SERVICING AND STORMWATER MANAGEMENT REPORT - 355 FRANKTOWN ROAD

Project No.: CCO-22-0402

Prepared for:

11309455 Canada Inc
190 Lisgar St,
Ottawa, ON L2P 0C4

Prepared by:

McIntosh Perry Consulting Engineers Ltd.
115 Walgreen Road
Carp, ON K0A 1L0

July 15, 2022

Rev 04: June 14, 2024

McINTOSH PERRY

TABLE OF CONTENTS

LIST OF TABLES

APPENDICES

1.0 PROJECT DESCRIPTION

1.1 Purpose

McIntosh Perry (MP) has been retained by the 11309455 Canada Inc to prepare this Servicing and Stormwater Management Report in support of the site plan approval for the proposed development at 355 Franktown Road within the Town of Carleton Place.

The main purpose of this report is to demonstrate that the proposed development has access to sufficient public services in accordance with the recommendations and guidelines provided by the Town of Carleton Place (Town), the Mississippi Valley Conservation Authority (MVCA) and the Ministry of the Environment, Conservation and Parks (MECP). This report will address access to water, sanitary and storm servicing for the development, ensuring that existing services will adequately service the proposed development.

1.2 Site Description

The property is located at 355 Franktown Road in the Town of Carleton Place. The site, which is not considered to include the commercial plaza, covers approximately 1.34 ha and is located between the proposed second phase of Coleman Street Subdivision and Franktown Road.

The existing site is currently undeveloped, consisting of wooded and grassed areas. Adjacent lots to the north and south are also undeveloped. Coleman Street Subdivision Phase 2 flanks the eastern portion of the property and existing commercial and residential developments along Franktown Road are located to the west.

The development proposes two 4-Storey condominium buildings on the western portion of the property and six townhouses on the eastern portion of the property. The condominium buildings will be separated from the townhouse blocks by a public ROW. The future ROW will connect the proposed development to the lands to the north and eventually to the Coleman subdivision via the lands to the south.

2.0 BACKGROUND DOCUMENTS

Background documents available under separate cover include:

- JLR Watermain Capacity – Future Development_Final (Dated September 16, 2013 completed by J.L. Richards & Associates Ltd.)
- Functional Servicing Report – 347 Franktown Road (Dated August 13, 2021 completed by Mcintosh Perry Consulting Engineers Ltd.)
- Servicing and Stormwater Management Report – 347 Franktown Road (Date June 22, 2022 completed by Mcintosh Perry Consulting Engineers Ltd.)
- Servicing and Stormwater Management Report – Coleman Central Subdivision Phase 2 (Dated February 12, 2024) Note: This subdivision is currently ongoing approvals. Servicing and references will be updated to reflect the approved documents when complete.

3.0 WATERMAIN

3.1 Existing Watermain

The following subsections outline the existing water infrastructure within Franktown Road and the proposed infrastructure within Coleman Street Subdivision Phase 2.

3.1.1 *Franktown Road*

There is an existing 200 mm diameter watermain that runs north along Franktown Road, ending in a stub located at Findlay Avenue. Just before the stub there is a hydrant that services the existing commercial development adjacent to the subject site.

3.1.2 *Coleman Central Subdivision*

Although not yet constructed, the infrastructure within the proposed Coleman Central Subdivision Phase 2 is anticipated to be constructed prior to the proposed construction of the subject property. There is a proposed 200 mm diameter watermain that services the subdivision. The design of the Coleman Street Subdivision Phase 2 has taken the future development into account with stubs extending westward from the subdivision located both northeast and southeast of the subject site.

3.2 Proposed Watermain

The existing 200 mm watermain within Coleman Central Subdivision Phase 2 will be extended along the future municipal road. In accordance with the Watermain Capacity – Future Development provided by the Town of Carleton Place, a new 200mm watermain is proposed to connect the extended main within the future municipal ROW. A 150mm PVC water lateral will extend from the proposed 200mm watermain to service the condo buildings, as shown on drawing C102. The townhouse block will be serviced via 19mm copper ‘k’ type laterals extending from the 200mm watermain within the future municipal road. A new service will be extended to the existing mall from the proposed 200mm watermain within the site. The proposed watermain will be extended through the site and connect to the existing municipal watermain within Franktown Road.

The Fire Underwriters Survey 2020 (FUS) method was utilized to determine the required fire flow for the proposed Phase 1 development. All buildings in the development were evaluated for the worst-case fire flow scenario. It was determined that the townhouse block is the worst case. Detailed water and fire calculations for the development can be found in Appendix ‘C’ of this report.

The ‘C’ factor (type of construction) for the townhouses was determined to be 1.5 (Wood Frame). The total floor area (‘A’ value) for the FUS calculation was determined to be 1132.0 m^2. The results of the calculations yielded a required fire flow of 11,000 L/min. The detailed calculations for the FUS can be found in Appendix ‘C’.

The water demands have been calculated to adhere to the *Ottawa Design Guidelines – Water Distribution* manual and can be found in Appendix ‘C’. *Table 1*, below, summarizes the design criteria and total calculated demands.

Table 1: Water Supply Design Criteria and Water Demands

Water Demand Rate (Commercial)	28,000 L/gross ha/d
Water Demand Rate (Residential)	280 L/c/day
1-Bedroom Apartment	1.4 Persons/unit
2-Bedroom Apartment	2.1 Persons/unit
Townhouse	2.7 Persons/unit
Residential Peaking Factor (Day)	4.9 x avg. day
Residential Peaking Factor (Hour)	7.4 x max. day
Site Area (ha)	2.07
Average Day Demand (L/s)	0.86
Maximum Daily Demand (L/s)	3.42
Peak Hourly Demand (L/s)	5.27
FUS Fire Flow Requirement (L/s)	183.33
Max Day + Fire Flow (L/s)	186.75

3.3 Hydraulic Water Model Results

With reference to the Watermain Capacity – Future Development Pg. 18, pressures under peak demand were analyzed and a hydraulic water model was completed using Bentley's WaterCAD modelling software based on those conditions. A total of three (3) scenarios were analyzed. The performance of the proposed water distribution system within the development was analyzed under each scenario. The following summarizes the modelling scenarios that were analyzed.

Scenario 1: Average Day Demands (w/ Maximum HGL)

Scenario 2: Peak Hour Demands (w/ Minimum HGL)

Scenario 3: Max Day Plus Fire Flow (w/ Reduced Minimum HGL)

The normal operating pressure range is anticipated to be 449 kPa to 462 kPa and will not be less than 275 kPa (40 psi) or exceed 689 kPa (100 psi). *Table 2*, below, summarizes the resultant water pressures at each junction per scenario.

Table 2: Water Pressure at Junctions per Scenario

Junction	Scenario 1: Average Day Demand (psi)	Scenario 2: Peak Hourly Demand (psi)
J-2	67	67
J-3	68	68
J-4	68	67
J-5	68	68
J-6	68	67
J-7	68	67
J-8	68	68
J-9	68	68
J-10	68	68
J-11	68	68
J-12	68	68
J-13	72	71
J-14	68	67
J-15	68	67
J-16	67	66
J-17	66	65
J-18	64	63
J-19	64	63

To analyze the maximum day demands plus fire flow scenario, the fire flow calculation tool in the water modelling software was used to run multiple iterations of the scenario while gradually increasing fire flows being applied to a single junction until the minimum pressure of 20 psi is reached at any point in the system. A summary of the maximum available fire flow results is provided in Appendix C. Please note the results are considered conservative, as reductions were applied to the HGL at the connection point within Franktown Road.

The water model results determined that the proposed 200mm watermain can adequately provide enough fire flow to meet the required flow rate of 11,000 L/min (183.33 L/sec) at the location of the proposed hydrants H-4 and H-3 (junctions J-15 and J-19), with available fire flows ranging from 13,532 L/min to 11,488 L/min (225.54 L/sec to 191.46 L/sec) while maintaining a minimum residual pressure of 20 psi in the network. Fire flow to the proposed townhouse block will be provided in part by the proposed hydrant within the Coleman Subdivision, given the distance between Hydrant H-4 and the townhouse block will exceed 150m. The water model results determined that the proposed hydrant within the Coleman Subdivision (J10) will provide 15,312 L/min of fire flow, or 255.20 L/s, while maintaining a residual pressure of 20 psi in the network. Refer to the Hydraulic Water Modelling results and figure C1 in Appendix C for more details.

To provide fire flow to the proposed condo buildings internal fire suppression system, a private hydrant (H-4) within 45m of the siamese connection is proposed. A hydrant summary based on the water model can be seen in *Table 3*, below.

Table 3: Fire Protection Confirmation

Building	Max Fire Flow Demand (L/min.)	Fire Hydrant H-3 (L/min.)	Fire Hydrant H-4 (L/min.)	Coleman Subdivision Hydrant (J10) (L/min.)	Combined Fire Flow (L/min.)
Condo Buildings	9,000	13,532.4	11,488	-	>9,000
Townhouse Block	11,000	13,532.4	-	15,312	>11,000

4.0 SANITARY DESIGN

4.1 Existing Sanitary Sewer

Although not yet constructed, Coleman Street Subdivision Phase 2 has a proposed 200 mm diameter sanitary sewer with stubs located to the northeast and southeast of the subject site.

4.2 Proposed Sanitary Sewer

The 200 mm sanitary sewer stub within Coleman Street Subdivision is proposed to be extended along the future municipal road to service the subject property. A 200 mm sanitary sewer is proposed to be extended from the municipal road within the drive aisles bounding the condo buildings. The condo buildings will have shared servicing through a 200 mm sanitary service connection to the proposed 200 mm diameter sanitary sewer. The proposed sewer will also service the existing mall to the west. Each townhouse will be serviced by 135mm sanitary laterals extending from the 200mm sewer within the future municipal road. Refer to drawing C102.

The peak design flow was calculated for the proposed site using the Ottawa Sewer Design Guidelines (SDG). Design criteria used in the sanitary demand calculation can be seen in *Table 4*, below.

Table 4: Sanitary Design Criteria

1-Bedroom Apartment	1.4 persons/unit
2-Bedroom Apartment	2.1 persons/unit

Townhouse	2.7 persons/unit
Average Daily Demand	280 L/day/person
Site Area (Condos, Townhouses, and Existing Mall))	2.07 ha
Residential Peaking Factor	3.52
Commercial	2,800 L/(1000m²/d)
Extraneous Flow Allowance	0.33 L/s/ha

Table 5¸ below, summarizes the estimated wastewater flow from the proposed development. Wastewater flows from the proposed Chadha development are not included in this summary but have been accounted for in sanitary sizing and capacity. Detailed calculations for each contributing area can be found in Appendix 'D'.

Table 5: Summary of Estimated Sanitary Flow

Average Dry Weather Flow	0.97L/s
Peak Dry Weather Flow	2.66 L/s
Peak Wet Weather Flow	3.24 L/s

Based on the calculation provided in the Coleman Central Subdivision Phase 2 Servicing Report and the results shown in *Table 5*, above, it is anticipated that there will be no downstream capacity concerns. Flow from the subject site has been accounted for in the Coleman Central Subdivision design, refer to subdivision design documents for details.

Further to the above, the town has initiated its own analysis to confirm the capacity of the receiving network.

5.0 STORM DESIGN

5.1 Existing Storm Sewer

There is no existing storm infrastructure within the subject property. Stormwater runoff currently sheet drains to the southeast where it is collected by the existing creek. The existing mall adjacent to the site currently outlets to a storm water management area within the development. There is a 975mm concrete storm sewer to be extended from the Coleman Phase 2 subdivision specifically to provide an outlet for 347 and 355 Franktown developments. The 975mm sewer ultimately outlets to an existing ditch that has been realigned as part of the Coleman Central Subdivision Phase 2 development. Please refer to the subdivision documents for details.

5.2 Proposed Storm Sewer

The proposed development will be serviced by a new storm network extended from the future 975mm storm sewer within the future municipal road that will be extended from the existing storm sewer within Coleman Central Subdivision Phase 2. A new outlet to the realigned ditch within the Coleman Central Subdivision Phase 2 pond block is proposed to accommodate flows from the proposed development. As part of the ditch realignment, flows from the subject site have been considered. As existing flows from the adjacent mall currently flow to the site, they will also be considered in the proposed storm water management network and restricted.

Runoff from the condo buildings, drive aisle, rear yard, existing mall and southern landscaped area will be captured and restricted.

Flow attenuation for the above-mentioned areas will be provided via a 180mm plug style orifice located on the upstream invert of the outlet pipe for the ponding area. Flows greater than the allowable release rate will be stored in a landscape area complete with a 2.00m weir at the southeast of the site.

Runoff from the townhouses and the proposed municipal road will sheet drain without attenuation to the future municipal Row.

6.0 STORMWATER MANAGEMENT

6.1 Design Criteria and Methodology

Stormwater management for the proposed site will be maintained through attenuated surface storage provided in a landscape area the southeast of the site. Catch basins will be collect runoff from at-grade areas within the site. The quantitative and qualitative properties of the storm runoff for both the pre & post development flows are further detailed below. The post-development 5 and 100-year flows will be restricted to the pre-development 5 and 100-year flows.

6.2 Runoff Calculations

Runoff calculations presented in this report are derived using the Rational Method, given as:

$$Q = 2.78CIA \text{ (L/s)}$$

Where C = Runoff coefficient

I = Rainfall intensity in mm/hr (City of Ottawa IDF curves)

A = Drainage area in hectares

It is recognized that the Rational Method tends to overestimate runoff rates. As a result, the conservative calculation of runoff ensures that any stormwater management facility sized using this method is anticipated to function as intended.

The following coefficients were used to develop an average C for each area:

Roofs/Concrete/Asphalt	0.90
Gravel	0.60
Undeveloped and Grass	0.20

As per the *City of Ottawa - Sewer Design Guidelines*, the 5-year balanced ‘C’ value must be increased by 25% for a 100-year storm event to a maximum of 1.0.

The time of concentration (Tc) used for pre-development and post-development shall be calculated using a Tc of 10 minutes.

6.3 Pre-Development Drainage

The existing site drainage limits are demonstrated on the Pre-Development Drainage Area Plan. A summary of the Pre-Development Runoff Calculations can be found in *Table 6, below.* Please note the SWM area and Site Area vary slightly as a portion of the townhouse block will be directed to Coleman Phase 2 and accounted for in their stormwater management calculations.

Table 6: Pre- Development Runoff Summary

Drainage Area	Area (ha)	Runoff Coefficient (5-Year)	Runoff Coefficient (100-Year)	5-year Peak Flow (L/s)	100-year Peak Flow (L/s)
A1	1.33	0.20	0.25	77.11	165.18
A2	0.69	0.20	0.25	40.04	85.78
A3	4.47	0.20	0.25	259.20	555.25

See CCO-22-0402 - *PRE* in Appendix ‘E’ and Appendix ‘G’ for calculations.

6.4 Post-Development Drainage

The proposed site drainage limits are demonstrated on the Post-Development Drainage Area Plan. See CCO-22-0402 - *POST* in Appendix ‘F’ of this report for more details. A summary of the Post-Development Runoff Calculations can be found in *Table 7*¸ below.

Table 7: Post Development Flow Rate

Drainage Area	Area (ha)	Runoff Coefficient (5-Year)	Runoff Coefficient (100-Year)	5-year Peak Flow (L/s)	100-year Peak Flow (L/s)
B1	0.28	0.47	0.54	37.77	74.32
B2	0.74	0.64	0.73	137.34	265.21
B3	0.57	0.87	0.97	143.13	272.82
B4	0.32	0.56	0.64	51.37	99.91
B5	4.47	0.20	0.25	259.20	555.25
Total	6.36			628.80	1267.51

See Appendix ‘G’ for calculations. Runoff for area B1-B3 will be restricted before draining to the sewer within the future municipal ROW. The flow will be controlled through the use of a 180mm plug style ICD. Runoff from areas B4 will leave the site unrestricted. Quantity and quality control will be further detailed in Sections 6.5 and 6.7.

6.5 Quantity Control

The total post-development runoff for this site has been restricted to match the 5-year and 100-year pre-development flow rates calculated with a combined C value. Note that areas A3 and B5 are offsite and will outlet to the storm sewer within the future public road at full buildout conditions therefore these areas are not included in the site quantity calculations. These values create the following allowable release rate and storage volumes for the development site.

Table 8: Allowable Release Rate Summary

Drainage Area	Area (ha)	Runoff Coefficient 5-Year	Runoff Coefficient 100-Year	Required Restricted Flow 5-Year (L/s)	Required Restricted Flow 100-Year (L/s)
A1	1.33	0.20	0.25	77.11	165.18
A2	0.69	0.20	0.25	40.04	85.78
Total	2.02			117.15	250.95

See Appendix 'G' for calculations.

Reducing site flows will be achieved using a flow restriction and will create the need for onsite storage. Runoff from area B1 to B3 will be restricted as shown in *Table 9*, below.

Table 9: Post-Development Restricted Runoff Summary

Drainage Area	Post Development Unrestricted Flow (L/s)		Post Development Restricted Flow (L/s)		
	5-Year	100-Year	5-Year	100-Year	
B1	37.77	74.32	55.77	135.10	Restricted – ICD
B2	137.34	265.21			
B3	143.13	272.82			
B4	51.37	99.91	51.37	99.91	Unrestricted
Total	369.60	712.25	107.13	235.01	

See Appendix 'G' for calculations.

Runoff from areas B1 to B3 will be restricted using an ICD within the inlet of DICB5. This will backup stormwater runoff from the site to a landscaped area southeast of the site. The area will pond to elevations of 133.17 and 133.47 for the 5-year and 100-year storms, respectively. The landscaped area will be complete with a 2.00m earth weir.

A storage summary can be seen in *Table 10*, below.

Table 10: Storage Summary

Drainage Area	Storage Required (m^3)	Storage Available (m^3)	Storage Required (m^3)	Storage Available (m^3)
	5-Year		100-Year	
B1	195.9	199.9	331.7	353.7
B2				
B3				

6.6 Hydraulic Grade Line Analysis

The hydraulic grade line was reviewed within the proposed storm sewer network to evaluate the need for sump pumps within the proposed condo buildings and townhouse block. PCSWMM was used to evaluate the HGL based on a 100-year Chicago Storm with a 3-hour duration. The results of the HGL analysis indicated that sump pumps will be required for the townhouse block, as the 100-year HGL elevation will be greater than the USF elevation. Results can be found in *Table 11*, below. Please refer to Appendix 'G' for additional information.

Table 11: Junction HGL vs USF Elevation

Max HGL (m) (MH111)	Max. HGL (m) (MH106)	USF Elev. (m) (Condo Buildings)	USF Elev. (m) (Townhouse Block)
131.85	131.70	132.75	131.36

Additional notes have been added to drawing C102 regarding the requirement for sump pumps and back-flow preventors.

6.7 Quality Control

The development of this lot will employ Best Management Practices (BMP's) wherever possible. The intent of implementing stormwater BMP's is to ensure that water quality and quantity concerns are addressed at all stages of development. BMP's at this site will be implemented at the lot level. Lot level BMP's typically include temporary retention of the parking lot runoff, minimizing ground slopes and maximizing landscaped areas.

A quality treatment unit has been sized to provide a TSS removal rate of 80% as per the Mississippi Valley Conservation Authority (MVCA) requirements. The Oil and Grit Separator (OGS) will provide a water quality of at least 80% TSS. The OGS Unit shall be placed downstream of the restriction unit to provide the required water quality treatment for the site runoff before discharging to the existing creek southeast of the site.

7.0 EROSION AND SEDIMENT CONTROL

7.1 Temporary Measures

Before construction begins, temporary silt fence, straw bale or rock flow check dams will be installed at all natural runoff outlets from the property. It is crucial that these controls be maintained throughout construction and inspection of sediment and erosion control will be facilitated by the Contractor or Contract Administration staff throughout the construction period.

Silt fences will be installed where shown on the final engineering plans, specifically along the downstream property limits. The Contractor, at their discretion or at the instruction of the City, Conservation Authority or the Contract Administrator shall increase the quantity of sediment and erosion controls on-site to ensure that the site is operating as intended and no additional sediment finds its way off site. The rock flow, straw bale & silt fence check dams and barriers shall be inspected weekly and after rainfall events. Care shall be taken to properly remove sediment from the fences and check dams as required. Fibre roll barriers are to be installed at all existing curb inlet catchbasins and filter fabric is to be placed under the grates of all existing catchbasins and manholes along the frontage of the site and any new structures immediately upon installation. The measures for the existing/proposed structures are to be removed only after all areas have been paved. Care shall be taken at the removal stage to ensure that any silt that has accumulated is properly handled and disposed of. Removal of silt fences without prior removal of the sediments shall not be permitted.

Although not anticipated, work through winter months shall be closely monitored for erosion along sloped areas. Should erosion be noted, the Contractor shall be alerted and shall take all necessary steps to rectify the situation. Should the Contractor's efforts fail at remediating the eroded areas, the Contractor shall contact the City and/or Conservation Authority to review the site conditions and determine the appropriate course of action. As the ground begins to thaw, the Contractor shall place silt fencing at all required locations as soon as ground conditions warrant. Please see the *Erosion & Sediment Control Plan* for additional details regarding the temporary measures to be installed and their appropriate OPSD references.

7.2 Permanent Measures

It is expected that the Contractor will promptly ensure that all disturbed areas receive topsoil and seed/sod and that grass be established as soon as possible. Any areas of excess fill shall be removed or levelled as soon as possible and must be located a sufficient distance from any watercourse to ensure that no sediment is washed out into the watercourse. As the vegetation growth within the site provides a key component to the control of sediment for the site, it must be properly maintained once established. Once the construction is complete, it will be up to the landowner to maintain the vegetation and ensure that the vegetation is not overgrown or impeded by foreign objects.

8.0 SUMMARY

- Two new condominium buildings and a block of townhouses are proposed at 355 Franktown Road.
- A new 200mm water main will be extended from the proposed Phase 2 of Coleman Central Subdivision to Franktown Road.
- The FUS method estimated fire flow indicated 11,000 L/min is required for the proposed development.
- Based on boundary conditions provided by the Town, the proposed 200 mm watermain and two private hydrants in the vicinity of the development are capable of meeting daily and fire flow demands.
- A new 200mm sewer main will be installed and connected to the proposed stub at phase 2 of Coleman Central Subdivision

- The development is anticipated to have a peak wet weather flow of 3.24 L/s. A proposed 200 mm diameter sanitary main will collect and outlet flow to the proposed 200 mm diameter sanitary stub located within Phase 2 of the Coleman Central Subdivision. 135mm services will service the block of townhouses, extending from the Phase 2 Coleman sewer. Based on the sanitary analysis conducted in the Coleman Street Subdivision Phase 2 Servicing Report, the subdivisions sanitary network has sufficient capacity for the subject site's flow.
- A new storm system will be installed on-site to capture storm runoff and restrict flows to pre-development rates. The new storm system will discharge future sewer located within Phase 2 of the Coleman Street Subdivision.
- Storage for the 5 and 100-year storm events will be provided via surface storage.

9.0 RECOMMENDATION

Based on the information presented in this report, we recommend that Town of Carleton Place approve this Servicing and Stormwater Management Report in support of the proposed development at 355 Franktown Road.

This report is respectfully being submitted for approval.

Regards,

McIntosh Perry Consulting Engineers Ltd.

Robert Freel, P.Eng.
Senior Project Manager, Land Development
E: r.freel@mcintoshperry.com

Francis J. Valenti, EIT.
Engineering Intern, Land Development
E: f.valenti@mcintoshperry.com

U:\Ottawa\01 Project - Proposals\2022 Jobs\CCO\CCO-22-0402 Heafey_Sub'd_355 Franktown Road, Carleton Place\03 - Servicing\Report\Subm4\CCO-22-0402 - Servicing Report.docx

10.0 STATEMENT OF LIMITATIONS

This report was produced for the exclusive use of the 11309455 Canada Inc group. The purpose of the report is to assess the existing stormwater management system and provide recommendations and designs for the post-construction scenario that are in compliance with the guidelines and standards from the Ministry of the Environment, Parks and Climate Change, Town of Carleton Place and local approval agencies. McIntosh Perry reviewed the site information and background documents listed in Section 2.0 of this report. While the previous data was reviewed by McIntosh Perry and site visits were performed, no field verification/measures of any information were conducted.

Any use of this review by a third party, or any reliance on decisions made based on it, without a reliance report is the responsibility of such third parties. McIntosh Perry accepts no responsibility for damages, if any, suffered by any third party as a result of decisions or actions made based on this review.

The findings, conclusions and/or recommendations of this report are only valid as of the date of this report. No assurance is made regarding any changes in conditions subsequent to this date. If additional information is discovered or becomes available at a future date, McIntosh Perry should be requested to re-evaluate the conclusions presented in this report, and provide amendments, if required.