STORMWATER MANAGEMENT AND SERVICEABILITY REPORT RÉVISION 1

110 O'CONNOR STREET
OTTAWA

CITY OF OTTAWA, ONTARIO

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ÉQUIPE LAURENCE INC.
File: 60.09.01
September 2025

PROJECT: 110 O'CONNOR STREET – City of Ottawa
Stormwater Management and Serviceability Report

FILE: 60.09.01

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1.0 INTRODUCTION

This project consists of the residential development located at 110 O'Connor Street in the city of Ottawa. Équipe Laurence Inc. was mandated to carry out the design of the drinking water, storm and sanitary sewer systems that serve the proposed building as well as the stormwater management report. The preliminary civil engineering plans depicting the general features of the site, such as the sewer structures and landscaping is attached to this report in Appendix A.

In this report, the design and calculations of the sanitary sewer, domestic water and stormwater management systems will be discussed. The design was completed in accordance with the following design guidelines and regulations:

- Ottawa Sewer Design Guidelines (October 2012)
- *Pre-Consultation Preliminary Assessment* written by Jean-Charles Renaud, Planner III, Development Review- Central. File No. PC2023-0282
- Ottawa Design Guidelines – Water Distribution (July 2010)
- Ottawa Technical Bulletin ISTB-2018-02 (March 2018)
- Water Supply for Public Fire Protection, *Fire Underwriters Survey* (2020)

2.0 STORMWATER MANAGEMENT

As part of the stormwater management system, the flow of water will be controlled on-site and discharged through a 250 mm diameter service connection. This pipe will be connected to the existing 450 mm diameter storm sewer below O'Connor Street as shown on the attached plans.

According to a complementary land survey completed by *Annis, O'Sullivan, Vollebekk Ltd.* on July 18th, 2023, attached in Appendix B, the subject site is primarily occupied by an existing 14 storey precast building and a ramp to an underground parking garage.

For the design of the stormwater management system, the calculations were done to ensure that the 2-year post-development flows are equivalent to or lesser than the pre-development overland flow. Hence, the stormwater flows for the developed site as well as the storage requirements will be explored in the following sections.

2.1 Calculation of Pre-development Flows

The pre-development overland flow was determined using the criteria outlined in the *Ottawa Sewer Design Guidelines (2012)* as well as the following site information:

- The proposed site area of 0.21 hectare.
- The Rational Method for the calculation of flow as indicated in Section 5.4.4.1 of the design guideline.
- The IDF curves and equations as indicated in Section 5.4.2 of the design guideline.
- The runoff coefficients as shown in Table 5.7 of the design guideline.

The time concentration used for the calculations of the 2-yr storm event of the pre-developed site flow is 10 minutes. The runoff coefficient was determined to be 0.5. These specifications were calculated as described in the *Ottawa Design Guidelines*.

Using these values, the pre-development overland flow is 21.3 L/s for the 2-yr storm events. The detailed calculations are attached in Appendix C.

2.2 Design Criteria for Post-Development Flows

According to the *Pre-Consultation Preliminary Assessment*, the allowable release rate to the minor system for the proposed site will be equivalent to the pre-development flow of the 2-year storm event. As mentioned in the previous section, the pre-development flow for the 2-year storm is 21.3 L/s. Moreover, it is mentioned that flows in excess of the 2-yr storm allowable release rate, up to and including the 100-yr storm event, must be retained on site. Hence, these storm events must be considered for the post-development flow calculations.

In addition, to account for the effects of climate change, a 20% increase will be added to the rainfall intensities of the 100-yr storm event, as per the *Ottawa Sewer Design Guideline*, section 8.3.12.

2.3 Catch Basin Sub-Areas

The catch basins sub-areas are used to collect the stormwater from its associated area. The areas of impervious and pervious surfaces are determined for each catch basin. The catch basin sub-areas are depicted in Appendix C.

The runoff coefficient used for the post-development flow calculations of the 100-year storm event for concrete and roof areas is 1.00. The 100-year runoff coefficient is determined by increasing the minor system coefficient by 25%, as per the *Ottawa Sewer Design Guideline*.

Using this information, the average runoff coefficient corresponding to a 100-yr storm event is calculated. The results are shown in Table 1 and the detailed calculations are presented in Appendix C.

Table 1: Average Runoff Coefficients for the Various Catch Basin Sub-Areas

Drainage area	Total area (m^2)	100-year runoff coefficient
CB-01 (covered by roof)	56.8	1.0
CB-02	80.5	1.0
CB-03	80.5	1.0
CB-04	80.5	1.0
CB-05	44.0	1.0
CB-06	44.0	1.0
CB-07	67.7	1.0
Building	1530	1.0
UNR-01	39.1	1.0

2.4 Post-Development: Uncontrolled Flows

For the proposed stormwater management system, there is an uncontrolled flow on the side of the building – i.e. on the surfaces parallel to the O'Connor Street and Slater Street. The total uncontrolled surface is of 39.1 m^2, and the calculated time of concentration is of 10 minutes. Therefore, the uncontrolled flows for the 100-year storm events are 2.3 L/s, from the concrete sidewalks area. The detailed calculations are described in the Appendix C.

The uncontrolled flow will be subtracted to the pre-development flowrate for 2-year event to determine the allowable flowrate for the design recurrence.

2.5 Post-Development: Controlled Flows and Storage Requirements

The controlled flow for the developed site as well as the required storage were calculated using the Rational Method. The outflow to the storm sewers will be the subtraction of the 100-year uncontrolled flow to the 2-year pre-development flow, resulting in a maximum allowable flowrate of 18.8 L/s for the whole site.

Therefore, the project will have a maximum flowrate of 18.8 L/s and a total retention requirement of 81.4 m^3. This is the maximum requirement including the 20% increase for the climate change as required by the city and using the average release rate and a 10% increase to the volume to apply a safety factor. The detailed calculations are found in Appendix C.

Water collected from the roof drains will be directed to an underground concrete tank equipped with an inlet control device (ICD) at the end of the basin, which will control a maximum flow rate of 15.3 L/s. It is important to note that there will be no roof controlled flow drains. In addition, another concrete tank will collect water from the remainder of the project, regulated by a separate ICD to maintain a maximum flow rate of 3.5 L/s.

According to the pre-consultation memo, the City of Ottawa requires an average release rate equal to 50% of the peak allowable rate to estimate the necessary storage volume. Alternatively, two submersible pumps can be used to ensure constant release rates, of 15.3 L/s and 3.5 L/s, respectively. As a result, the required storage will be retained in the two underground concrete tanks, with both submersible pumps conveying water to the proposed manhole outside the building through two 250 mm diameter pipes, as detailed in the Appendix C.

Furthermore, two overflow pipes will be incorporated into each underground tank with an invert at the water retention elevation, directing excess water into the same manhole. The proposed stormwater storage distribution is illustrated in Table 2.

Table 2: Proposed Stormwater Storage - 110 O'Connor Street

Parameters	Values	Units
100-year required storage of the project[1,2]	81.5	m^3
Volume retained in underground concrete tank #1 (from roof drainage)	63.5	m^3
Volume retained in underground concrete tank #2 (from surface water)	19.5	m^3
Total storage volume available	83.0	m^3

1 - A 10% increase was included in the volume requirement as an extra safety measure
2 - A 20% increase to rainfall was included for the climate change effects

The following item related to rooftop drainage will need to be completed by the mechanical and structural engineer responsible for the design:

- Design of the underground concrete tank. See appendix C.

2.6 Erosion and Sediment Control

Prior to, during and after construction, the following erosion and sediment control measures should be implemented to avoid the sediment transfer to existing streams and storm sewer systems.

Pre-Construction

- Installation of a silt fence (geotextile)
- Installation of inserts inside all existing manholes adjacent to construction zone
- Control measures to be inspected once installed
- Installation of a mud mat at the site access point

Construction

- Minimize the extent of disturbed areas
- Protect disturbed areas of runoff
- Provide cover if disturbed areas will not be reinstated within a reasonable period.
- Inspect silt fence regularly during construction. Clean and repair, as required.
- Control dust during construction

After Construction

- Provide permanent cover to disturbed areas (i.e. topsoil and seed)
- Remove all temporary erosion and sediment control items (silt fence and filter cloths) once disturbed areas have been reinstated

Inspections

- Erosion and sediment control measures will be inspected upon completion
- Control measures are to be inspected weekly

All control measures are to be inspected once installed as well as during construction.

3.0 SANITARY SEWER DESIGN FLOWS

The proposed sanitary sewer service connections for the new building is 250 mm in diameter and made of PVC. The pipe will be connected on the existing 450 mm diameter municipal sewer pipe under O'Connor Street.

The proposed sanitary system is designed in accordance with the City of Ottawa's Sewer Design Guidelines. The calculations for the proposed development flows are shown in the following sections.

3.1 Population Density

The population density of the proposed development is calculated using the number and type of housing units within this development. The detailed calculations are shown in Table 4 below and in the Appendix D.

Table 4: Population Density Calculation

Unit Types	Number of Units	Persons Per Unit	Population Density
Studio	128	1.4	179
1-bedroom	183	1.4	256
2-bedroom	80	2.1	168
3-bedroom	22	3.1	68
		Total	672

Using the values in Table 4.2 of the Sewer Design Guidelines for per unit populations, the population density of the proposed development is found to be 672 persons. This value will be used in the following sections to determine the sewer design flows.

3.2 Average Wastewater Flows and Peaking Factors

The average wastewater flow coefficient for residential developments is 280 L/c/d according to the Sewer Design Guidelines. The new building will also include 488 m^2 of commercial areas, therefore the average wastewater flow coefficient for commercial use is 28,000 L/gross ha/d. Using this information, the total average wastewater flow for the proposed development is calculated below.

Average wastewater flow per capita for residential use:	280	L/c/d
Average wastewater flow for residential use:	188 048	L/d
Average wastewater flow for commercial use:	28,000	L/gross ha/d
Commercial areas: 488 m^2	1 368	L/d

The Harmon equation is then used to calculate the residential peak factor. Moreover, a peak factor of 1.50 is used for commercial areas.

$$P.F. = 1 + \left(\frac{14}{4 + \left(\frac{P}{1000}\right)^{1/2}} \right) \times K, \qquad where\ K = 1$$

Hence, the peak factor for residential use is of 3.9.

3.3 Extraneous Flows

In accordance with Article 4.4.1.4 of the Sewer Design Guidelines, an allowance for flows from extraneous sources must be considered in the calculation of the peak design flow.

The average infiltration allowance is of 0.28 L/s/gross ha for wet-weather inflow into the manholes and pipes. Therefore, with a total site area of 2.092 ha, the infiltration flow is 0.59 L/s.

3.4 Total Sanitary Sewer Design Flow

Combining the results from the above calculations, the total sanitary sewer design flow is calculated as follows:

$$Q_{design} = [(3.90 \times 188\,048\ L/d) + (1.50 \times 1\,368\ L/d)] \times \frac{1}{86\,400\ sec/d} + 0.59\ L/s$$

$$Q_{design} = 9.11\ L/s$$

The summary of this calculation is shown in Appendix D.

4.0 DOMESTIC WATER DEMAND

The proposed water service connection for the new building will consist of two separate branch connections: one on O'Connor Street and one on Slater Street. Each connection will be 200 mm in diameter and made of PVC. The first connection will be connected to the existing 406 mm diameter municipal watermain on O'Connor Street, while the second on the existing 381 mm diameter municipal watermain on Slater Street. Two shutoff valves will be installed at the property line for each connection as per the City guidelines. Additionally, both connections will be looped at the service entry inside the building, and an isolation valve will be placed between the two water service connections.

The proposed water system is designed in accordance with the City of Ottawa's Design Guidelines for water distribution. The calculations for the proposed water demand are shown in the following sections.

We can determine the average day demand for the proposed development using the values found in Table 4.2 of the Design Guidelines as the population density of the development was determined to be 672 people in Section 2.1. Hence, average day demands of 280 L/c/d and 28,000 L/gross ha/d are used for the residential and commercial spaces, respectively.

Average day demand per capita for residential use:		280	L/c/d
Average day demand for residential use:		188 048	L/d
Average day demand for other commercial use:		28,000	L/gross ha/d
Commercial Area:	488 m²	1 368	L/d

Therefore, the total average day demand is:

$$Q_{avg,day} = \left(188\ 048\frac{L}{d} + 1\ 368\ L/d\right) \times \frac{1}{86{,}400} sec/d = 2.19\ L/s$$

The maximum daily demand and the maximum hour demand are calculated using the factors found in Table 4.2 of the Design Guidelines.

$$Q_{max,day} = \left(2.5 \times 188\ 048\frac{L}{d} + 1.5 \times 1\ 368\ \ L/d\right) \times \frac{1}{86{,}400} sec/d = 5.46\ L/s$$

$$Q_{max,hr} = \left(2.2 \times 2.5 \times 188\ 048\frac{L}{d} + 1.8 \times 1.5 \times 1\ 368\ L/d\right) \times \frac{1}{86{,}400} sec/d$$

$$Q_{max,hr} = 12.01\ L/s$$

The detailed calculations for domestic water demand are found in Appendix E.

4.1 Boundary Conditions

This section presents the existing boundary conditions for the water distribution system for the connection sites. Note, this information is based on current operation of the city's water distribution system. See the boundary conditions received from the city of Ottawa in table 5.

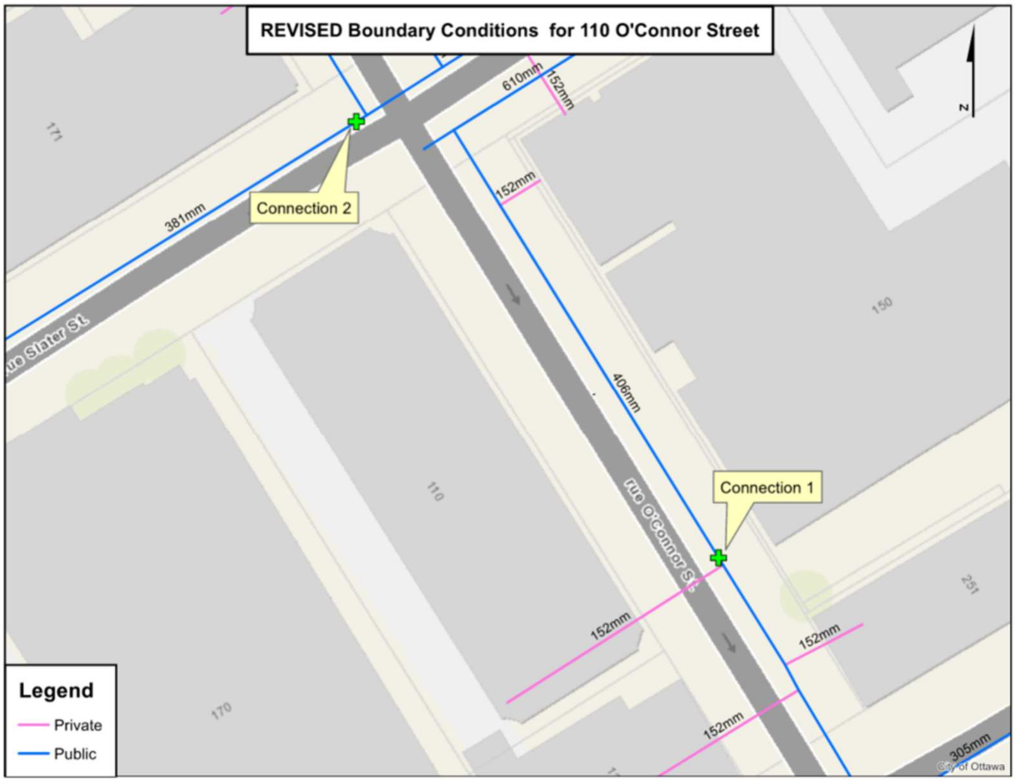

Figure 1: Service connection locations for the water distribution system (City of Ottawa)

Table 5: Boundary conditions received from the City of Ottawa

Demand Scenario	Head (m)
Maximum HGL	115.5
Minimum HGL	106.8
Max Day + Fire Demand (100 L/s)	109.6

The pressure at the service points on O'Connor Street and Slater Street has been calculated, with detailed calculations provided in Appendix E of this report.

It must be noted that the static pressure at any fixture shall not exceed 552 kPa (80 psi) according to the Ontario Building Code for areas that may be occupied. Hence, the following pressure control measures shall be considered:

1. If possible, the systems are to be designed to residual pressures 345 to 552 kPa (50 to 80 psi) for all occupied areas outside of the public right-of-way without special pressure control equipment.

2. Pressure reducing valves are to be installed immediately downstream of the isolation valve in the building, located downstream of the meter so that it is maintained by the owner.

These pressure control measures are presented in order of preference.

5.0 REQUIRED FIRE DEMAND

The flow rates required for fire protection vary according to the zoning, the type of units, the fire resistivity of the construction materials, the ground floor area as well as many other factors. The method described in *Water Supply for Public Fire Protection*, written by the Fire Underwriters Survey (FUS) (2020) is used to estimate the fire demand required for fire protection, as per the City Guidelines.

Essentially, the required flow rate (F), expressed in liters per minute, is calculated based on the floor area of the building (A) in square meters and the type of construction (C), using the following equation.

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

The value of C used is 0.6 for a fire resistive construction. According to the FUS, a fire resistive construction is "any structure having all structural members including walls, columns, piers, beams, girders, trusses, floors and roofs made of non-combustible material and constructed with a minimum 2-hour fire resistance rating." In this case, the building will be full non-combustible construction both for the construction type and exterior cladding.

The value of A represents the gross floor area of the building, that is, the sum of the surface area of all floors. See in the table below that surface area of each floor. The effective area is to be calculated as per the 2020 regulations for the Water Supply for Public Fire Protection in Canada, the total effective area is to be calculated as the largest floor with the addition of 25% of the next 2 adjacent floors.

Table 6: Gross Floor Area for the Proposed Development

Floor	Surface Area Per Floor (m²)	Number of Floors	Floor Area (m²)
Ground Floor	1216	1	1216
Level 2	1460	1	1460
Levels 3-6	1471	4	5882
Levels 7-25	967	19	18 366
Roof	479	1	479
		Total	27 403

Finally, according to the FUS method, certain reductions and increases may be applied depending on a variety of factors such as the combustibility of the occupying materials or furniture, the presence of automatic sprinklers systems as well as the development's distance from neighbouring buildings. For example, for buildings protected by automatic sprinklers designed in accordance with the NPFA 13, the flow rate required for fire protection, F, can be reduced by 50%.

Using this method, the total fire demand was determined to be 6000 L/min. Moreover, for a duration of water supply of 2 hours, the required volume of water is 720 m³. The details of the fire flow calculations are shown in the Appendix F.

6.0 REFERENCES

W.R. Newell, P. Eng., Sewer Design Guidelines, Second Edition (2012), City of Ottawa.

W.R. Newell, P. Eng., Ottawa Design Guidelines – Water distribution, First Edition (2010), City of Ottawa.

Fire Underwriters Survey, Water Supply for Public Fire Protection – A guide to recommended practice in Canada (2020).

APPENDIX A

Civil Engineering Plans

APPENDIX B

Land Survey by Annis, O'Sullivan, Vollebekk Ltd. on July 18th, 2023

ELEVATION NOTES

1. Elevations shown are geodetic and are referred to the CGVD28 geodetic datum.
2. It is the responsibility of the user of this information to verify that the job benchmark has not been altered or disturbed and that it's relative elevation and description agrees with the information shown on this drawing.

UTILITY NOTES

1. This drawing cannot be accepted as acknowledging all of the utilities and it will be the responsibility of the user to contact the respective utility authorities for confirmation.
2. Only visible surface utilities were located.
3. A field location of underground plant by the pertinent utility authority is mandatory before any work involving breaking ground, probing, excavating etc.

SURVEYOR'S REAL PROPERTY REPORT

PART 1 Plan of **LOT 43 and PART OF LOT 42 (South Slater Street) PART OF LOTS 42 and 43 (North Laurier Avenue) REGISTERED PLAN 3922 CITY OF OTTAWA**

Surveyed by Annis, O'Sullivan, Vollebekk Ltd.

Scale 1 : 150

6 4.5 3.0 1.5 0 3 6 Metres

Metric

DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048

Surveyor's Certificate

I CERTIFY THAT :

1. This survey and plan are correct and in accordance with the Surveys Act, the Surveyors Act and the Land Titles Act and the regulations made under them.
2. The survey was completed on the 4th day of July, 2023.

July 18, 2023
Date

E. H. Herweyer
Ontario Land Surveyor

PART 2
THIS PLAN MUST BE READ IN CONJUNCTION WITH SURVEY REPORT DATED : JULY 14, 2023.

ANNIS, O'SULLIVAN, VOLLEBEKK LTD. grants to GROUPE MACH ("The Client"), their solicitors, mortgagees, and other related parties, permission to use original, signed, sealed copies of the Surveyor's Real Property Report in transactions involving The Client.

Notes & Legend

—□—	Denotes	Survey Monument Planted
—■—	"	Survey Monument Found
SIB	"	Standard Iron Bar
SSIB	"	Short Standard Iron Bar
IB	"	Iron Bar
CC	"	Cut Cross
CP	"	Concrete Pin
(WIT)	"	Witness
(AOG)	"	Annis, O'Sullivan, Vollebekk Ltd.
Meas.	"	Measured
Acc.	"	Accepted
(P1)	"	Plan 4R-401
(P2)	"	Plan 4R-19750
(P3)	"	(857) Plan May 12, 1980
(P4)	"	(857) Plan January 19, 1970
(P5)	"	(647) Plan September 29, 1982
(P6)	"	(857) Plan March 14, 2006
(P7)	"	(AOG) Plan May 27, 2019
(D1)	"	Inst No. NS34440
MF	"	Metal Fence
CLF	"	Chain Link Fence
CRW	"	Concrete Retaining Wall
+ 65.00	"	Location of Elevations
+ 65.00	"	Top of Concrete Curb / Wall Elevation
VC	"	Valve Chamber (Watermain)
MH–ST	"	Maintenance Hole (Storm Sewer)
MH-S	"	Maintenance Hole (Sanitary)
MH-B	"	Maintenance Hole (Bell Telephone)
MH-T	"	Maintenance Hole (Traffic)
MH-H	"	Maintenance Hole (Hydro)
C/L	"	Centreline
———	"	Property Line
B	"	Bollard
C/L	"	Centreline
GM	"	Gas Meter
T/G	"	Top of Grate
Fdn.	"	Foundation
WV	"	Water Valve
FP	"	Pole
TSL	"	Traffic Light
CB	"	Catch Basin
GV	"	Gas Valve

ASSOCIATION OF ONTARIO LAND SURVEYORS
PLAN SUBMISSION FORM
V-53839
THIS PLAN IS NOT VALID UNLESS IT IS AN EMBOSSED ORIGINAL COPY ISSUED BY THE SURVEYOR In accordance with Regulation 1026, Section 29 (3).

Bearings are grid derived from the northerly limit of Laurier Avenue West shown to be N59° 00'10"E on a Plan by AOG) Dated May 24, 2019, and are referred to the Central Meridian of MTM Zone 9 (76°30' West Longitude) NAD-83 (original).

For bearing comparisons, a rotation of 0°56'40" counter-clockwise was applied to bearings on plan P1.

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Ontario Land Surveyors

Job No. 24412-23 GroupeMACH 110O'ConnorSt POS F | DG

APPENDIX C

Stormwater Flows and Storage Requirements

Detailed calculations

Storage tank drawing

STORMWATER CALCULATIONS

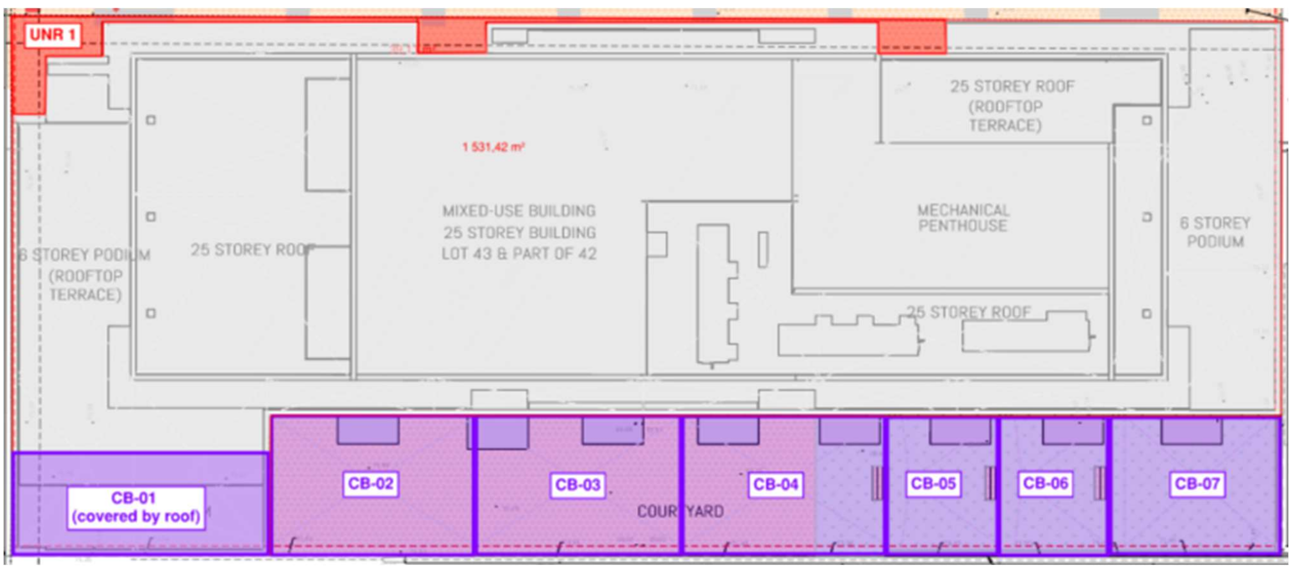
IDF CURVES FOR THE CITY OF OTTAWA

IDF curve equations (Intensity in mm/hr)

100 year Intensity	$= 1735.688 / (\text{Time in min} + 6.014)^{0.820}$
50 year Intensity	$= 1569.580 / (\text{Time in min} + 6.014)^{0.820}$
25 year Intensity	$= 1402.884 / (\text{Time in min} + 6.018)^{0.819}$
10 year Intensity	$= 1174.184 / (\text{Time in min} + 6.014)^{0.816}$
5 year Intensity	$= 998.071 / (\text{Time in min} + 6.053)^{0.814}$
2 year Intensity	$= 732.951 / (\text{Time in min} + 6.199)^{0.810}$

WATERSHED

The watersheds of the project are as displayed in the drawing below. The red zones represent the areas that are considered uncontrolled flow as the water will lease the site without control. The other watersheds are named based on the catch basin numbers associated.



HYPOTHESE

- The roof is a part of the drainage areas draining downstream of one of the underground tank.

Here are the calculations for the pre-development flowrate as asked by the city. The IDF curves provided above and a runoff coefficient of 0.50 were used.

TABLE 1 - 2-YEAR PRE-DEVELOPMENT

Time of concentration (min)	Intensity (mm/hr)	Flowrate (L/s)
5.0	103.57	0.029
10.0	**76.81**	**0.021**
15.0	61.77	0.017
20.0	52.03	0.014

*The IDF curves were taken from the city of Ottawa sewer design guidelines and C=0.50.

*The total area of the project is 1 975m^2

TABLE 2 – PROPOSED POST-DEVELOPMENT CATCHMENT AREAS

Drainage area	Total area (m²)	Impervious surfaces		Grass surfaces		100-year runoff coefficient
		Area (m²)	Runoff coefficient	Area (m²)	Runoff coefficient	
CB-01 (covered by roof)	56.8	56.8	0.9	0	-	1.0
CB-02	80.5	80.5	0.9	0	-	1.0
CB-03	80.5	80.5	0.9	0	-	1.0
CB-04	80.5	80.5	0.9	0	-	1.0
CB-05	44.0	44.0	0.9	0	-	1.0
CB-06	44.0	44.0	0.9	0	-	1.0
CB-07	67.7	67.7	0.9	0	-	1.0
Building	1530	1530	0.9	0	-	1.0
Total Regulated	1975	1975	-	0	-	1.0
UNR-01	39.1	39.1	0.9	0	-	1.0
Total Unregulated	39.1	39.1	-	0	-	1.0

RUNOFF COEFFICIENT CALCULATION

$$C = \frac{\sum(Ai \times Ci)}{\sum A}$$

Where Ai is the Area of a certain material type

Ci is the runoff coefficient of a certain material type

Example:

$$C_{CB-04} = \frac{698 \times 0.900 + 186 \times 0.250}{698 + 186} = 0.763$$

TABLE 3 - PROPOSED UNCONTROLLED FLOW

Parameters	Values	Units
Impervious surfaces	39.1	m²
Grass surfaces	0	m²
Total area	39.1	m²
100-year Runoff coefficient	1.0	-
Time of concentration	10	min
Uncontrolled 100-year flow	2.3	ℓ/s

* The 100-year runoff coefficients are determined by increasing the 2-year runoff coefficients by 25% as per the city of Ottawa sewer design guidelines.

TABLE 4 - PROPOSED CONTROLLED FLOW

Parameters	Values	Units
2-year pre-development flow	21.1	ℓ/s
100-year uncontrolled flow	2.3	ℓ/s
Allowable release rate / Controlled flow	18.8	ℓ/s
Average release rate for calculations	18.8	ℓ/s
Release rate controlled by ICD 1 (submersible pump)	15.3	ℓ/s
Release rate controlled by ICD 2 (submersible pump)	3.5	ℓ/s
100-year storage requirement *	81.3	m^3

*Storage requirement calculations includes a 20% increase in rainfall

*Storage requirement calculations includes a 10% increase in volume

TABLE 5 - PROPOSED STORMWATER STORAGE

Parameters	Values	Units
100-year required storage[1,2]	81.3	m^3
Volume retained in underground concrete tank #1	63	m^3
Volume retained in underground concrete tank #2	18.3	m^3
Total storage volume available	81.3	m^3

1 - A 10% increase was included in the volume requirement as an extra safety measure

2 - A 20% increase to rainfall was included for the climate change effects

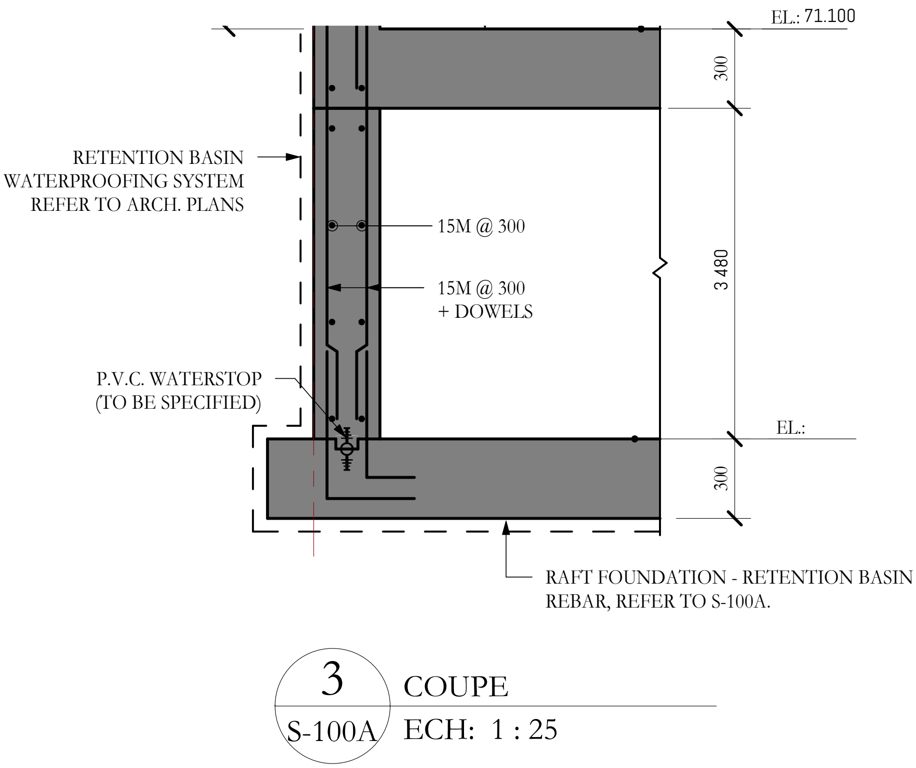
TABLE 6 – INLET CONTROL DEVICE (ICD)

Zone	Pipe	Flowrate (L/s)	Water level	Invert (m)	Water head (m)	Type *
1	250 mm PVC	15.3	70.120	68.940	1.180	Submersible pump
2	250 mm PVC	3.5	70.120	68.940	1.180	Submersible pump

*The type of ICD and specifications has to be validated with the manufacturer and mechanical engineer

Concrete Tank Design

Detail is from the structural plans S-200.



TANK #1

MAXIMUM WATER LEVEL : 70.120

HEIGHT : 3.48 m

MAXIMUM VOLUME : 63.5 m3

CONCRETE TANK INVERT : 68.940

TANK #2

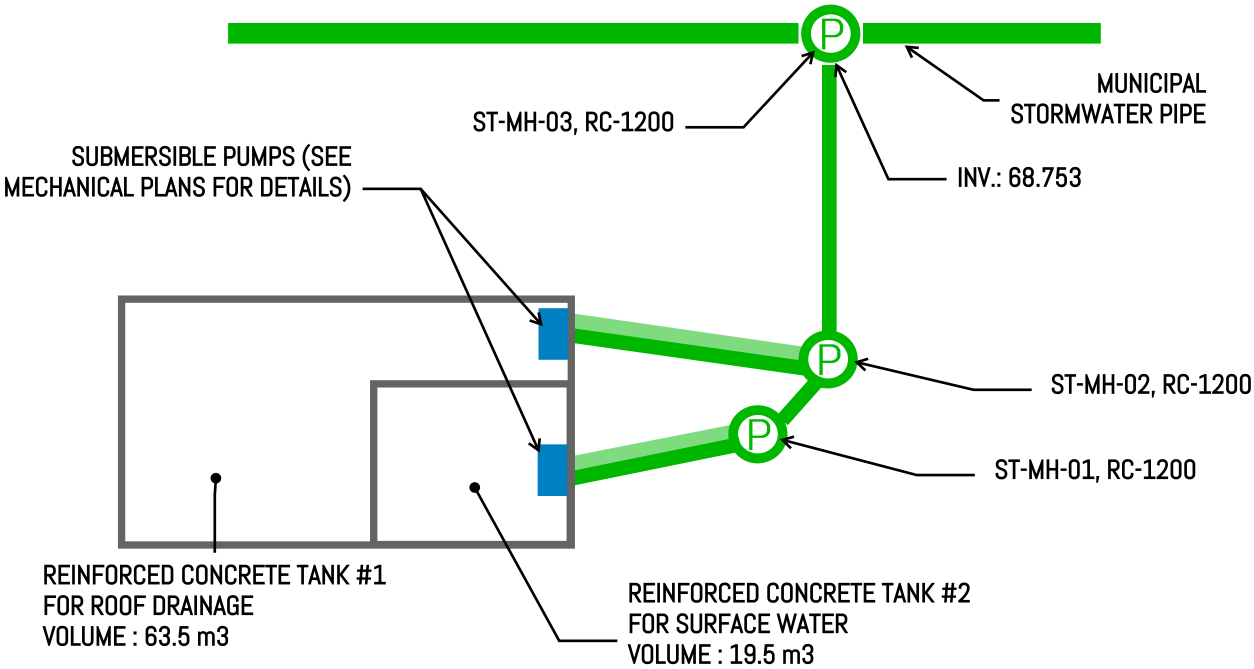
MAXIMUM WATER LEVEL : 70.120

HEIGHT : 3.48 m

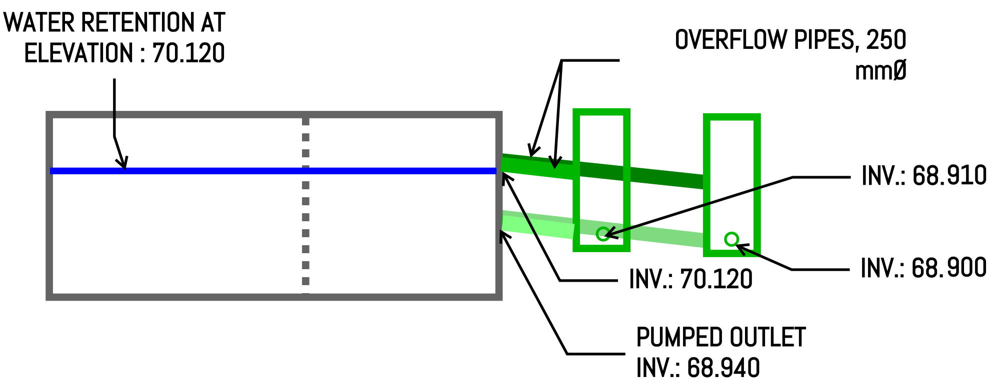
MAXIMUM VOLUME : 19.5 m3

CONCRETE TANK INVERT : 68.940

UNDERGROUND CONCRETE TANK DETAILS



PLAN VIEW



PROFIL VIEW

APPENDIX D

Sanitary Sewer Design Flows

Detailed Calculations

File : 600901
Project : 110 O'Connor Street

SANITARY SEWER DESIGN FLOWS - 110 O'Connor

Reference : Ottawa Sewer Design Guidelines, *Infrastructure Services Department* , October 2012

A. Population Density

(Article 4.3, Table 4.2)	Number of units	Persons Per Unit	Population Density
Studio	128	1,4	179
1-bedroom	183	1,4	256
2-bedroom	80	2,1	168
3-bedroom	22	3,1	68

Total population density: 671,6

B. Average Wastewater Flows

(Article 4.4.1, Figure 4.3)

Average wastewater flow per capita for residential use: 280 L/c/d
Average wastewater flow for residential use: 188 048 L/d

Average wastewater flow for commercial use: 28 000 L/gross ha/d
Commercial Areas: 488 m² 1 368 L/d

C. Peaking Factors

(Article 4.4.1, Figure 4.3)

Residential peak factor: Harmon Equation
K=1

$$P.F. = 1 + \left(\frac{14}{4 + \left(\frac{P}{1000}\right)^{1/2}}\right) \times K$$

Residential peak factor: 3,90
Commercial peak factor: 1,50

D. Extraneous Flows

(Article 4.4.1.4)

Infiltration allowance: 0,28 L/s/effective gross ha for 2.092 ha
Inflitration flow: 0,59 L/s

F. Total Wastewater Design Flow

Q_{design} = [(3.90 x 188 048 L/d) + (1.50 x 1 368 L/d)] x 1/86 400 sec/d + 0.59 L/s

Q_{design} = 9,11 L/s

SANITARY SEWER CALCULATION SHEET

Manning's n = 0,013

LOCATION			RESIDENTIAL AREA AND POPULATION						COMM		INDUST			INST		C+I+I	INFILTRATION				PIPE				
STREET	FROM M.H.	TO M.H.	AREA (ha)	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FACTOR (per	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	LENGTH (m)	DIA. (mm)	SLOPE (%)	CAP. (FULL) (l/s)	VEL. (FULL) (m/s)
O'Connor	BUILD.	SAN-MH-01	0,21	672			3,91	8,50	0,0576	0,0576	-	-	1,5	-	-	0,0280	0,2668		0,59	9,12	2,75	250	2,00	84,10	1,71
O'Connor	SAN-MH-01	City																			6,90	250	2,00	84,10	1,71

DESIGN PARAMETERS

Average Daily Flow =	280 l/p/day	Industrial Peak Factor =	as per MOE Graph
Comm/Inst Flow =	28000 L/ha/da	Extraneous Flow =	0,28 L/s/ha
Industrial Flow =	28000 L/ha/da	Minimum Velocity =	0,86 m/s
Max Res. Peak Factor =	3,90	Manning's n =	0,013
Commercial / Inst Peak Factor =	1,50		

Designed:	PROJECT: 110 O'CONNOR		
Checked:	LOCATION: 110 O'CONNOR STREET, OTTAWA, ON		
Dwg. Reference: C-204	File Ref.: 600901	Date: 2024-11-01	Sheet No. 1 of 1

APPENDIX E

Domestic Water Demand

Detailed Calculations

Watermain Pressure

File : 600901
Project : 110 O'Connor Street

DOMESTIC WATER DEMAND CALCULATION

Reference : Ottawa Design Guidelines - Water Distribution, *Infrastructure Services department* , July 2010

A. Population Density

(Article 4.2.8, Table 4.1)	Number of units	Persons Per Unit	Population Density
Studio	128	1,4	179,2
1-bedroom	183	1,4	256,2
2-bedroom	80	2,1	168
3-bedroom	22	3,1	68,2
		Total population density:	672

B. Average Day Demand

(Article 4.2.8, Table 4.2)

Average day demand per capita for residential use: 280 L/c/d
Average day demand for residential use: 188 048 L/d

Average day demand for other commercial use: 28 000 L/gross ha/d
Commercial Areas: 488 m² 1 368 L/d

Total average day demand: 189 416 L/d = 2,19 L/s

C. Maximum Daily Demand

(Article 4.2.8, Table 4.2)

Maximum daily demand = 2.5 x 188 048 L/d + 1.5 x 1 368 L/d
= 470 120 L/d + 2 052 L/d = 472 171 L/d
= 5,46 L/s

D. Maximum Hour Demand

(Article 4.2.8, Table 4.2 and Technical Bulletin ISD-2010-2)

Maximum hour demand = 2.2 x (Max Day$_{res)}$ L/d + 1.8 x (Max Day$_{com)}$ L/d
Maximum hour demand = 2.2 x 470 120 L/d + 1.8 x 2 052 L/d = 1 037 957 L/d
= 12,01 L/s

F. Results

Population density = 672 people
Average day demand = 2,19 L/s

File : 600901
Project : 110 O'Connor Street

Maximum daily demand =	5,46	L/s
Maximum hour demand =	12,01	L/s

Valérie,

The following are the boundary conditions and HGL for hydraulic analysis at 110 O'Connor Street (zone 1W) assumed connected via two connections to the 406mm watermain on O'Connor Street AND the 381mm watermain on Slater Street (see attached PDF for location).

Both Connections:
Minimum HGL: 106.8 m
Maximum HGL: 115.5 m
Max Day + Fire Flow (100 L/s): 109.6 m

These are for current conditions and are based on computer model simulation.

Disclaimer: The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation.

Regards,

Brett Hughes BEng.
Project Manager, Infrastructure
Development Review Central
PLANNING, DEVELOPMENT & BUILDING SERVICES (PDBS)
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1. Data and hypothesis

Maximum flow

Fire flow	6000	L/min	=	0,1	m^3/s
Max daily demand	5,5	L/s	=	0,0055	m^3/s
Max total flow	0,1055	m^3/s			

*The max flow rate will be used for both service points pressure loss calculations. The calculated pressure drops will be conservative and will assume that both service points can handle the full fire flow if the other service point is not in use.

Piping between the O'Connor Street service point and the water main valve

Pipe nominal diameter	200	mm
Pipe material	PVC DR-18	
Pipe inside diameter	204	mm
Pipe length	12,36	m

Piping between the Slater Street service point and the water main valve

Pipe nominal diameter	200	mm
Pipe material	PVC DR-18	
Pipe inside diameter	204	mm
Pipe length	12,79	m

Pressure data

Minimum HGL	106,8	m
Maximum HGL	115,5	m

*The HGL value is taken from a computer model simulation of the network and is the same for both service points

O'Connor Street service point elevation	71,32	m
Slater Street service point elevation	70,86	m

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2.0 Street service point pressure

2.1 Pressure at the O'Connor street service point (ground level)

Maximum flow pressure	35,48 m	=	50,42	psi
Minimum flow pressure	44,18 m	=	62,78	psi

2.2 Pressure at the Slater street service point (ground level)

Maximum flow pressure	35,94 m	=	51,07	psi
Minimum flow pressure	44,64 m	=	63,43	psi

3.0 Pressure loss betweet the street service point and the water main valve

3.1 Pressure at the building water main valve on O'Connor Street (ground level)

Dynamic pressure loss

Hazen-Williams equation
Équation de Hazen-Williams :

$$Hf = 10{,}654 \times \left(\frac{Q}{C}\right)^{\frac{1}{0{,}54}} \times \left(\frac{1}{D^{4{,}87}}\right) \times L$$

Q =	Flow rate (m^3/s)	0,1055	(m^3/s)
C =	Hazen-Williams coefficient	130	*Hypothesis, new PVC pipe
D =	Pipe internal diameter	0,204	m
L =	Pipe length	12,36	m
Hf =	Friction pressure loss	0,57	m
	Security factor	10%	
Hf =	Friction pressure loss	0,90	psi

Static pressure loss

Ground elevation at the street service point	71,32	m
Ground elevation at the water main valve	71,38	m

Static pressure loss 0,06 m = 0,085 psi

Result

Dynamic pressure at the water main valve	49,4	psi
Static pressure at the water main valve	61,8	psi

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3.2 Pressure at the building water main valve on Slater Street (ground level)

Dynamic pressure loss

Hazen-Williams equation
Équation de Hazen-Williams :

$$Hf = 10{,}654 \times \left(\frac{Q}{C}\right)^{\frac{1}{0{,}54}} \times \left(\frac{1}{D^{4{,}87}}\right) \times L$$

Q =	Flow rate (m³/s)	0,1055	(m³/s)
C =	Hazen-Williams coefficient	130	*Hypothesis, new PVC pipe
D =	Pipe internal diameter	0,204	m
L =	Pipe length	12,79	m
Hf =	Friction pressure loss	0,59	m
	Security factor	10%	
Hf =	Friction pressure loss	0,93	psi

Static pressure loss

Ground elevation at the street service point	70,86	m
Ground elevation at the water main valve	70,87	m

Static pressure loss 0,01 m = 0,014 psi

Result

Dynamic pressure at the water main valve	50,1	psi
Static pressure at the water main valve	62,5	psi

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Conclusion

According to the Design Guideline for Drinking-Water Systems, chapter 10, the minimum pressure under maximum day demand plus fire flow is 20 psi and the minimum pressure in normal operation is 40 psi.

O'Connor Street service point

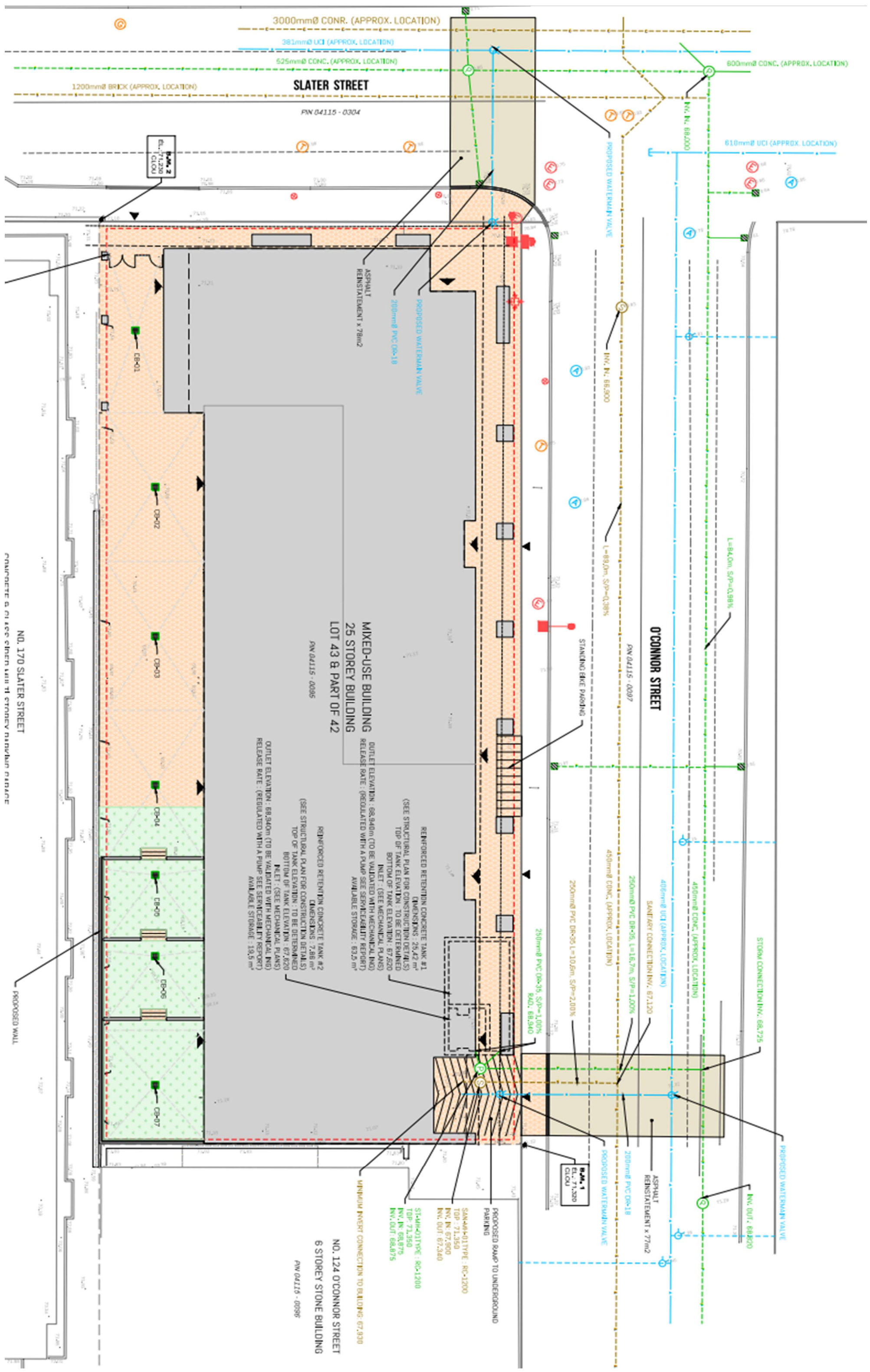
For the O'Connor Street service point, the calculated dynamic pressure (49,4 psi) is greater than the minimum of 20 psi and the calculated static pressure (61,8 psi) is greater than the minimum of 40 psi.

The pressure on the O'Connor Street service point is therefore compliant to the Design guideline for Drinking-Water Systems.

Slater Street service point

As for the Slater street service point, the calculated dynamic pressure (50,2 psi) is greater than the minimum of 20 psi and the calculated static pressure (62,5 psi) is greater than the minimum of 40 psi.

The pressure on the Slater Street service point is therefore compliant to the Design guideline for Drinking-Water Systems.



Taken from the plans 110 O'CONNOR STREET, OTTAWA, PLAN VIEW, SITE SERVICING PLAN AND DRAINAGE AREA, issued for SITE PLAN APPLICATION on november 29th 2024, by Équipe Laurence

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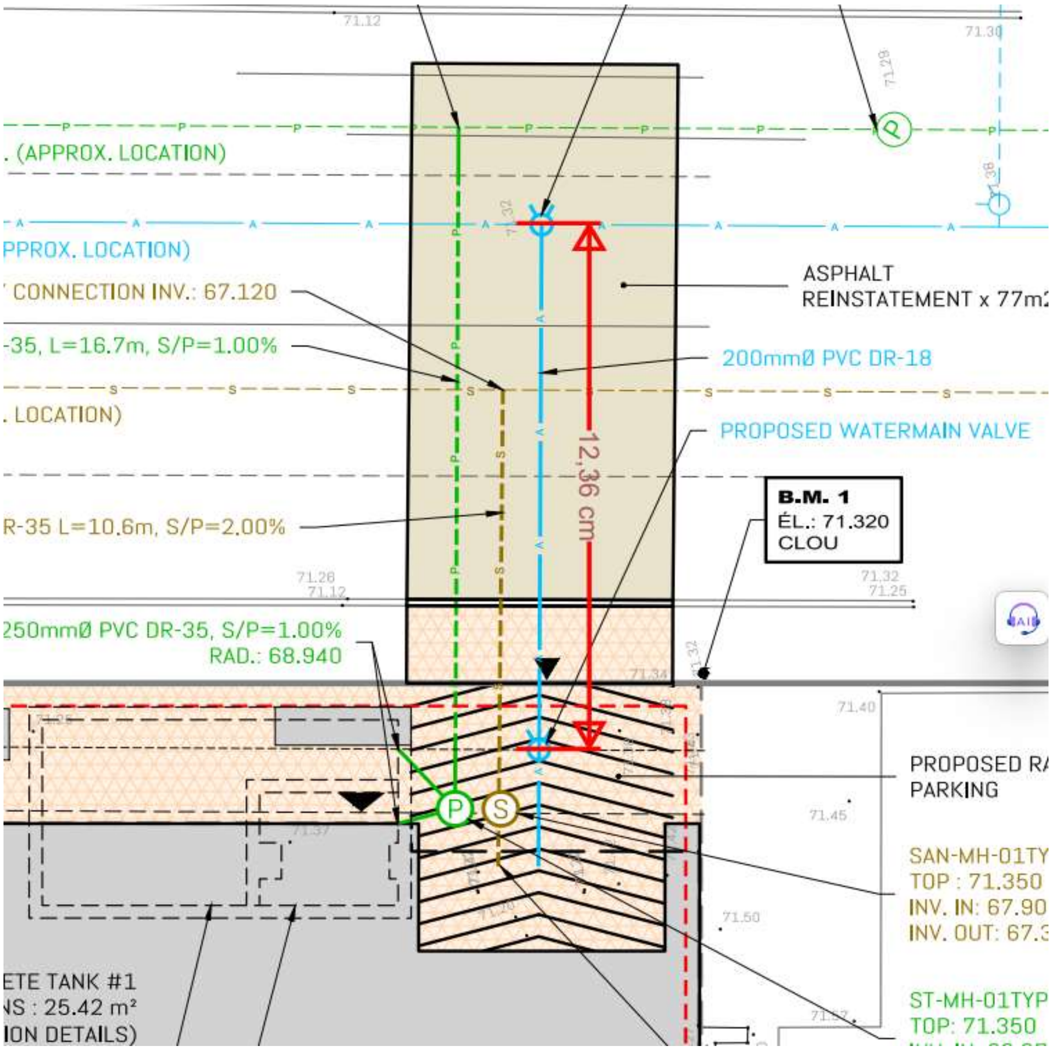
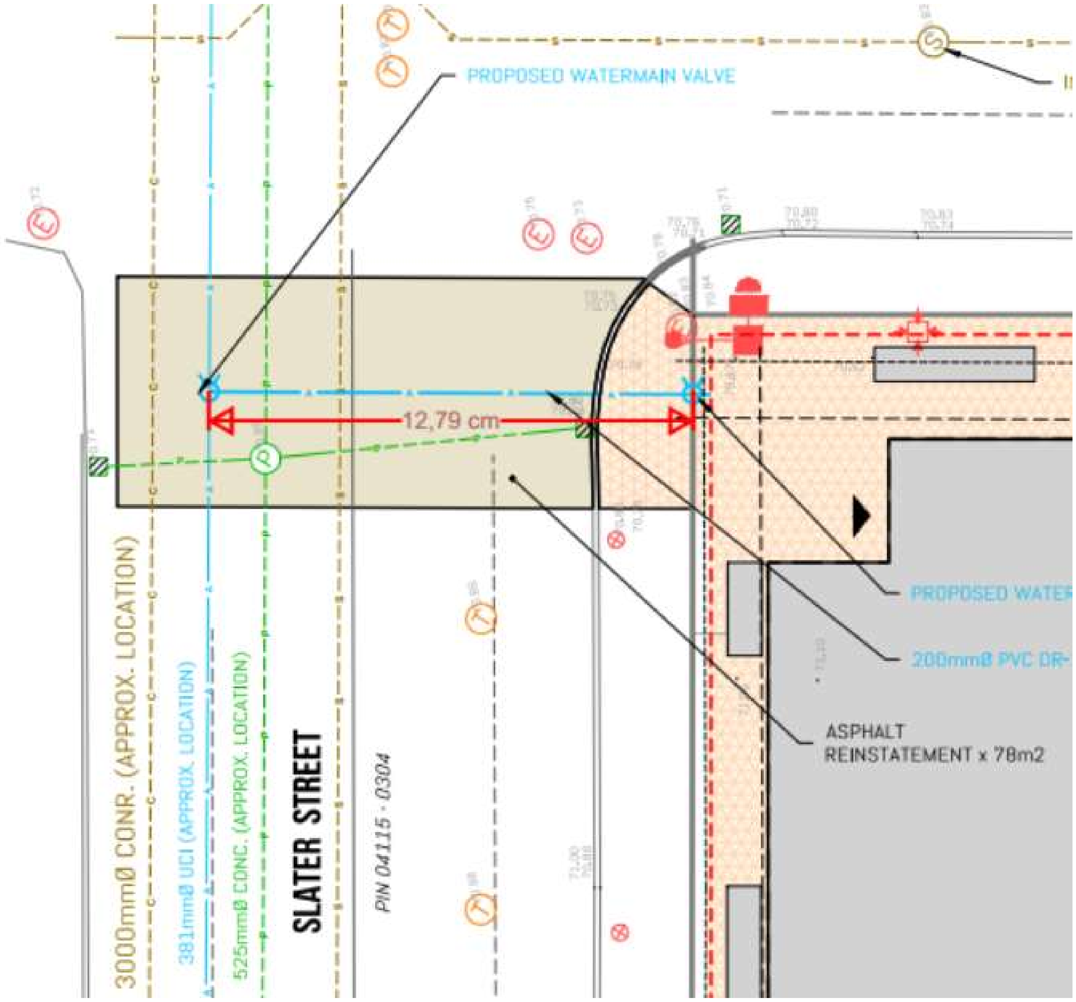


O'Connor street service point, taken from the plans 110 O'CONNOR STREET, OTTAWA, PLAN VIEW, SITE SERVICING PLAN AND DRAINAGE AREA, issued for SITE PLAN APPLICATION on november 29th 2024, by Équipe Laurence



Slater Street service point, taken from the plans 110 O'CONNOR STREET, OTTAWA, PLAN VIEW, SITE SERVICING PLAN AND DRAINAGE AREA, issued for SITE PLAN APPLICATION on november 29th 2024, by Équipe Laurence

APPENDIX F

Required Fire Demand

Detailed Calculations

REQUIRED FIRE DEMAND CALCULATION

References : Ottawa Technical Bulletin ISTB-2018-02, March 2018
Water Supply for Public Fire Protection, *Fire Underwriters Survey*, 2020

A. Type of construction

Fire Resistive Construction (Class 6): C = 0,6

B. Total Effective Area

	Surface Area Per Floor	Number of Floors	Floor Area
Ground Floor	1 216 m²	1	1 216 m²
Levels 2	1 460 m²	1	1 460 m²
Levels 3-6	1 471 m²	4	5 882 m²
Levels 7-25	967 m²	19	18 366 m²
Roof	479 m²	1	479 m²

A = Largest floor area + 25% of each of the two immediately adjoining floors

A = 1 471m^2 + 25% * 1 460m^2 + 25% * 1471 m^2

A = 2203 m^2

D. Base Fire Flow

$\boxed{F = 220 \times C\sqrt{A}}$ = 6 196 L/min

The base fire flow must be rounded to the nearest 1,000 L/min, hence : *F =* 6 000 L/min

E. Fire Flow Adjustments

E.1 Building occupancy (adjustments to the value obtained in D) 1

Occupancy : Limited Combustible -15% *F =* 5 100 L/min

E.2 Automatic sprinkler system (adjustments to the value obtained in E.1)

NPFA 13 Designed system:	Yes	-30%
Standard water supply:	Yes	-10%
Fully supervised system:	Yes	-10%

E.3 Exposure surcharge (adjustments to the value obtained in E.1)

Lenght-Height Factors (no impact on exposure surcharge calculations since distances > 30m)

North side L (121m) * H (22 storeys) = 2662

East side	L (69.3m) * H (18 storeys) =	1139
South side	L (40,4m) * H (6 storeys) =	242
West side	L (85.5m) * H (3 storeys) =	257

North side	21.7 m (20.1 to 30 m)	8%
East side	23.4m (20.1 to 30 m)	10%
South side	1.7 m (0 to 3 m)	20%
West side	3 m (0 to 3 m)	25%

Reductions from E.2 =	-50%	=	-2 550	L/min	2
Increases from E.3 =	63%	=	3 213	L/min	3

1 + 2 + 3 *F =* 5 763 L/min

The fire flow must be rounded to the nearest 1,000 L/min, hence : *F =* 6 000 L/min

F. volume of Water Required During the Fire

The duration of water supply for a fire is: 2 hours

Required Volume = 720 000 L = 720 m³

Fire Demand =	6 000	L/min	Required Volume =	720	m³