SERVICING AND STORMWATER MANAGEMENT REPORT
347 FRANKTOWN ROAD

Project No.: CCO-22-0025

Prepared for:

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1.0 PROJECT DESCRIPTION

1.1 Purpose

McIntosh Perry (MP) has been retained by Dr. Neel Chadha to prepare this Servicing and Stormwater Management Report in support of the Draft Plan of Subdivision for the proposed development at 347 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 and proposed services will adequately service the proposed development.

1.2 Site Description

The property is located at 347 Franktown Road in the Town of Carleton Place. The subject land covers approximately 3.0 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 Phase 1 development proposes a retirement home on the northwest portion of the property. A senior’s apartment building is proposed in Phase 2. A medical clinic is proposed in Phase 3. A row of townhouses is proposed in Phase 4. Phases 1-3 will be separated from the Townhouse blocks (Phase 4) by a public ROW. The future ROW will connect the proposed development to the south and ultimately the Coleman subdivision.

Based on consultation with the Town of Carleton Place, separate Development Permit applications will be submitted for each phase of the proposed development. This report will provide a servicing and stormwater management strategy that supports the ultimate development.

2.0 PRE-CONSULTATION SUMMARY

A pre-consultation meeting was conducted with the Town regarding the proposed site on May 21st, 2021. The notes from this meeting can be found in Appendix 'B'. Background documents available under separate cover include:

- JLR Watermain Capacity – Future Development Final (Dated September 16, 2013, completed by J.L. Richards & Associates Ltd.)

3.0 WATERMAIN

3.1 Existing Watermain

The following subsections outline the existing water infrastructure within Franktown Road and 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 south of the subject site. Just before the stub there is a hydrant that services the existing commercial development adjacent to the subject site.

3.1.2 *Coleman Street Subdivision*

Although not yet constructed, the infrastructure within the proposed Coleman Street 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. Servicing for the site is contingent on adjacent developments completion of water construction up to the property line.

3.2 Proposed Watermain

The existing 200 mm watermain within Coleman Street Subdivision Phase 2 will be extended along the future municipal road to service the proposed development. The Phase 1 development will be serviced via a 150 mm water service lateral, as shown by C102. In accordance with the Watermain Capacity – Future Development provided by the Town of Carleton Place, the 200 mm watermain will be connected to the existing 200 mm watermain within Franktown Road. The existing municipal watermain within Franktown Road is proposed to be extended in order to connect with the proposed 200 mm watermain.

The Fire Underwriters Survey 2020 (FUS) method was utilized to estimate the required fire flow for the site. Fire flow requirements were calculated per City of Ottawa Technical Bulletin ISTB-2018-03. Due to the various phases of the development, all phases and buildings were evaluated for the worst-case scenario. It was determined that the proposed Phase 1 building was the worst case. Detailed water and fire calculations can be found in Appendix 'C' of this report.

The 'C' factor (type of construction) for the FUS calculation was determined to be 1 (ordinary construction). The total floor area ('A' value) for the FUS calculation was determined to be 11,691 m^2. The results of the calculations yielded a required fire flow of 13,000 L/min. The detailed calculations for the FUS can be found in Appendix 'C'.

The water demands for the proposed buildings have been calculated to adhere to the *Ottawa Design Guidelines – Water Distribution* manual and can be found in Appendix 'C'. *Table 1* and *Table 2*, below, summarizes the design criteria and calculated demands.

Table 1: Water Supply Design Criteria and Water Demands

Water Demand Rate (Residential)	280 L/c/day
Bachelor/1-Bedroom Apartment	1.4 Persons/unit
2-Bedroom Apartment	2.1 Persons/unit
Residential Peaking Factor (Day)	4.9 x avg. day
Residential Peaking Factor (Hour)	7.4 x max. day
Commercial Rate	28,000 L/ha/day
Commercial Peaking Factor (Day)	1.5 x avg. day
Commercial Peaking Factor (Hour)	1.8 x max. day

Table 2: Summary of Estimated Water Flow – Phase 1-4

	Phase 1	Phase 2	Phase 3	Phase 4
Average Day Demand (L/s)	0.74	0.35	0.04	0.16
Maximum Daily Demand (L/s)	3.50	1.68	0.06	0.78
Peak Hourly Demand (L/s)	5.30	2.54	0.10	1.18
FUS Fire Flow Requirement (L/s)	216.67	166.67	116.67	166.67

With reference to the Watermain Capacity – Future Development Pg. 18, pressures under peak demand were analyzed and a water model was completed using Bentley's WaterCAD based on those conditions. The results determined that the proposed 200 mm watermain can adequately service the proposed development and provide sufficient fire flow since the proposed Hydrant H-1 and H-2 produced available fire flows of 13,174.2 L/min and 14,482.8 L/min. Refer to drawing C101 for Hydrant locations. The results are available in Appendix 'C' of this report.

The normal operating pressure range is anticipated to be 63 psi to 72 psi and will not be less than 275 kPa (40 psi) or exceed 689 kPa (100 psi). The proposed watermain will meet the minimum required 20 psi (140 kPa) at the ground level under maximum day demand and fire flow conditions. *Table 3*, below, summarizes the water pressure at junctions per scenario.

Table 3: Water Pressure at Junctions per Scenario

Junction	Average Day (psi)	Peak Hourly (psi)	Max. Day + Fire Flow (psi)
J-17	66	65	268.42 L/s @ 20 psi
J-21	66	65	241.38 L/s @ 20 psi
J-22	66	65	166.23 L/s @ 20 psi
J-23	66	65	232.34 L/s @ 20 psi
J-24	66	65	218.24 L/s @ 20 psi
J-25	64	63	235.37 L/s @ 20 psi
J-26	66	65	219.57 L/s @ 20 psi
J-27	66	65	218.61 L/s @ 20 psi

In order to provide the required fire flow for the worst case but also for all other cases, two private hydrants have been proposed within the site. The proposed hydrants have been placed to ensure a maximum distance of 45 m to the proposed development. Location details are shown on the Site Servicing Plan included with the report. A hydrant summary can be seen in *Table 4*, below.

Table 4: Fire Protection Confirmation

Building	Fire Flow Demand (L/min.)	Fire Hydrant(s) within 75m	Fire Hydrant(s) within 150m	Combined Fire Flow (L/min.)
347 Franktown Road	13,000	2	2	>18,000

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. Based on coordination with Town staff, this infrastructure needs to be installed to be available for connection.

4.2 Proposed Sanitary Sewer – Ultimate

The proposed 200 mm sanitary sewer stub within the Coleman Street Subdivision is proposed to be extended along the future municipal road, through 355 Franktown Road, to service all four future phases within the subject site. Town staff have noted that updates to the Town infrastructure may be required to support the developments. Based on coordination, an updated analysis is being conducted by the Town.

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 5*, below.

Table 5: Sanitary Design Criteria

Bachelor/1-Bedroom	1.4 persons/unit
2-Bedroom	2.1 persons/unit
Average Daily Demand	280 L/day/person
Residential Peaking Factor	3.51 – 3.65
Commercial Peaking Factor	1.5
Extraneous Flow Allowance	0.33 L/s/ha

Table 6¸ below, summarizes the estimated wastewater flow from the proposed development. Refer to Appendix 'D' for detailed calculations.

Table 6: Summary of Estimated Sanitary Flow – Phase 1-4

	Phase 1	Phase 2	Phase 3	Phase 4	Total
Average Dry Weather Flow	0.76 L/s	0.40 L/s	0.06 L/s	0.18 L/s	1.40 L/s
Peak Dry Weather Flow	2.53 L/s	1.28 L/s	0.08 L/s	0.60 L/s	4.49 L/s
Peak Wet Weather Flow	2.86 L/s	1.60 L/s	0.19 L/s	0.71 L/s	5.36 L/s

Sanitary sewers have been sized to accommodate the full-build out. Refer to sizing sheet and Sanitary Drainage Plan located in Appendix 'D'.

Further downstream of Coleman Street Subdivision Phase 2 a sanitary sewer upgrade is to take place as per *Section 4.3.2* of the *Servicing & Stormwater Management Report – Coleman Central Submission – Phase 2* included in Appendix 'D' for reference. Flows from the subject site were taken into consideration in the report for the full build-out of the development area.

4.3 Proposed Sanitary Sewer – Phase 1

A 200 mm diameter service lateral will be connected from the Phase 1 building to the proposed 200 mm diameter sanitary sewer extension from the Coleman Street subdivision up to the site.

Table 7¸ below, summarizes the estimated wastewater flow from the proposed Phase 1 development. Refer to Appendix 'D' for detailed calculations.

Table 7: Summary of Estimated Sanitary Flow

Average Dry Weather Flow	0.76 L/s
Peak Dry Weather Flow	2.53 L/s
Peak Wet Weather Flow	2.86 L/s

Based on the calculation provided in the Coleman Street Subdivision Phase 2 Servicing Report and the results shown in *Table 7*, above, it is anticipated that there will be no downstream capacity concerns within the Coleman subdivision.

Flow from the subject site has been accounted for in the Coleman Street Subdivision design, as demonstrated by the calculation sheet included in Appendix 'D'.

5.0 STORM DESIGN

5.1 Existing Storm Sewer

There is an existing storm sewer located within Franktown Road.

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 channel, tributary to the Mississippi River.

5.2 Proposed Storm Sewer

The proposed development will be serviced by a new storm network that will outlet to the existing creek located to the southeast. This creek is being regraded in order to accommodate storm flows from Coleman Street Subdivision Phase 2. Flows from the subject site will also be considered. Unrestricted runoff will be directed off site and restricted flow within Phases 1-3 will be stored as required and released to the proposed storm sewer network at the allowable release rate. It is expected that a combination of roof storage, surface storage, and subsurface storage will be required to meet the SWM criteria provided by the Town of Carleton Place. Based on the findings of the hydraulic grade line analysis completed for the downstream storm sewer system, it is expected that sump pumps will be required to service the townhouse blocks. The need for sump pumps will be confirmed through modeling during the detailed design phase.

6.0 STORMWATER MANAGEMENT

6.1 Design Criteria and Methodology

Stormwater management for the proposed site will be maintained through positive drainage away from the buildings and towards the adjacent ROW's. The post-development 5 and 100-year flows will be restricted to the pre-development 5 and 100-year flows. External drainage will be collected and conveyed through the sites without flow attenuation. The quantitative and qualitative properties of the storm runoff for both the pre & post development flows are further detailed below.

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 8, below.*

Table 8: 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	2.73	0.20	0.25	158.19	338.87
A2	0.24	0.20	0.25	14.18	30.38
A3	0.29	0.20	0.25	16.55	35.44
A4	1.33	0.20	0.25	77.30	165.58
A5	0.42	0.20	0.25	24.48	52.43

Area A1 encompasses the site boundary and will be used to determine the allowable release rate for the site. Areas A2 and A3 consist of external drainage collected from the rear yards of 349 and 347 Franktown, respectively. Area A4 represents external drainage collected from northwest of the site, and Area A5 represents external drainage from Franktown Road which currently drains toward the existing outlet.

See CCO-22-0025 – *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-22-0025 – *POST* in Appendix 'F' of this report for more details. A summary of the Post-Development Runoff Calculations can be found in *Table 9*¸ below.

Table 9: 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)
B101	0.27	0.90	1.00	69.76	132.84
B102	0.27	0.65	0.73	51.50	99.43
B103	0.32	0.50	0.57	46.76	91.55
B104	0.17	0.68	0.76	33.91	65.32
B105	0.23	0.81	0.91	54.84	104.84
B106	0.03	0.20	0.25	1.72	3.69
B201	0.36	0.78	0.87	80.64	154.39
B202	0.19	0.90	1.00	48.67	92.68
B301	0.37	0.74	0.83	80.02	153.55
B401	0.32	0.54	0.61	49.74	97.02
B402	0.19	0.70	0.78	38.87	74.79
Total (Site)	2.73	-	-	556.44	1070.10
B501	0.24	0.20	0.25	14.18	30.39
B502	0.29	0.20	0.25	16.54	35.44
B503	1.33	0.20	0.25	77.30	165.58
Total (Site + Collected External Drainage)	4.59	-	-	664.47	1301.51
B504	0.42	0.20	0.25	24.45	52.38
Total (Franktown)	0.42	0.20	0.25	24.45	52.38

See Appendix 'G' for calculations.

Runoff for area B101–B105, B201–B202, and B301 will be restricted before discharging to the existing channel located to the southeast. Runoff is anticipated to be controlled by flow restricted roof drains and inlet control devices.

Runoff from areas B401-B402 will be unrestricted and compensated for in areas with flow attenuation.

External drainage from areas B501–503 & drainage from area B106 will be collected and conveyed to the existing channel without restriction. Runoff from area B504 will be directed towards the existing storm sewer within Franktown Road.

Quantity and quality control will be further detailed in Sections 6.5 and 6.6.

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. (See Appendix 'B' for pre-consultation notes). These values create the following allowable release rate and storage volumes for the development.

Table 10: 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	2.73	0.20	0.25	158.19	338.87

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 B101-B105, B201-B202, and B301 will be restricted as shown in *Table 11*, below.

Table 11: 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	
B101	69.76	132.84	3.84	3.84	Restricted – Roof Drains
B102	51.50	99.43			Restricted – ICD
B103	46.76	91.55	12.66	13.85	Restricted – ICD
B104	33.91	65.32			Restricted - ICD
B105	54.84	104.84	12.66	13.55	Restricted - ICD
B106	1.72	3.69	1.72	3.69	Unrestricted
B201	80.64	154.39	18.62	19.92	Restricted – Roof Drains
B202	48.67	92.68	1.60	1.60	Restricted - ICD
B301	80.02	153.55	18.47	19.77	Restricted - ICD
B401	49.74	97.02	49.74	97.02	Unrestricted
B402	38.87	74.79	38.87	74.79	Unrestricted
Total	556.44	1070.10	158.19	248.03	

See Appendix 'G' for calculations.

Runoff from area B101 will be controlled using flow restricted roof drains before discharging to the proposed storm sewer, downstream of *MH102*. Emergency roof scuppers will be installed to ensure ponding does not exceed the proposed ponding limit.

Runoff from areas B102-B104 will be restricted by an ICD located within the outlet of *MH4*. The restriction of runoff within *MH4* will cause runoff to backup towards the proposed LID SWM storage area northwest of the Phase 1 Building. The SWM area will pond to elevations of 134.16 and 134.47 for the 5-year and 100-year storms, respectively.

Runoff from areas B105 will be restricted by an ICD located within the outlet of *CB101-6*, resulting in shallow surface ponding within the Phase 1 drive aisle and parking lot during the 5- and 100-year events. Should the available surface storage volume determined during detailed design prove insufficient, subsurface storage will be required to restrict area B105 to the allowable release rate. It is expected that subsurface storage, if required, will be provided with underground storage chambers.

External drainage from area B106 will be collected by *DICB101-4* and directed to *MH102,* downstream of the restriction within *MH4.* Runoff from area B106 will be unrestricted and compensated for in areas for in areas with flow attenuation.

Runoff from areas B201 will be restricted by an ICD located within the outlet of *CBMH101-8*, resulting in shallow surface ponding within the Phase 2 drive aisle and parking lot during the 5- and 100-year events. Should the available surface storage volume determined during detailed design prove insufficient, subsurface storage will be required to restrict area B201 to the allowable release rate. It is expected that subsurface storage, if required, will be provided by underground storage chambers or a cistern incorporated into the design of the Phase 2 building.

Runoff from area B202 will be controlled using flow restricted roof drains before discharging to the proposed storm sewer, downstream of *MH103*. Emergency roof scuppers will be installed to ensure ponding does not exceed the proposed ponding limit.

Runoff from areas B301 will be restricted by an ICD located within the outlet of *CB101-13*, resulting in shallow surface ponding within the Phase 3 parking lot during the 5- and 100-year events. Should the available surface storage volume determined during detailed design prove insufficient, subsurface storage will be required to restrict area B301 to the allowable release rate. It is expected that subsurface storage, if required, will be provided with underground storage chambers.

Runoff from areas B401 & B402 will consist of unrestricted runoff from the townhouse blocks and future public road. Runoff will be collected by a series of catch basins and directed to the proposed 675-825 mm diameter storm sewer within the future public road without restriction.

External drainage from area B501 will be collected by *DICB101-4* and directed to *MH102,* downstream of the restriction within *MH4.* The proposed storm sewer network will be sized to accommodate this external drainage area, however runoff from area B501 will not be restricted or counted towards the allowable release rate for the site.

External drainage from area B502 will be collected by *DICB101-1* and directed to *MH102,* downstream of the restriction within *MH4.* The proposed storm sewer network will be sized to accommodate this external drainage area, however runoff from area B502 will not be restricted or counted towards the allowable release rate for the site.

External drainage from area B503 will be collected by DICB101-9 and directed to *MH104* within the future public road. Runoff will be conveyed within the storm sewer network to the discharge point within the Coleman Subdivision.

A storage summary can be seen in *Table 12*, below.

Table 12: Storage Summary

Drainage Area	Storage Required (m^3)	Storage Available (m^3)	Storage Required (m^3)	Storage Available (m^3)
	5-Year		100-Year	
B101	67.08	70.25	150.61	160.56
B102	105.31	105.67	243.86	246.02
B103				
B104				
B105	29.18	*TBD*	73.38	*TBD*
B201	42.90	*TBD*	108.13	*TBD*
B202	54.40	59.40	119.67	124.74
B301	42.58	*TBD*	107.65	*TBD*

6.6 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.

An LID SWM area is proposed within Phase 1, complete with grassed swales along the property boundary. The SWM area and grasses swales will provide an opportunity for infiltration, as well as filtration and sedimentation of suspended solids.

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 *Site Grading, Drainage and Sediment & Erosion 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

- A new retirement home, apartment building, medical clinic, and townhouse block are proposed to be constructed at 347 Franktown Road within the town of Carleton Place.
- A new 200mm watermain will be extended from the proposed Phase 2 of Coleman Subdivision to Franktown Road.
- The FUS method estimated fire flow indicated 13,000 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 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 Subdivision
- The development is anticipated to have a peak wet weather flow of 5.36 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 Coleman Street Subdivision. 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 to the existing creek southeast of the site.
- It is expected that storage for the 5 and 100-year storm events will be provided via roof storage and surface storage. Subsurface storage may be required depending on the grading schemes developed during detailed design.

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 Draft Plan of Subdivision proposal for 347 Franktown Road.

This report is respectfully being submitted for approval.

Regards,

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10.0 STATEMENT OF LIMITATIONS

This report was produced for the exclusive use of Dr. Neel Chadha. 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. Egis reviewed the site information and background documents listed in Section 2.0 of this report. While the previous data was reviewed by Egis 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. Egis 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, Egis should be requested to re-evaluate the conclusions presented in this report, and provide amendments, if required.