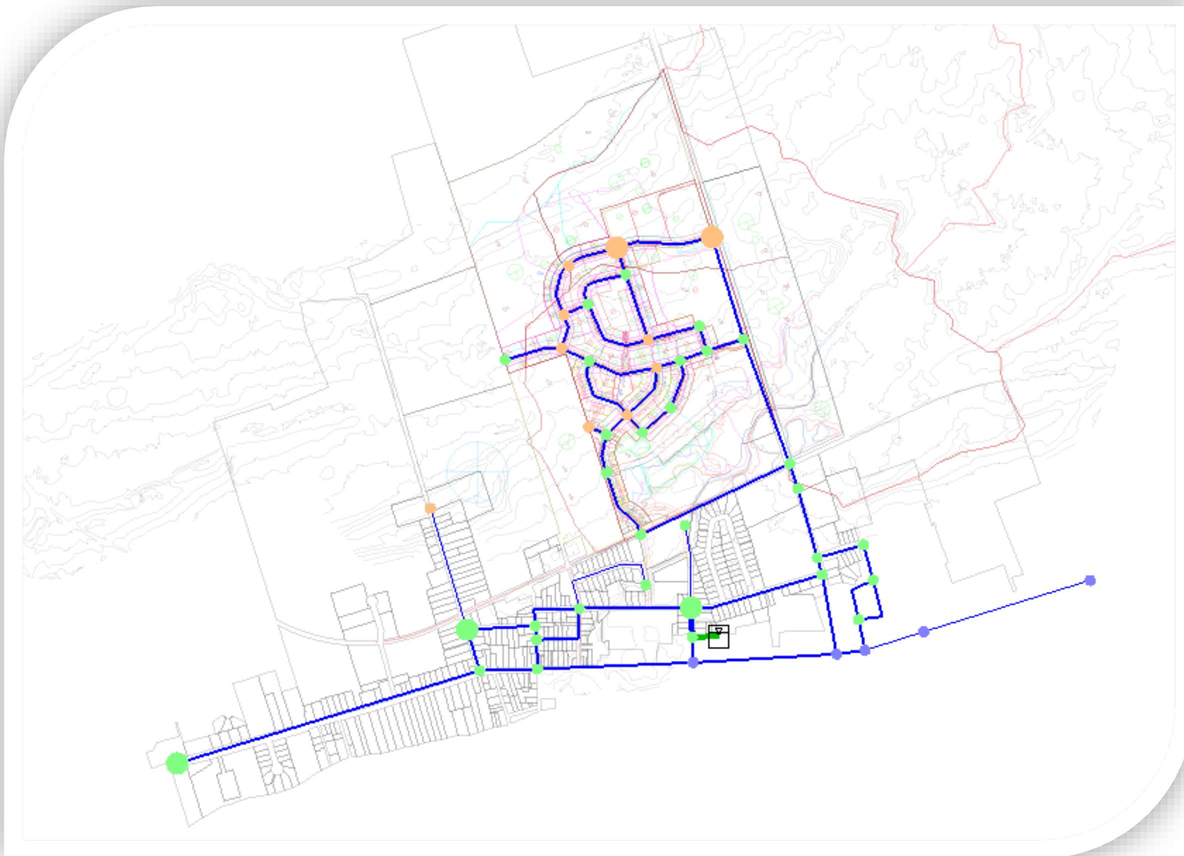

FIGURE 5: WATER MODEL REPRESENTATION OF STUDY AREA

Model Validation

The hydraulic model developed with the design basis described above is compared to the given peak hour demand service pressures as shown in **Figure 1** above on page 2. The model was evaluated under peak hour demand conditions and compared to the given data as summarized in **Table 3** below.

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TABLE 3: WATER MODEL VALIDATION OF EXISTING SYSTEM (PHD PRESSURE COMPARISON)

Location	Provided [1]		Model		Error	
	[psi]	[kPa]	[psi]	[kPa]	[ΔkPa]	[Δ%]
Loyalist Pkwy East of Belleville St.	54.00	372.32	54.97	379.00	6.68	1.8%
Main St West of Bellville St.	54.00	372.32	56.42	389.00	16.68	4.5%
Belleville St. North of 1st Ave	43.00	296.47	42.21	291.00	-5.47	-1.8%
Maple St. North of Niles	46.00	317.16	44.82	309.00	-8.16	-2.6%
Consecon St. North of 2nd Ave	38.00	262.00	34.37	237.00	-25.00	-9.5%
Min	**38**	**262**	**34**	**237**	**-25**	**-9.5%**
Max	**54**	**372**	**56**	**389**	**17**	**4.5%**
Average	**47**	**324**	**47**	**321**	**-3**	**-2%**

1. Provided PHD pressure is based on the annotations from the third-party model as excerpted in Figure 1

The water model of the existing system directly influencing the proposed development provides very good agreement with the given PHD pressure to within 10% margin of error.

Proposed Development Water Service

The proposed development, based on the draft plan provided to Dillon, was reviewed according to the design basis summarized above. The validated model was simulated with a primary interconnection only. The results of this analysis suggested that a secondary interconnection would be beneficial for both service pressure and available fire flow as described in the following section.

The demand applied to the study area includes low density residential (detached and semi-detached dwellings), medium density residential (townhouses), high density residential (apartments), and commercial (golf course). The development density is consistent with the sanitary design parameters provided by Jewell, with the additional allowance of the golf course irrigation discussed earlier:

- Low Density Residential (LDR): 3.0 ppu
- Medium Density Residential (MDR): 2.5 ppu
- High Density Residential (Apartment): 2.5 ppu
- Total number of apartment units: 60 units (block 12 only)
- Golf Course Clubhouse: 25 ppu
- MDR Desnity: 29 units/ha
- LDR Density: 19 units/ha

The proposed draft plan average day demand corresponds to a population equivalent of 1527 capita equivalent. The total flow rate for the steady state demand scenarios are summarized as follows:

- Average Day Demand (residential only): 7.42 L/s
- Average Day Demand: 9.97 L/s
  (Including 2.22ha commercial allowance, and 85.13 ha golf-course allowance at 0.019 L/s/ha)

- Maximum Day Demand: 20.98 L/s
  (Excluding golf course irrigation)
- Maximum Day Demand: 83.73 L/s
  (Including golf course irrigation at 33% area per day, 5 cm water per week)
- Peak Hour Demand: 59.23 L/s
  (Excluding golf course irrigation)

Steady State Service Pressure

The projected service pressure based on the above steady-state scenarios and with primary- and primary plus secondary interconnects are summarized in **Table 4** below. The projected service pressures are compared to the MOECP guideline criteria for water supply pressure. Model results which are inferior to the guideline criteria are highlighted.

TABLE 4: WATER MODEL SERVICE PRESSURE SCENARIOS

Scenario		Min	Max	Average	Std. Dev	MOECP Guideline Criteria	
		[kPa]	[kPa]	[kPa]	[kPa]	Min [kPa]	Max [kPa]
Primary Interconnect Only	ADD	**224.00**	334.00	280.00	31.00	275.00	700.00
	MDD	**198.00**	**308.00**	**254.00**	32.00	350.00	480.00
	MDD + Golf Course Allowance	-	-	-	-	350.00	480.00
	PHD	**21.00**	**155.00**	**87.00**	39.00	275.00	480.00
Primary and Secondary Interconnect	ADD	**227.00**	337.00	283.00	31.00	275.00	700.00
	MDD	**209.00**	**320.00**	**265.00**	32.00	350.00	480.00
	MDD + Golf Course Allowance	**-**	**-**	**-**	**-**	350.00	480.00
	PHD	**105.00**	**221.00**	**170.00**	37.00	275.00	480.00

The proposed water system is not expected to meet MOECP guideline criteria for service pressure. The grade elevation of the proposed development is generally higher than the existing system, resulting in lower average pressure. In particular the MDD and PHD demand conditions are expected to result in pressures below guideline criteria even at maximum service pressure.

The inclusion of a secondary interconnection point improves the service pressure under maximum and peak demand conditions but will not be expected to meet guideline criteria. The model results indicate an improvement of up to 195% to average service pressure under the higher dynamic loading of the peak demand conditions, but the average still falls below the guideline criteria. Based on this analysis, two points of connection are recommended for the proposed development as well as an increase in supply pressure to the study area through additional booster pumping, increased water storage elevation, or both.

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The proposed development is expected to require additional engineering to increase service pressures to meet MOECP guideline criteria. It is suspected that the existing distribution system—including the areas of Washburn Drive and of Consecon St.—may also experience similar pressure constraints due to surface elevation above the supply system. The grade generally increases in elevation north of the lake shoreline up to an elevation approximately 93 meters in the proposed development area. Consequently an increased pressure head may be beneficial for the entire distribution system in Wellington Ontario. The operational details of the existing system outside of the proposed development area are not presently known in sufficient detail to provide an engineered solution for the entire system.

Available Fire Flow

The available fire flow was evaluated with the existing standpipe at a water elevation of 116.3 meters. This may not reflect the minimum available supply head at the end of a fire event, but is sufficient for preliminary assessment. Fire flow reinforcement from the water treatment plant high lift pumps was conservatively excluded from this analysis. The firm pumping capacity and clearwell volume are not known; consequently, it is not possible to evaluate the firm pumping duration for the purpose of fire flow. Additional fire flow capacity could be included if the WTP capacity can be demonstrated for the required duration of fire events.

The existing water treatment facility was originally designed to achieve 37.5 L/s residential fire flow. This is documented in the report to the Committee of the Whole dated June 28, 2018 with the title "*Wellington Water Pressure Issues*", (Prince Edward County, June 28, 2018. The understood residential fire flow design criteria is 37.5 L/s as documented on the report page 6 of 12 from the "Preliminary Design Report" (Totten Sims Hubicki Associates, 1994) as excerpted in the report. The report to the Committee of the Whole includes references to MOEE guidelines for sizing of water storage facilities. These references should not be misunderstood as equivalent to design fire flow available at hydrants. The application of storage volume sizing criteria to hydrant design flow capacity is a misrepresentation.

The proposed system was evaluated with a boundary hydraulic grade elevation of 116.3 m elevation and without irrigation at the golf course parcels. The available fire flow at each junction in the study area was evaluated at a residual pressure of 140 kPa at the junction according to MOECP pressure criteria. The simulation results are summarized in **Table 5** below.

TABLE 5: AVAILABLE FIRE FLOW MODEL RESULTS AT 116.3M HGL AND 140KPA RESIDUAL

Parameter	Primary Interconnection Only	Primary and Secondary Interconnections
Minimum Available Fire Flow [L/s]	17.69	31.8
Maximum Available Fire Flow [L/s]	46.71	77.61
Mean Available Fire Flow [L/s]	20.05	37.83
Number of Nodes Evaluated	24	24
Fraction of Nodes Achieving 37.5 L/s	4%	25%

The available fire flow is not expected to achieve the minimum residential fire flow threshold. The inclusion of a second interconnection significantly improves the projected fire flow capabilities with but does not achieve the uniform residential fire flow requirements.

Commercial and high-density residential structures were not evaluated beyond the understood residential fire flow criteria. These structures may be subject to different criteria due to construction, sprinklers, or other construction and insurance requirements. These architectural design details are not presently known; consequently, the fire protection criteria specific to these structures were not established separate from the residential flow rate of 37.5 L/s.

Summary

The proposed development is expected to require additional engineering to provide adequate pressure and available fire flow. Several options are available, each would require additional analysis:

1. Dedicated booster pumping servicing the study area minimum hour and peak demand;
2. Dedicated elevated storage with a working hydraulic elevation of between 123 meters to 136 meters; or,
3. Municipal elevated storage with a working hydraulic elevation of between 123 meters to 136 meters complete with high lift pumping upgrades as needed.

The advantage of a Municipal system upgrade may include potential cost sharing. It is not known if the existing water standpipe meets the volume criteria for supply to the expanded population, nor is it known if the water treatment plant capacity has sufficient reserve for the proposed demands. An evaluation of the existing pressure zone management and high lift pumping capacity is beyond the present scope of this report and is recommended.

The fire flow criteria are based on the original water treatment plant residential fire flow design of 37.5 L/s according to the preliminary design brief. Additional fire flow criteria for buildings other than low density residential may be required for insurance purposes. This analysis should be revisited if fire flow criteria are refined.

APPENDIX D:
R.V. ANDERSON SANITARY CAPACITY ANALYSIS

Wellington Sanitary Sewer

Capacity Analysis for Proposed Development

Draft

Prepared for:
County of Prince Edward

RVA 183934.20

May 15, 2018

1.0 BACKGROUND

The following technical memorandum is a summary of the hydraulic capacity analysis of the Wellington sanitary sewer system completed for the County of Prince Edward. Hydraulic modelling was performed to review the capacity of the existing system and its ability to accommodate four proposed developments (Berkhout, Schickedanz, Kaitlin and Hirschfield).

1.1 Existing Conditions

The Wellington sanitary sewer system services a population of approximately 1932 (2016 census). The system has pumping stations at the bottom of Wellington Street and Wharf Street that lift flows into a gravity trunk sewer on Main Street that discharges to the main pumping station at the Wellington wastewater treatment plant (WWTP). Figure 1.1 presents an overview of the sanitary sewer system.

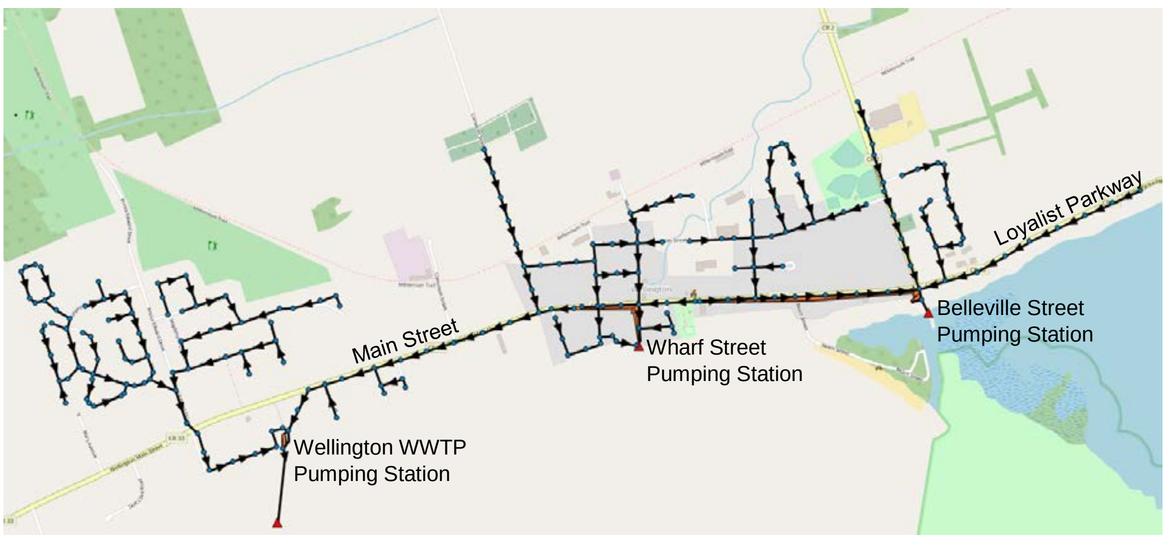

Figure 1.1 – Wellington Sanitary Sewer Plan

1.1.1 Flows Measured at Wellington WWTP (2017)

The Wellington WWTP has a rated capacity of 1,500 m^3/day (17.4 L/s) and an approved peak daily flow rate of 4,550 m^3/day (52.7 L/s). The flows at the WWTP as reported in the 2017 Annual Performance Report were as follow in Table 1.1.

Table 1.1 – Wellington WWTP Influent Quantity Flow Data, 2017

Month	Monthly Average		Monthly Peak	
	m^3/day	L/s	m^3/day	L/s
January	1,157	13.4	1,882	21.8
February	855	9.9	1,240	14.4
March	830	9.6	1,112	12.9
April	1,278	14.8	3,790	43.9
May	1,718	19.9	5,202	60.2
June	1,203	13.9	3,453	40.0
July	911	10.5	1,264	14.6
August	632	7.3	778	9.0
September	620	7.2	1,557	18.0
October	774	9.0	2,343	27.1
November	1,095	12.7	2,648	30.6
December	683	7.9	844	9.8
2017	980	11.3	5,202	60.2

* Wellington Wastewater Treatment Plant, Operations Reports 2017, Prince Edward County.

The following observations were made based on the 2017 flow data:

- The average flow to the WWTP plant was 11.3 L/s and equated to approximately 500 L/capita/day (including inflow and infiltration).
- The average flow during the driest month (September) was 7.2 L/s and equated to approximately 320 L/capita/day (including inflow and infiltration). This is less than the typical residential design flow of 450 L/capita/day.
- The highest daily flow in May was 60.2 L/s and equated to approximately 2,700 L/capita/day (including inflow and infiltration). This is an indication of significant inflow and infiltration during wet conditions (106.4 mm over 3 days at Belleville climate station).

1.1.2 Collection System Pumping Capacities

Table 1.2 presents the current capacities of the 3 pumping stations in the Wellington sanitary sewer system. These capacities are as reported in the Wellington Water Pollution Control Plant's Certificate of Approval (2006) and have not been field confirmed.

Table 1.2 – Sanitary Sewer System Pumping Capacities

Pumping Station	# Pumps	Rated Capacity
Belleville Street PS	2	30.3 L/s @ 13.7 m TDH
Wharf Street PS	2	34.1 L/s @ 11.6 m TDH
WWTP	3	29.5 L/s @ 6.7 m TDH

1.2 Proposed Developments

Four primarily residential developments have been considered in the analysis of the sanitary sewer system's capacity to accommodate increased wastewater flows. The proposed developments are as follow, with their location shown on Figure 1.2 (following page):

Table 1.3 – Proposed Development Population Summary

Proposed Development		# Units	Est. Population
Berkhout	Phase 1	100	200
	Phase 2	287	574
Schickedanz	---	95	190
Kaitlin	Phase 1	300	600
	Phase 2	160	320
Hirschfield	Phase 1	164	328
	Phase 2	355	710
Proposed Development		**1,461**	**2,922**
Existing Development		---	**1,932**
Total at Full Buildout		---	**4,854**

The number of units proposed for development and the associated population increase (approximately 150%) indicate that wastewater flows in the Wellington sanitary sewer system would increase significantly.

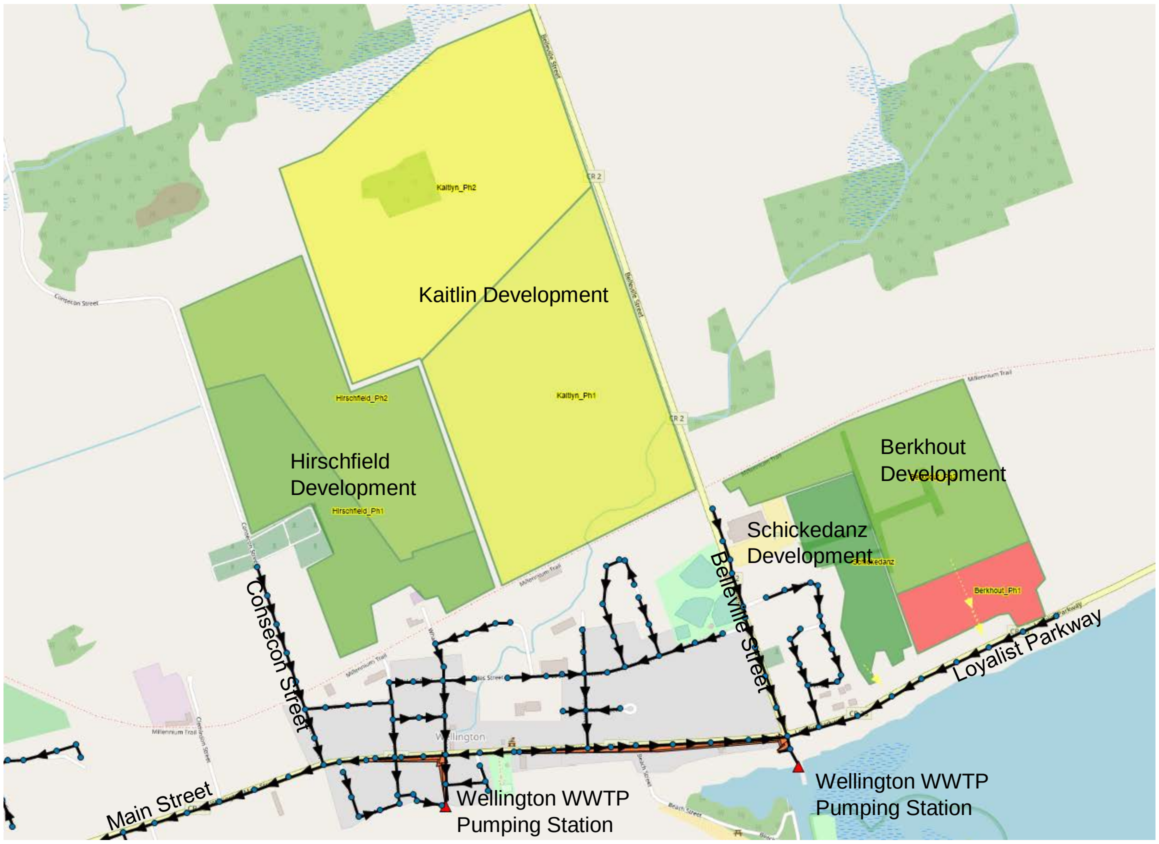

Figure 1.2 – Locations of the Proposed Developments

Table 1.4 –Wastewater Flow Summary for the Proposed Developments

Proposed Developments		Residential Daily Avg (L/s)	Residential Peak (L/s)	Infiltration (L/s)	Design Flow (L/s)
Berkhout	Phase 1	1.0	4.2	1.4	5.6
	Phase 2	3.0	11.9	2.4	14.3
Schickedanz	---	1.0	4.0	1.1	5.1
Kaitlin	Phase 1	3.1	12.5	4.0	16.5
	Phase 2	1.7	6.7	4.0	10.7
Hirschfield	Phase 1	3.1	12.3	1.6	13.9
	Phase 2	3.2	13.0	2.6	15.6
Total Proposed Development		**16.1**	**64.6**	**17.1**	**81.7**

2.0 CAPACITY ANALYSIS

The following paragraphs present the results of the capacity analyses performed for the sanitary sewer system for current development and the development scenarios requested by the County.

2.1 Current Development

Capacity analysis was performed for the current level of development. The County suggested that wet weather flow conditions match those of a 25-year return period storm, and as such would be less than the maximum daily flow (60.2 L/s) measured during May 2017. A review of recent wastewater treatment plant operational reports suggested that this peak flow would be approximately 50 L/s. This is 4.4 times the average flow during 2017, 6.9 times the dry month average flow (September) and 2.5 times the wet month average flow (May).

The wastewater flow for current development was distributed throughout the sanitary sewer model based on population, length of sewer, number of manholes and age of development. The age of development was used to account for the likelihood that inflow and infiltration would be higher in the older sewer sections rather than recent development such Wellington on the Lake.

The following tables summarize the results of the current development capacity analysis**.**

Table 2.1 – Pumping Capacity Summary – Current Development

Location	**Capacity** (L/s)	**Peak Flow** (L/s)
Belleville Street	2 pumps rated @ 30.3 L/s ea.	18
Wharf Street	2 pumps rated @ 34.1 L/s ea.	39
WWTP	3 pumps rated @ 29.5 L/s ea.	49

During wet weather conditions the second (standby) pump is required at both the Wharf Street pumping station and the WWTP. The flow is elevated at the Wharf Street pumping station due to the pump cycles from the Belleville Street station which discharges approximately 300 m upstream with insufficient attenuation in this length of sewer to reduce the peak flow. During dry weather conditions and lesser rainfall events there is sufficient capacity within the Wharf Street wet well to attenuate the peak flow such that only one pump (duty pump) is required.

There is attenuation in the Main Street sewer downstream of the Wharf Street pumping station (approximately 1,000 m length) as the peak flow is reduced from 70 L/s near Consecon Street to 49 L/s at the WWTP.

A review of the pump operations during normal (average) flow conditions indicates that sufficient capacity exists at all 3 pumping stations.

Table 2.2 – Trunk Sewer Capacity Summary – Current Development

Sewer Section	Diameter (mm)	Peak Flow (L/s)	Percentage of Capacity
Loyalist Parkway (east of Belleville)	200	3	10
	250	6	15
Main Street d/s of Belleville FM	300	33	30-35
Main Street d/s of Wharf St FM	355	70	55-65
	400	49	30-45

Table 2.2 presents the peak flows modelled in the trunk sewer along Loyalist Parkway / Main Street from the east end of Wellington to the WWTP. The peak flows indicate that the existing gravity sewers have sufficient capacity to accommodate all peak wet weather flows.

It should be noted that no capacity issues were identified for the gravity sewers that discharge to the Loyalist Parkway and Main Street trunk sewers. As such, they are not included in the capacity summary tables.

2.2 Berkhout Development

The Berkhout development is expected to proceed in two phases. Phase 1 will include 100 units to be completed by 2021. Phase 2 (287 units) will proceed at a rate of approximately 30 lots per year to full buildout of 387 lots.

The capacity of the sanitary sewer system to accommodate Phases 1 and 2 of the Berkhout development are summarized in the following sections.

2.2.1 Berkhout Development Phase 1

Phase 1 of the Berkhout development is expected to increase the peak flow in the gravity sewer east of Belleville Street by approximately 5.6 L/s. This increases flow to the Belleville Street pumping station, however, the impacts are not evident in the downstream peak flow rates (i.e. single pump operation maintained at Belleville Station pumping station). The main impacts are an increased number of pump cycles at the pumping stations, higher average flows in the gravity sewers and an increased daily volume of wastewater to the WWTP.

Tables 2.3 and 2.4 present capacity summaries for the pumping stations and main trunk sewer.

Table 2.3 – Pumping Capacity Summary – Berkhout Phase 1

Location	Capacity (L/s)	Peak Flow (L/s)
Belleville Street	2 pumps rated @ 30.3 L/s ea.	23
Wharf Street	2 pumps rated @ 34.1 L/s ea.	39
WWTP	3 pumps rated @ 29.5 L/s ea.	49

Table 2.4 – Trunk Sewer Capacity Summary – Berkhout Phase 1

Sewer Section	Diameter (mm)	Peak Flow (L/s)	Percentage of Capacity
Loyalist Pkwy (east of Belleville)	200	3	10
	250	6	15
Main Street d/s of Belleville FM	300	33	30-35
Main Street downstream of Wharf St FM	355	70	55-65
	400	49	30-45

No upgrades to the gravity sewer are required to accommodate the first phase of the Berkhout development.

2.2.2 Berkhout Development Full Buildout

Full buildout of the proposed Berkhout development is expected to increase the peak flow in the gravity sewer east of Belleville Street by approximately 19.9 L/s. This will impact the flow in the gravity sewer as well as pumping required at the Belleville Street station. Tables 2.5 and 2.6 present results of the capacity analysis.

Table 2.5 – Pumping Capacity Summary – Berkhout Full Buildout

Location	Capacity (L/s)	Peak Flow (L/s)
Belleville Street	2 pumps rated @ 30.3 L/s ea.	38
Wharf Street	2 pumps rated @ 34.1 L/s ea.	69
WWTP	3 pumps rated @ 29.5 L/s ea.	89

The increased peak flow from the Berkhout development results in dual pump operation at the Belleville Street pumping station. This has downstream impacts, as the peak flow to the Wharf Street station increases accordingly. The peak flow to the Wharf Street station (69 L/s) would exceed its dual pumping capacity (68 L/s). This could be significant if there is a loss of pumping efficiency associated with the two pumps sharing a header / forcemain. As such, an upgrade to the Belleville pumping station would be required to accommodate peak flows from the full Berkhout development.

There is a significant increase in the peak flow to the WWTP as well. The expected peak flow is equivalent to the combined capacity of all 3 pumps operating at the plant. As noted above, this could be significant if there is a loss of pumping efficiency associated with pumps sharing a header / forcemain. As such, an upgrade to the WWTP pumping station would be required to accommodate peak flows from the full Berkhout development.

Table 2.6 – Trunk Sewer Capacity Summary – Berkhout Full Buildout

Sewer Section	Diameter (mm)	Peak Flow (L/s)	Percentage of Capacity
Loyalist Pkwy (east of Belleville)	200	23	100-110
	250	26	70
Main Street d/s of Belleville FM	300	63	60-65
Main Street downstream of Wharf St FM	355	74	55-65
	400	74	40-65

The data in Table 2.6 indicates that 500 m of 200 mm diameter trunk sewer on Loyalist Parkway east of Belleville Street would operate at or above capacity during peak flow conditions. This is the only section of gravity sewer that would need to be upgraded (to 250 mm diameter) to accommodate full buildout of the Berkhout development.

2.3 Full Buildout of Proposed Developments

Full buildout of Berkhout, Schickedanz, Kaitlin and Hirschfield developments is expected to increase the population in Wellington by approximately 150% (1,461 units, 2922 people). Capacity analyses were performed to identify which sections of the sanitary sewer system would require upgrades to accommodate the peak flow (81.7 L/s) expected from these developments.

Conceptual servicing plans for the Hirschfield and Kaitlin developments included a new trunk sewer along the Millennium Trail. This sewer would convey wastewater from each development to Cleminson Street then Main Street. The proposed gravity sewer shown on the Kaitlin development plan was proposed to be 250 mm diameter but based on our analysis we'd recommend that it be 300 mm from the Kaitlin development to the Hirschfield development connection and 375 mm diameter downstream to Main Street.

The capacity of the sanitary sewer system to accommodate full buildout of the proposed developments is summarized in the following tables.

Table 2.7 – Pumping Capacity Summary – Full Development Buildout

Location	Capacity (L/s)	Peak Flow (L/s)
Belleville Street	2 pumps rated @ 30.3 L/s ea.	43
Wharf Street	2 pumps rated @ 34.1 L/s ea.	69
WWTP	3 pumps rated @ 29.5 L/s ea.	147

Peak flow (43 L/s) to the Belleville pumping station will require dual pump operation, however the average daily flow (10 L/s) will remain much less than the capacity of a single pump. No upgrades are required based on capacity, however the County should review the wet well sizing and expected cycling of pumps on/off to ensure the mode efficient operation of the station.

As with full buildout of the Berkhout development, dual pump operation will be required for peak flow to the Wharf Street station. The peak flow (69 L/s) would exceed the existing dual pumping capacity (68 L/s). This could be significant if there is a loss of pumping efficiency associated with the two pumps sharing a header / forcemain. As such, an upgrade to the Wharf Street pumping station would be required to accommodate peak flows from full buildout. It should be noted that the conceptual plans do not route any wastewater flows from the Kaitlin and Hirschfield developments to the Belleville Street and Wharf Street pumping stations. We agree with this concept of maximizing gravity flow as a preferred routing option.

The most significant peak flow increase is expected at the WWTP. The expected peak flow (147 L/s) greatly exceeds the combined capacity of all 3 pumps operating at the plant. An upgrade to the WWTP pumping station would be required to accommodate peak flows from full buildout of the 4 proposed developments.

Table 2.8 – Trunk Sewer Capacity Summary – Full Development Buildout

Sewer Section	Diameter (mm)	Peak Flow (L/s)	Percentage of Capacity
Loyalist Parkway (east of Belleville)	200	28	100-130
	250	31	80
Main Street d/s of Belleville FM	300	63	60-65
Main Street downstream of Wharf St FM	355	74	55-65
	400	119	70-110

The Schickedanz development is proposed to be adjacent to the Berkhout development and will also discharge to the trunk sewer on Loyalist Parkway east of Belleville Street.

An upgrade of 500 m of 200 mm diameter trunk sewer to 250 mm diameter would be required to accommodate the Berkhout and Schickedanz developments.

The only other section of gravity sewer that would not have sufficient conveyance capacity is the 400 mm diameter sewer on Main Street between Cleminson Street and the WWTP. Upgrading the pipe to 450 mm diameter would have it operate at 80-90% of capacity. It may be preferable to upgrade to 525 mm diameter to accommodate additional growth or added Wharf Street pumping capacity.

3.0 CONCLUSIONS

The existing sanitary sewer system has sufficient capacity to convey current peak design flows. Flows presented in the 2017 Operations Report for the WWTP indicate that the sewer system is susceptible to wet weather inflow and infiltration and that flows (5,202 m3/day) exceed the rated capacity of the plant (4,500 m3/day) at least one day in May.

Four proposed developments (Berkhout, Schickedanz, Kaitlin and Hirschfield) could increase the population in Wellington by 150% from 1,932 (2016 census) to approximately 4,854. This is expected to increase the daily average wastewater flow to the plant from 11.3 L/s (2017 Operations Report) to 43.5 L/s and the peak instantaneous flow from approximately 50 L/s to 147 L/s. These flow increases require the following conveyance capacity upgrades to the sanitary sewer system.

- The 500 m long section of 200 mm diameter sewer downstream of the proposed Berkhout and Schickedanz developments will need to be upgraded to 250 mm diameter to operate between 60% and 80% of capacity.

Figure 3.1 – Loyalist Parkway sewer upgrade to 250 mm (shown as red).

- The capacity of the Wharf Street pumping station (approximately 68 L/s) would need to be increased to accommodate a peak flow of 69 L/s. This would not require a significant increase in pumping capacity and could likely be accomplished by upgrading the pumps without replacing the existing forcemain. It should be noted, however, that the pumping station, header and forcemain should all be inspected to confirm the state of repair.
- A new 300mm / 375 mm diameter gravity sanitary sewer should be installed to collect wastewater from the Kaitlin and Hirschfield developments. As proposed in the conceptual plans for these developments, the sewer would be constructed along the Millennium Trail and turn down Cleminson Street to connect to the sanitary sewer on Main Street.

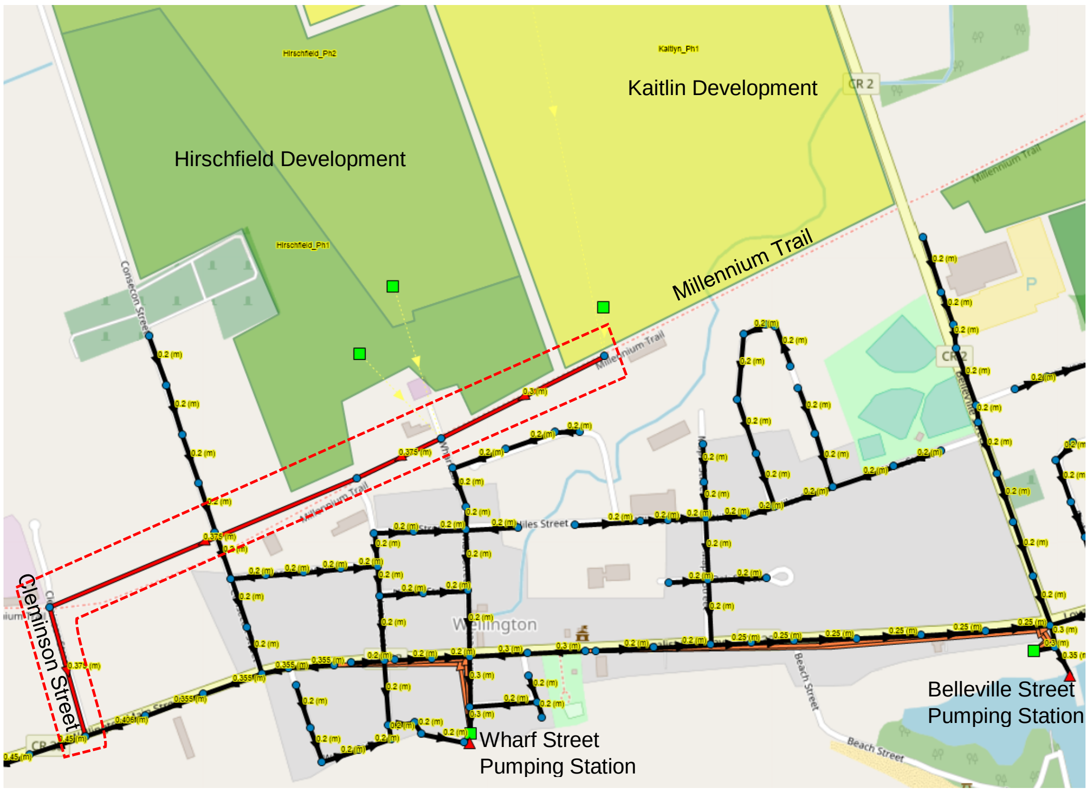

Figure 3.2 – Proposed sewer along Millennium Trail (shown as red).

- The approximately 635 m long section of 400 mm diameter sewer downstream of Cleminson Street on Main Street will need to be upgraded to a minimum 450 mm diameter to operate between 80% and 90% of capacity. It may be preferable to upgrade to 525 mm diameter to accommodate additional growth or added capacity at the Wharf Street pumping station.

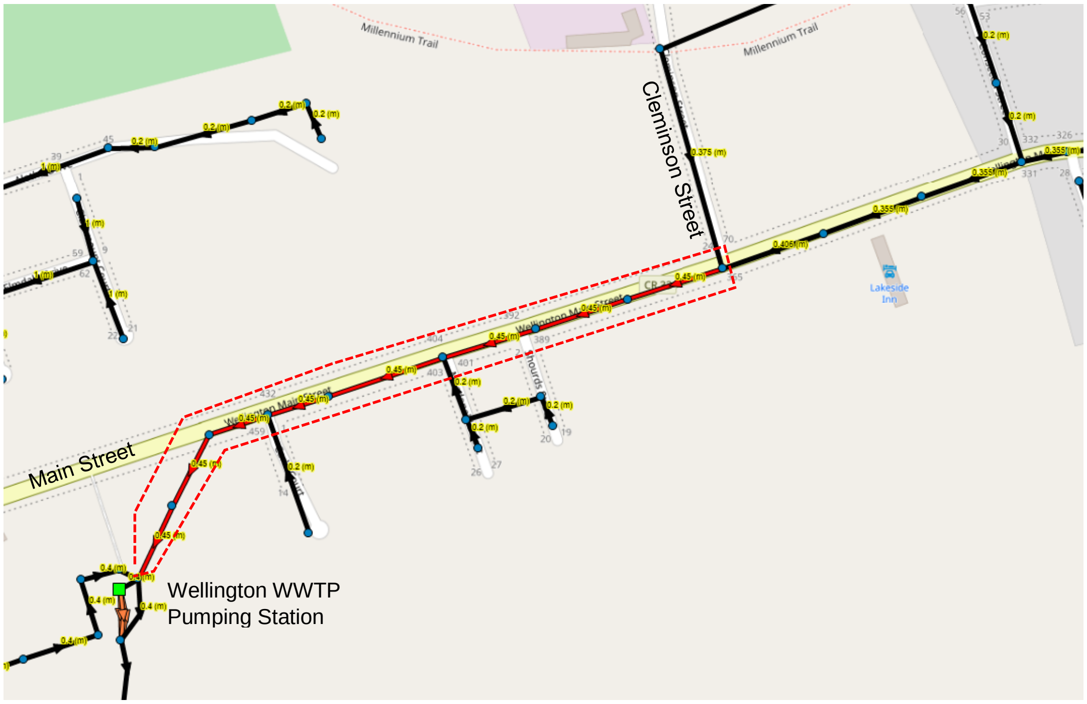

Figure 3.3 – Main Street sewer upgrade to 450 mm (shown as red).

- The capacity of the main pumping station at the Wellington WWTP (approximately 90 L/s) would need to be increased to accommodate a peak flow of 147 L/s. This is a significant increase in pumping capacity and could also require modifications to the header and forcemain, which should be should inspected to confirm sizes and the states of repair.

4.0 CLOSING

We trust the information presented in this report will be sufficient to assist the County in assessing the sanitary sewer system upgrades required to accommodate the four proposed developments. If you have any questions or require additional information, please contact us at your convenience.

Yours very truly,

R.V. ANDERSON ASSOCIATES LIMITED

Troy Poirier, P.Eng.
Associate

APPENDIX E:
WELLINGTON WWTP 2017 AND 2018 BYPASS RECORDS

Wellington STP Bypass Events

B. Keene, P.Eng.

2017	Start	End	Duration	Volume	Attributed Cause
05-May			7 days	Not Reported	Precipitation
29-May	7:12	7:16	0:04	3.75	Power Outage
15-Oct	16:58	21:26	4:28	160.67	Power Outage

2018	Start	End	Duration	Volume	Attributed Cause
15-Jan	14:04	14:08	0:04	5.46	Power Outage
26-Feb	16:22	19:12	2:50	222	Power Outage
26-Mar	18:23	8:09	13:46	302.63	Chlorine Injection Failure
04-Apr	14:06	16:47	2:41	105.84	Power Outage
17-Apr			6 days	Not Reported	Precipitation
30-Jun	5:50	6:14	0:24	8.36	Power Outage
24-Jul	12:12	13:01	0:49	20.75	Power Outage
16-Sep	5:08	6:05	0:57	10.42	Chemical Injection Failure
10-Oct					Not at Plant
17-Oct	14:34	14:36	0:02	0.1864	Power Outage
25-Nov	8:00	9:45	1:45	61.09	Power Outage