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SERVICING FEASIBILITY REPORT
PROPOSED INDUSTRIAL WAREHOUSE DEVELOPMENT
2726 MOODIE DRIVE
CITY OF OTTAWA, ONTARIO

Prepared For:

1000198532 Ontario Inc.
1172 Walkley Road
Ottawa, Ontario
K1V 1P7

PROJECT #: 221099

DISTRIBUTION
City of Ottawa
1000198532 Ontario Inc.
Kollaard Associates

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1 INTRODUCTION

Mr. TJ Sohal of 1000198532 Ontario Inc. retained Kollaard Associates Inc. to complete the civil engineering design including site servicing and stormwater management for the proposed industrial warehouse development at 2726 Moodie Drive in the City of Ottawa, Ontario. The proposed development will occupy a mostly vacant parcel of land along the west side of Moodie Drive at Fallowfield Road. For the purposes of this design, Moodie Drive is considered to be oriented along a north south axis.

This brief will address the serviceability of the proposed industrial warehouse development with respect to the water and sanitary demands, as well as outline the proposed design to meet these requirements. The brief will also provide a summary of the stormwater management design.

The development site is an approximately 6.607 hectares (16.3 acres) and resembles a rectangularly shaped parcel with the northwest corner missing. The development site extends about 390 metres west of Moodie Drive with approximately 240 metres of frontage along the south side Fallowfield Road. The remaining about 150 metres extends behind existing residential development along the south side of Fallowfield Road. The site has approximately 170 metres of frontage along Moodie Drive.

The proposed development will consist of 5 Industrial Warehouse Buildings together with associated parking, roadway and truck turning areas. The proposed warehouses will be oriented parallel to Fallowfield Road with 3 buildings adjacent to Moodie Drive and 2 buildings behind the existing residential lots along Fallowfield Road. Buildings A and B and buildings D and E will be placed back to back to shield the adjacent residential properties from the loading bays and by extension the vast majority of the truck traffic.

The proposed warehouse buildings will consist of the following:

- Building A: 5,158.4 m^2 (includes an office space allotment of 2,380.8 m^2)
- Building B: 6,040.1 m^2 (includes an office space allotment of 2,788.2 m^2)
- Building C: 5,318.3 m^2 (includes an office space allotment of 2,454.6 m^2)
- Building D: 4,797.0 m^2 (includes an office space allotment of 2,380.8 m^2)
- Building E: 5,158.4 m^2 (includes an office space allotment of 2,214.0 m^2)

The proposed buildings will be designed assuming a medium industrial occupancy. There will be no floor grates or drains within the warehouse portion of the buildings.

1.1 Background

The subject property for the proposed development is located along the west side of Moodie Drive south of Fallowfield Road. There is municipal water available along Fallowfield Road. In addition, the site is currently provided with municipal water by means of a 51 mm diameter copper water service extended along Moodie Drive from Fallowfield Road. There are no other municipal services.

The proposed development site is part of the Jock River Watershed. The east side of the site outlets to the Jock River via the roadside ditches along Fallowfield Road and Moodie Drive while the west portion of the site outlets to the Jock River by means of the Monahan Drain. The existing ground surface elevations varies from about 118.1 metres at the crest of the site to about 115 metres at the southwest corner and about 115.5 m along the east side of the site.

The subsurface conditions at the site in general consist of topsoil overlying silty sand glacial till. The normal groundwater level is expected to range from 1 to 2 metres below the existing ground surface.

1.2 Proposed Servicing

The proposed development will be serviced by means of a private onsite sanitary sewer system, and municipal water. The stormwater requirements will be addressed with a stormwater management facility developed for the site which discharges to the roadside ditch at the northeast corner of the site.

2 STORMWATER MANAGEMENT DESIGN

The stormwater management design for the site has been completed under a separate report Stormwater Management Report, Proposed Industrial Warehouse Development, 2726 Moodie Drive, City Of Ottawa, Ontario prepared by Kollaard Associates Inc, File No. 221099.

The stormwater management for the site consists of controlling the post-development release rate from the stormwater originating on the impervious areas of the site such that the total post-development runoff rate from the site during a 100 year design storm is less than the runoff rate during pre-development site conditions for a 2 year design storm. Quality control requirements consist of an enhanced level of treatment for the runoff discharged from the site.

The stormwater runoff generated on site will be managed by means of low impact development techniques for the perimeter areas of the site as well as a stormwater management facility for the remainder of the site. The stormwater management facility will consist of underground storage tanks in combination with hydrodynamic oil grit seperators and a storage swale which will provide storage and treatment for the runoff from the site. Discharge from the stormwater management facility will be directed to the roadside ditch along Moodie Drive.

2.1 Storm Sewer Design

The storm sewer design for this project was completed in two sections; upstream of the inlet control device (ICD) located within catchbasin manhole number 64, and downstream of catchbasin manhole number 64.

Upstream of the ICD the flow rates in each pipe are limited by the release rate which is governed by the head on the ICD. According to calculations within Appendix F of the SWM Report, the maximum release from the ICD during a 100 year storm event is 51 L/s. This value was used to ensure that during all storm events the restriction upstream of the ICD is made only by the ICD and that all storm sewers have sufficient capacity to convey the flow to CBMH-64.

Due to the physical nature of the site, pipe slopes were kept to a minimum to provide proper drainage of the site while minimizing grade raise wherever possible. In keeping with this practice and in the interest of consistency, the main sewer pipes were designed as 375 mm diameter PVC pipes. Under gravity flow conditions and using a minimum slope of 0.25% as per Table 6.1 of the Ottawa Sewer Design Guidelines the calculated capacity is equal to 91.5 L/s. Therefore, since all pipes upstream of the ICD have a minimum slope of at least 0.30%, they are capable of conveying the runoff to the ICD without restriction.

Downstream of the ICD, the storm sewer system was evaluated to ensure that the storm sewer network is capable of conveying the 5-yr storm event to the storage swale without restriction. This ensures that the ICD on the outlet of the storage swale works as intended to control release rates as per the SWM Report. A Storm Sewer Design Sheet is attached as Appendix E of this report. Also attached is a schematic of the pipe network.

As described in the SWM Report, underground storage chambers are provided to allow for the retention of stormwater in order to satisfy discharge criteria. These chambers are to be installed level to allow for infiltration to the underlying soils. To conservatively confirm the sizing of the storm sewer pipes, calculations were undertaken considering that the inflow to these storm chambers is equal to the outflow downstream of the storage chambers. Additionally, the contribution to the storm network from the roof drains was assumed to be the maximum allowable release as controlled by the roof drains. These assumptions ensure that the storm sewer design is adequate to convey runoff to the storage swale without restriction and thus confirms that no surface ponding will occur.

3 SANITARY DESIGN

Sewage discharges will be domestic in type and in compliance with the City of Ottawa Sewer Use By-law. The anticipated maximum daily sanitary flow will be a total of 1.08 L/s when considering all five buildings on site. There are no municipal sanitary services available at the site or in close proximity to the site.

Sewage treatment and disposal will be provided by a Newterra Clear^3 MBR.ST-105 Pre-Engineered MBR Sewage Treatment System. This self-contained system is housed in a single modular enclosure which will be packaged on an epoxy painted carbon steel skid and placed within the landscaped area near the Moodie Drive entrance to the site. This proprietary system will be approved by the Ministry of Environment Conservation and Park for surface discharge.

Discharge from the treatment system will be to the stormwater management facility. Final discharge criteria will be determined by the MECP. A product brochure has been added to this report as Appendix F.

3.1 Sewer Design

Both the storm sewer and sanitary sewer pipes were designed in keeping with the minimum guidelines for burial depth and pipe slope as specified in the City of Ottawa Sewer design guidelines.

3.2 Design Flows

The sanitary sewage flow for the development was calculated based on the Ontario Building Code (O.B.C Table 8.2.1.3B) for the proposed occupancy. The calculations are provided in Table 3.1 on the following page. The office area was obtained from the information provided on the Architectural site plan. For the purposes of the calculations, it was assumed that there would be 10 employees in each unit. It is noted the calculation sheet indicates that the flow demand for the office space is much greater when calculated using the floor space than when based on the estimated number of employees. Since the sanitary flow demand for office space is governed by the greater of the demand based on floor space or number of employees, the exact number of office employees is not critical.

Final architectural drawings were not available at the time of submission. To ensure sufficient capacity for sanitary flow, calculations were taken with the assumption that each unit took full advantage of the office space and mezzanine space allowable. Standard practice is to exclude common areas such as hallways and staircases from office space calculations using Table 8.2.1.3B. Given that the final locations of these items are not determined at this time, a conservative estimate of 20 square metres per unit was deducted from the total office space for the development.

Table 3.1 Sanitary Flow Demand Calculations

	Establishment	*Office Floor*	*Quantity*	*Volume, L*	*Flow*
Building A					
	Office Building				
	a) per employee per 8 hour shift, or		52	75	3,900 L/day
x	b) per each 9.3 m² of floor space	2380.8	256	75	**19,200** L/day
	Warehouse				
x	a) per loading bay, and		11	150	**1,650** L/day
x	b) per water closet		0	950	**0** L/day
BUILDING B					
	Office Building				
	a) per employee per 8 hour shift, or		64	75	4,800 L/day
x	b) per each 9.3 m² of floor space	2788.2	300	75	**22,500** L/day
	Warehouse				
x	a) per loading bay, and		13	150	**1,950** L/day
x	b) per water closet		0	950	**0** L/day
BUILDING C					
	Office Building				
	a) per employee per 8 hour shift, or		64	75	4,800 L/day
x	b) per each 9.3 m² of floor space	2454.6	264	75	**19,800** L/day
	Warehouse				
x	a) per loading bay, and		11	150	**1,650** L/day
x	b) per water closet		0	950	**0** L/day
BUILDING D, E					
	Office Building				
	a) per employee per 8 hour shift, or		168	75	12,600 L/day
x	b) per each 9.3 m² of floor space	4428	477	75	**35,775** L/day
	Warehouse				
x	a) per loading bay, and		20	150	**3,000** L/day
x	b) per water closet		0	950	**0** L/day
ALLOWANCE FOR STAIRS					
	Office Building				
	a) 10 sqare metres on each floor for stairs	1500	162	75	-12,150 L/day

Total Daily Sewage Design Flow =	**93,375 L/day**

A sewage system application to the Ministry of Environment, Conservation and Parks will be prepared and coordinated with Newterra Corporation Inc.

It is noted that the post development release rate from the stormwater management facility during a 25 mm 4 hour Chicago Storm is 28 L/s. As such an average flow rate of 93,375 L/day (1.08 L/s) from the treatment unit into the stormwater management facility is insignificant.

3.3 Sanitary Service Laterals

A maximum daily sewage flow of 93,375 L/day results in a maximum daily flow of 1.08 L/sec and a peak hourly flow of about 1.73 L/s. The Ontario Building Code specifies minimum pipe size and maximum hydraulic loading for sanitary sewer pipe. OBC 7.4.10.8 (2) states "Horizontal sanitary drainage pipe shall be designed to carry no more than 65% of its full capacity." The capacity of a proposed 150 mm diameter PVC sanitary sewer lateral at a minimum slope of 1% is 15.2 L/sec. Since 0.65 x 15.2 = 9.9 L/s is much greater than 1.73 L/sec, service laterals of 150 mm diameter at a minimum 1% slope will be sufficient.

3.4 Sanitary Service Main

A proposed private 200 mm diameter sanitary sewer main will be extended along the front of each building. A 150 mm diameter sanitary sewer lateral will be extended from each unit to the sanitary main. The main will discharge by gravity to a lift station designed as part of the Newterra Treatment System to distribute the sanitary flow to the treatment system.

4 WATERMAIN DESIGN AND WATER DEMAND

The existing water lateral consists of a 51 mm diameter copper service which extends along Moodie Drive to the existing single family dwelling. The dwelling is to be demolished as part of the development. The proposed development will be provided with a 150 mm diameter water service connection to the existing 150 mm watermain along Fallowfield Road. Both services will be connected by means of "T" connections as indicated on the Site Servicing drawing 221099 - SER.

The water demand for the site consists of three parts which include: domestic water consumption, sprinkler flow allowance, and fire flow requirement.

4.1 Fire Flow Requirement

4.2 Boundary Conditions

4.2.1 Request

The water demand due to occupancy (domestic water demand) together with the fire flow requirements were provided to Water Services at the City of Ottawa on June 9, 2023.

The information provided as part of the report was as follows:

Amount of Fire Flow: 216.7 L/s (without firewalls)*
Amount of Fire Flow: 86.7 L/s (with firewalls)*
Sprinkler Demand: 53 L/sec
Average daily water demand: 2.68 L/s
Maximum daily water demand: 4.01 L/s
Maximum Hourly water demand: 7.23 L/s
Peak sanitary flow: N/A – Private Septic System

The fire flow and sprinkler calculations were revised following the request for boundary conditions. It is noted that the domestic water demands provided to Water Services remains unchanged.

4.2.2 Provided

The following are the boundary conditions, HGL, for hydraulic analysis that were provided by Water Services of the City of Ottawa in response to the above indicated peak hourly demand and fire flow demand.

Results:
Connection 1 - Moodie at Fallowfield

Demand Scenario	Head (m)	Pressure[1] (psi)	Flow Rate * (L/s)
Maximum HGL	153.3	51.3	0
Peak Hour	149.0	45.3	7.2
Max Day plus Fire Flow #1	140.5	33.1	91
Max Day plus Fire Flow #2	133.1	22.6	124
Max Day plus Fire Flow #3	N/A	< 0	221

[1] Ground Elevation = 117.2 m

*The column Flow Rate has been added, for simplicity, to the table provided by Water Services and represents the flow from the demand scenario.

It is noted that this table clearly shows that there is insufficient municipal water supply available to meet a flow demand of 221 L/s as the residual pressure at the proposed connection during a flow demand of 221 L/s is less than zero. It is considered that due to the insufficiency of municipal water supply to the site, the development for fire water calculations is considered to be rural.

4.2.3 Calculation Procedure - Fire Water Storage Design Rationale

For the City of Ottawa, water demand requirements for fire protection are governed by Section 4.2.11 of the Ottawa Design Guidelines – Water Distribution. A revision to this section was published in Technical Bulletin IWSTB-204-05 which provides guidance on the parameters which will determine whether fire storage should be calculated using FUS calculations or with calculations according to the Ontario Building Code (OBC).

To provide better protection from fire events, firewalls will be required to subdivide each of the buildings. Final architectural drawings will determine exact separation locations for each building.

As this site does not meet the requirements for FUS Superior Tanker Shuttle, OBC method is recommended to calculate the fire flow. These calculations are discussed further in the next section. The result of those calculations indicates that the required flow rate is less than 9,000 litres per minute. This results in a storage volume requirement equal to the storage volume requirement as per OBC calculations.

4.2.4 Building Construction and Construction Type Consideration

The proposed buildings are steel frame warehouse buildings with accessory office use. The buildings will be constructed with unprotected non-combustible structural systems and non-combustible exterior walls. In accordance with Technical Bulletin ISTB-2018-02, this type of building would fall into the FUS non-combustible construction type.

The warehouse portion of each building will be a single storey with a building height of approximately 7.6 metres. The buildings vary in size and number of units. The buildings may be occupied by one or more tenants. If a single building is occupied by one tenant, there is a potential that there are no dividing walls between units. The accessory office portion will be one storey with the potential for a mezzanine above the office or a second storey office area. If the office portion is two stories it will be constructed with a minimum fire resistance rating of 1 hour on all walls and ceilings. It is understood that there are no plans to provide fire resistance ratings for the steel columns or roof system. Since the structural steel members will not have a fire resistance rating, the buildings will be considered to be ordinary construction.

4.2.5 OBC Fire Water Storage Requirements

The fire flow demand calculation has been included in Appendix A. The building area considered for each building for fire flow calculation purposes was calculated with a maximum building footprint (that is the area bounded by firewalls) of 2,085 square metres which translates to a building volume of 15,846 cubic metres. According to the provided OBC calculations the minimum water supply requirement for firefighting purposes is 270,000 litres and the minimum flow rate is 6,300 litres per minute.

4.3 Sprinkler Flow Allowance

The sprinkler flow allowance is ultimately determined by the Mechanical Engineer during design for building permit purposes. For the purposes of verifying the adequacy of the available water supply and the required building service size, the sprinkler flow allowance has been determined in keeping with NFPS 13 Chapter 19.2.3. Excerpts of the NFPA 13 are included in Appendix B.

From Annex A - A.4.3 of NFPA 13, the proposed industrial warehouse building occupancy classification is Ordinary Hazard Group 2. From table 19.2.3.1.1 the minimum sprinkler water supply is 0.2 gpm /ft^2 using a minimum area of 1500 ft^2 or 8.1mm/min using a minimum area of 140 m^2. As previously indicated, the maximum area bounded by firewalls is 2085 m^2. Assuming that the sprinkler system will be designed to limit the sprinkler discharge to the area affected by a fire, the water demand area will be limited. For the purposes of estimating the sprinkler flow allowance, it was assumed that the sprinkler discharge would be limited to 20 percent of the unit.

The water demand area would be limited to 2085m^2 x 0.2 = 417 m^2

A sprinkler demand of 8.1 mm/min x 417 m^2 = 3378 L/min or 56.3 L/s.

4.4 Domestic Water Demand

The water demand for the proposed development was calculated based on the City of Ottawa Water Distribution Design Guidelines (as amended) as follows:

Industrial

The water demand was calculated as follows:
Industrial Property 35,000 L/ha/day x 6.607 ha = 231.65 m3/day = 2.63 L/sec
Max day = 2.63 L/s x 1.5 = 4.01 L/s
Max hour = 4.01 L/s x 1.8 = 7.23 L/s

4.5 Combined Total Flow Demand

The peak hourly demand is 7.2 L/s

The fire flow demand is 105 L/s
The sprinkler demand is 56.3 L/s

Total Fire Flow Demand + sprinkler demand = 161.3 L/s
Peak hourly + sprinkler demand = 63.5 L/s

Total Water Demand is 168.5 L/s.

Since the available flow at 22.6 psi is 120.0 L/s there is insufficient water supply to meet the total water demand.

4.6 Water Service Requirements and Pressure Loss Calculations

The maximum and minimum pressures were determined at the ground floor level of buildings A and E. These buildings represent the connections closest to the boundary conditions and the furthest away. The pressure losses were first calculated assuming a single 152mm diameter service. This set of calculations is indicated on the Water Pressure Loss Calculation Sheet included in Appendix D by the label single feed. Using the provided HGL at peak flow of 149 m, a flow rate of 7.2 L/s was determined to result in minimum pressures of 299, and 296 kPa for buildings A, and E respectively.

Using the provided HGL of 140.5 m for a flow rate of 87 L/s and linear interpolation, a flow rate of 63.5 L/s was determined to result in a HGL of 143 m.

Calculations were initially completed using a single watermain extended from the proposed connection to the municipal watermain to the mechanical room in each building. The results of the calculations show that the pressure loss along the single watermain, when starting with an HGL of 143 m and a flow rate of 63.5 L/s results in pressures ranging from less than 0 kPa to 127 kPa (18 psi). This indicates that there is insufficient pressure and water supply within the existing municipal watermain to meet the sprinkler flow requirements when using a single connection together with a single 152 mm watermain servicing the proposed buildings.

The calculations were then completed in consideration of looping the proposed watermain around the site in combination with two service connections. This set of calculations is indicated by the Label Looped Feed on the Water Pressure Loss Calculation Sheet. With a looped watermain, an HGL of 143 m and a flow rate of 63.5 L/s resulted in residual pressures of 201 kPa (29 psi), and 153 kPa (22 psi) for buildings A, and E respectively. As such there is sufficient water supply and pressure within the municipal watermain to meet the requirements of the proposed sprinkler system and domestic water demand if a looped private watermain is used.

The pressure loss to the mechanical room of the proposed buildings was calculated using Bernoulli's Equation in Combination with the Darcy – Weisbach Equation and the Colebrook Equation. The equations are shown below.

$$H_P + Z_1 - Z_2 + \frac{P_1 - P_2}{S} + \frac{V_1^2 - V_2^2}{2g} = h_f + h_m \quad \text{where:}$$

$$h_m = K_m \frac{V^2}{2g} \qquad \text{Re} = \frac{VD}{\nu} \qquad Q = VA \qquad A = \frac{\pi}{4} D^2$$

$$\text{Darcy – Weisbach Equation: } h_f = f \frac{L}{D} \frac{V^2}{2g} \quad \text{where:}$$

$$\text{If laminar flow } \left(\text{Re} < 4000 \text{ and any } \frac{e}{D}\right), \quad f = \frac{64}{\text{Re}}$$

$$\text{If turbulent flow } \left(4000 \le \text{Re} \le 10^8 \text{ and } 0 \le \frac{e}{D} < 0.05\right), \text{ then}$$

$$\text{Colebrook Equation: } \frac{1}{\sqrt{f}} = -2.0 \log\left(\frac{e/D}{3.7} + \frac{2.51}{\text{Re}\sqrt{f}}\right)$$

Head loss calculations for the proposed water mains are presented in Appendix D.

Based on the results of the analysis as presented in the appendix, when using 152 mm diameter service mains, the provided minimum and maximum HGL provide water pressures of between 296 KPa and 344 KPa at the proposed warehouse buildings when considering only domestic water demand. The minimum residual pressure, calculated at ground floor level, when there is sprinkler demand would be 153 to 201 kPa which is below the minimum recommended pressure of 276 kPa but above the minimum residual pressure of 141 kPa (20 psi). As such, an internal booster pump will be required to ensure adequate pressure and flow for the sprinkler systems.

Based on the above calculations and in consideration of the proposed building sprinkler demand a looped 152mm diameter service is proposed.

4.7 Existing Fire Hydrants

There are two municipal hydrants in the general area of the site. The first hydrant consists of a Class C fire hydrant (orange painted cap) across Moodie Drive approximately 100 metres east of Building B. The second hydrant consists of a Class AA fire hydrant (blue painted cap) more than 550 metres west of the site entrance to Fallowfield Road. From Technical Bulletin ISTB-2018-02 a Class AA fire hydrant is assumed to provide a maximum capacity of 1000 gpm when between 76 and 152 metres from a building. A Class C fire hydrant will produce somewhat less flow at this distance. A hydrant beyond 305 metres cannot be relied on to produce any flow. Given that the water demand is more than 4,500 gpm, the two municipal hydrants in the area are not considered adequate for the needs of the proposed development and cannot make any significant contribution to the fire flow demand.

In addition, as indicated above, there is not sufficient flow within the existing 150 mm diameter watermain to which the hydrants are connected to meet the fire flow requirements for the proposed development.

4.8 Proposed Firefighting Water Supply and Hydrants

Since there is insufficient pressure and flow within the municipal system to meet the fire water demand for firefighting purposes, onsite fire water storage is proposed.

It is proposed that this volume will be stored in a cast in place concrete storage tank within the landscaped island west of Building A. The fire water storage tank will be filled from the municipal water supply at a controlled rate. An isolation valve will be used to ensure that there is no contamination of the municipal water from the stored fire water. Water will be withdrawn from the storage tank by means of a dry hydrant connected to the storage tank. A remote fire protection – supply main complete with hydrants will be installed independent of the water main used to meet the domestic water and sprinkler requirements for the site. The proposed fire hydrants will be connected to the fire protection supply main in keeping with STD W54. The fire protection – supply main will consist of 200 mm PVC DR18 C900 Class 235 gasketed pressure pipe. Fire hydrants will be connected to the supply main using factory tees and 152 mm diameter leads.

5 CONCLUSIONS

The sanitary demand for the proposed development will be met by a private Newterra Clear3 MBR.ST-105 Pre-Engineered MBR Sewage Treatment System. This system will treat the sanitary sewage such that the effluent can be discharged into the stormwater storage facility. Since the flow rate is greater than 10,000 L/day, and application will be made with MECP for the sewage system.

The proposed buildings will be serviced by a private 200 mm diameter PVC sanitary main which will direct the effluent from each building to a sanitary lift station associated with the Newterra MBR System. Each unit will be provided with a 150 mm diameter sanitary service lateral which connects to the proposed sanitary main.

The domestic water demand and the water demand for the sprinkler system will be met with Municipal water. The municipal water will be distributed throughout the site by means of a 152 mm diameter gasketed PVC DR18 C900 Class 235 private main. The private main will be connected to the existing municipal watermain at 2 locations as the average daily demand for the development exceeds 50 m^3/day. The existing water service also will be connected to the private main.

The water demand for firefighting purposes will be met with underground water storage placed west of Building A. The water for firefighting will be distributed throughout the site by means of a fire protection – supply main (CofO STD W54) which will consist of 200 mm PVC DR18 C900 Class 235 gasketed pressure pipe.

This report provides a summary of the stormwater management design presented under separate cover. Based on the analysis and summary provided under separate cover, the conclusions are as follows:

Stormwater management for the site has been designed to ensure that post-development runoff rate from the site during a 100 year storm event does not exceed the pre-development runoff rate during a 2-year design storm of the same duration. Stormwater storage will be provided on site and released at a controlled rate. Discharge will be directed to the roadside ditch along Moodie Drive. Quality control will be provided by means of sedimentation and vegetative filtration within the stormwater storage swale.

We trust that this report provides sufficient information for your present purposes. If you have any questions concerning this report please do not hesitate to contact our office.

Sincerely,
Kollaard Associates Inc.

LICENSED PROFESSIONAL ENGINEER
3.MAR.2026
S.E. deWit
100079612
PROVINCE OF ONTARIO

Steve deWit, P.Eng.

Appendix A – Fire Flow Demand Calculations Using OBC

APPENDIX B: FIRE FLOW REQUIREMENTS

Client:	**1000198532 Ontario Inc.**
Job No.:	**221099**
Location:	**2725 Moodie Drive, Ottawa, ON**
Date:	**October 24, 2025**

Fire Water Storage and Supply Flow Rate Requirements

The following equation from the latest version of the Ontario Building Code (2024) was used for calculation of the on-site supply rates required to be supplied by the hydrants.

Formulae:

$$Q = KVS_{Tot}$$

$$S_{Tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + S_{side4} + ...]$$

OBC Classification of Building Use	Group, Division	Residential Group C		
Assumed Type of Construction	Combustible with Fire Separations	Buidling is of Combustible construction with fire separations and fire resistance ratings provided in accordance with Subsection 3.2.2 including Loadbearing walls, columns and arches.		
Water Supply Coefficient (Table 1, OBC)	K	17		
Exposure Distance 1		>10	m	
Exposure Distance 2		>10	m	
Exposure Distance 3		>10	m	
Exposure Distance 4		>10	m	
Spatial Coefficient 1	Sside	0		
Spatial Coefficient 2	Sside	0		
Spatial Coefficient 3	Sside	0		
Spatial Coefficient 4	Sside	0		
Total Spatial Coefficient	Stot	1		
Average Building Height	H	7.6	m	
Building Footprint	A	2,085	sq.m	
Total Building Volume	V	15,846	cu.m	
Minimum Supply of Water	Q	269,382	L	
Required Fire Flow	Qf	6300	L/min	*per Table 2 on A-3.2.5.7 of the OBC*
		105	L/s	
		1664	US gpm	

OBC - Table 2 of A-3.2.5.7. REQUIRE MINIMUM WATER SUPPLY FLOW RATE (L/min)		
Qf =	2700	If Q ≤ 108 000 L
Qf =	3600	108 000L < Q ≤ 135 000 L
Qf =	4500	135 000L < Q ≤ 162 000 L
Qf =	5400	162 000L < Q ≤ 190 000 L
Qf =	6300	190 000L < Q ≤ 270 000 L
Qf =	9000	Q > 270 000 L

Appendix B – Sprinkler Flow Calculations per NFPA

Excerpts from NFPA standards

NFPA 13

5.1 General.

5.1.1 Number of Supplies. Every automatic sprinkler system shall have at least one automatic water supply.

5.1.2 Capacity. Water supplies shall be capable of providing the required flow and pressure for the remote design area determined using the requirements and procedures as specified in Chapters 19 through 26 including hose stream allowance where applicable for the required duration.

19.2.3 Water Demand Requirements — Hydraulic Calculation Methods.

19.2.3.1 General.

19.2.3.1.1 The water demand for sprinklers shall be determined only from one of the following, at the discretion of the designer:

(1) For new systems, the density/area selected from Table 19.2.3.1.1 in accordance with the density/area method of 19.2.3.2
(2) For the evaluation or modification of existing systems, the density/area curves of Figure 19.2.3.1.1 in accordance with the density/area method of 19.2.3.2
(3) The room that creates the greatest demand in accordance with the room design method of 19.2.3.3
(4) Special design areas in accordance with 19.2.3.4

Table 19.2.3.1.1 Density/Area

Hazard	Density/Area [gpm/ft^2/ft^2 (mm/min/m^2)]
Light	0.1/1500 or 0.07/3000* (4.1/140 or 2.9/280)
Ordinary Group 1	0.15/1500 or 0.12/3000* (6.1/140 or 4.9/280)
Ordinary Group 2	0.2/1500 or 0.17/3000* (8.1/140 or 6.9/280)
Extra Group 1	0.3/2500 or 0.28/3000* (12.2/230 or 11.4/280)
Extra Group 2	0.4/2500 or 0.38/3000* (16.3/230 or 15.5/280)

*When required by 19.2.3.1.5.

19.2.4 Water Demand.

19.2.4.1* The water demand requirements shall be determined from the following:

(1) Occupancy hazard fire control approach and special design approaches of Chapter 19
(2) Storage design approaches of Chapter 20 through Chapter 25
(3) Special occupancy approaches of Chapter 26

19.2.4.2* The minimum water demand requirements for a sprinkler system shall be determined by adding the hose stream allowance to the water demand for sprinklers.

19.2.5 Water Supplies.

19.2.5.1 The minimum water supply shall be available for the minimum duration specified in Chapter 19.

19.2.6.2* Water allowance for outside hose shall be added to the sprinkler requirement at the connection to the city main or a private fire hydrant, whichever is closer to the system riser.

19.3.3 Water Demand Requirements — Hydraulic Calculation Methods.

19.3.3.1 General.

19.3.3.1.1 The water demand for sprinklers shall be determined only from one of the following, at the discretion of the designer:

(1) Density/area curves of Figure 19.3.3.1.1 in accordance with the density/area method of 19.3.3.2
(2) The room that creates the greatest demand in accordance with the room design method of 19.3.3.3
(3) Special design areas in accordance with 19.3.3.4

19.3.3.1.2 The minimum water supply shall be available for the minimum duration specified in Table 19.3.3.1.2.

19.3.3.1.3 The lower duration values in Table 19.3.3.1.2 shall be permitted where the sprinkler system waterflow alarm device(s) and supervisory device(s) are electrically supervised and such supervision is monitored at an approved, constantly attended location.

Table 19.3.3.1.2 Hose Stream Allowance and Water Supply Duration Requirements for Hydraulically Calculated Systems

	Inside Hose		Total Combined Inside and Outside Hose		
Occupancy	gpm	L/min	gpm	L/min	Duration (minutes)
Light hazard	0, 50, or 100	0, 190, or 380	100	380	30
Ordinary hazard	0, 50, or 100	0, 190, or 380	250	950	60–90
Extra hazard	0, 50, or 100	0, 190, or 380	500	1900	90–120

A.19.3.3.1.5.2(10) The gypsum board (or equivalent material) used as the firestopping will compartment the concealed space and restrict the ability for fire to spread beyond 160 ft^3 (4.5 m^3) zones covering multiple joist channels.

Appendix C – Correspondence With City of Ottawa

- Boundary Conditions

Boundary Conditions
2726 Moodie

Provided Information

Scenario	Demand	
	L/min	L/s
Average Daily Demand	161	2.7
Maximum Daily Demand	241	4.0
Peak Hour	434	7.2
Fire Flow Demand #1	5,220	87.0
Fire Flow Demand #2	7,200	120.0
Fire Flow Demand #3	13,002	216.7

Location

Results

Connection 1 - Moodie at Fallowfield

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	153.3	51.3
Peak Hour	149.0	45.3
Max Day plus Fire Flow #1	140.5	33.1
Max Day plus Fire Flow #2	133.1	22.6
Max Day plus Fire Flow #3	N/A	< 0
[1] Ground Elevation =	117.2	m

Notes

1. BC results represent governing head under existing and future pressurezone reconfiguration.
2. BC results did not consider upsizing existing watermains on Moodie or new connections to the 610 mm backbone watermain.
3. Head loss calculations for new watermains extended from the boundary condition location must be provided.
4. The average day demand for this site exceeds 50 m3/day. As such, two feeds are required to avoid the creation of a vulnerable service area.

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. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.

Appendix D – Water Flow Analysis – Pressure Loss Calculation Sheet

Water Flow Analysis - Pressure Loss Calculation Sheet

Client 1000198532 Ontario Inc.
Location 2725 Moodie Drive, Ottawa, ON
Date October 24, 2025

Kollaard File No. 221099

			Grade Elevation		Hydraulic Grade line		Pressure							
Pipe Sections	Along	End	Start m	End m	Start m	End m	P_{start} kPa	P_{end} kPa	P_{end} psi	Q m^3/sec	V m/sec	D m	Length m	A m^2
Maximum HGL - Daily Flow - 2.7 L/s														
Fallowfield	Main	Building A	117.2	118.2	153.3	153.26	354	344	50	0.0027	0.1488	0.152	315	0.018146
Fallowfield	Main	Building E	117.2	118.2	153.3	153.20	354	343	50	0.0027	0.1488	0.152	525	0.018146
Peak Hour 7.2 L/s														
Fallowfield	Main	Building A	117.2	118.2	149.0	148.74	312	299	43	0.0072	0.3968	0.152	315	0.018146
Fallowfield	Main	Building E	117.2	118.2	149.0	148.40	312	296	43	0.0072	0.3968	0.152	525	0.018146
Sprinkler - 56.3 L/s - Single Feed														
Fallowfield	Main	Building A	117.2	118.2	143.7	131.18	260	127	18	0.0563	3.1026	0.152	315	0.018146
Fallowfield	Main	Building E	117.2	118.2	143.7	117.48	260	-7	-1	0.0563	3.1026	0.152	525	0.018146
Sprinkler + Peak - 63.5 L/s - Single Feed														
Fallowfield	Main	Building A	117.2	118.2	143.0	127.32	253	89	13	0.0635	3.4994	0.152	315	0.018146
Fallowfield	Main	Building E	117.2	118.2	143.0	110.26	253	-78	-11	0.0635	3.4994	0.152	525	0.018146
Sprinkler - 56.3 L/s - Looped Feed														
Fallowfield	Main	Building A	117.2	118.2	143.7	138.96	260	203	30	0.0282	1.5513	0.152	315	0.018146
Fallowfield	Main	Building E	117.2	118.2	143.7	136.16	260	176	26	0.0282	1.5513	0.152	525	0.018146
Sprinkler + Peak - 63.5 L/s - Looped Feed														
Fallowfield	Main	Building A	117.2	118.2	143.0	138.67	253	201	29	0.0318	1.7497	0.152	315	0.018146
Fallowfield	Main	Building E	117.2	118.2	143.0	133.83	253	153	22	0.0318	1.7497	0.152	525	0.018146

Note: Any calculation yielding a residual pressure or Pend of less than zero simply means that there is insufficient water supply at the municipal connection to meet the design flow demand for the senario.

Kollaard Associates
Engineers

Appendix E – Storm Sewer Design

Storm Sewer Design Sheet

Storm Sewer Network Schematic

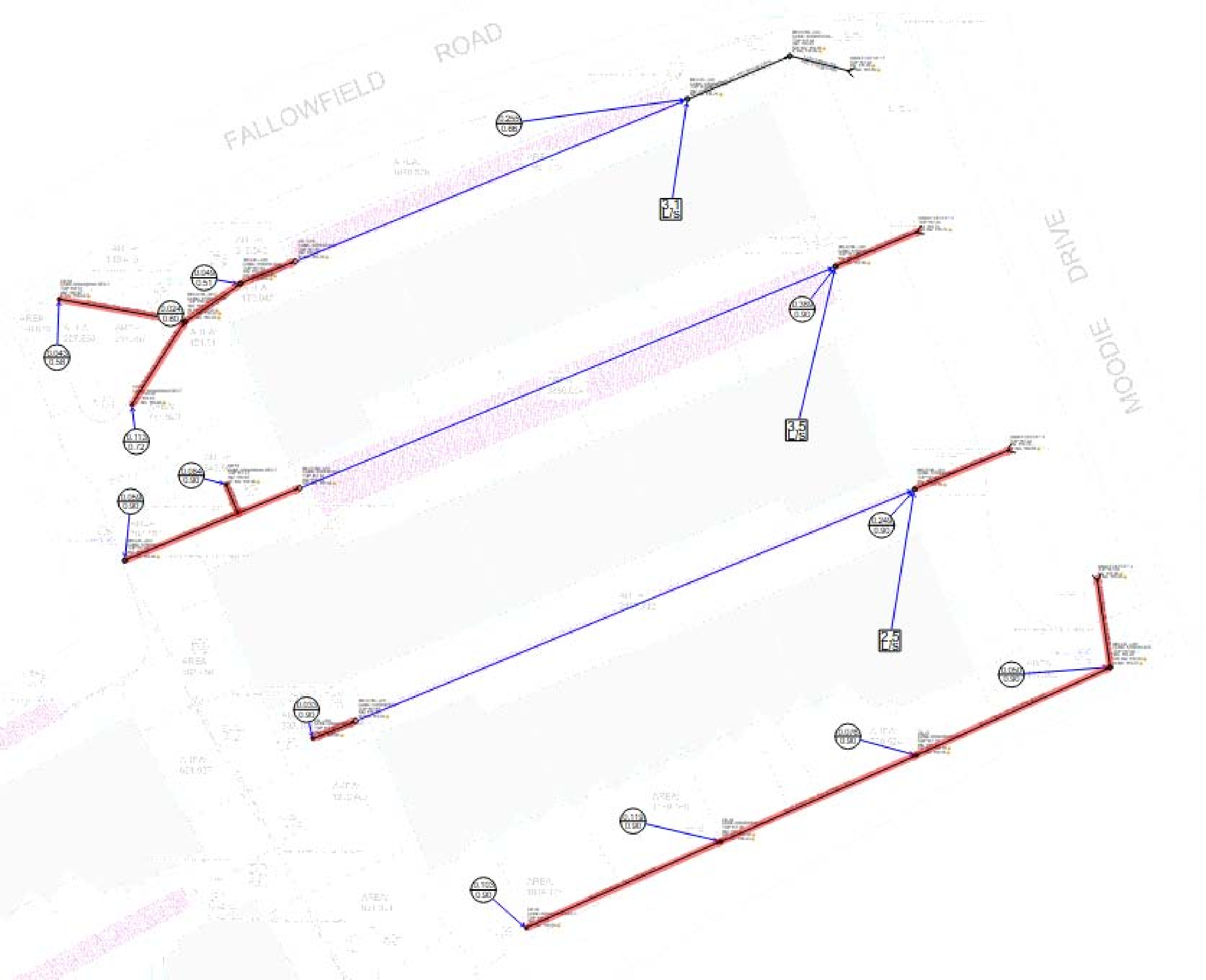
1000198532 Ontario Inc.
Project: 2726 Moodie Drive
File: 221099
Date: March 3, 2026

5-year IDF Data	
A:	998.71
B:	6.053
C:	0.814

IDF Equation: I = A/(t+B)^C
Units for t: minutes

5-year Storm Sewer Design Sheet

Location			Runoff								Constant Flow			Pipe Data						Pipe Results						
Catchment	From	To	Area (ha)	R	AxR	Cum. AxR	Inlet Tc (min)	Cum. Tc (min)	I (mm/hr)	Q (L/s)	Incr. Constant Flow (L/s)	Cum. Constant Flow (L/s)	Total Flow (L/s)	Nominal Size	Length (m)	Invert U/S (m)	Invert D/S (m)	Slope (%)	n	Full Flow Capacity (L/s)	Full Flow Velocity (m/s)	Actual Flow Velocity (m/s)	Time in Sect. (min)	Total Time (min)	Q Total/ Capacity (%)	Notes
SUB(A) 1	CB 60	MH-STM - (61)	0.043	0.58	0.025	0.025	10.00	10.00	104.26	7.2		0.0	**7.2**	375	30.22	116.52	116.42	0.33	0.013	105.2	0.92	0.51	0.98	10.98	**6.9**	
SUB(A) 2	CB 62	MH-STM - (61)	0.113	0.72	0.081	0.081	10.00	10.00	104.26	23.5		0.0	**23.5**	375	23.36	116.49	116.42	0.30	0.013	100.1	0.88	0.72	0.54	10.54	**23.5**	
SUB(A) 3	MH-STM - (61)	MH-CB - (29)	0.024	0.60	0.014	0.121	10.00	10.98	99.36	33.3		0.0	**33.3**	375	15.81	116.39	116.33	0.38	0.013	112.7	0.99	0.86	0.31	11.29	**29.5**	
SUB(A) 4	MH-CB - (29)	CB - (30)	0.049	0.51	0.025	0.146	10.00	11.29	97.92	39.6		0.0	**39.6**	375	14.02	116.30	116.25	0.36	0.013	109.2	0.96	0.87	0.27	11.55	**36.2**	Cumulative AxR is added from CB - (30) to the link between MH-CB - (32) and MH-STM - (33) to account for the underground storage chambers.
ROOF A FLOW		MH-CB - (32)									3.1	3.1														
SUB(A) 5	MH-CB - (32)	MH-STM - (33)	0.255	0.66	0.167	0.313	10.00	11.55	96.71	84.0		3.1	**87.1**	375	26.23	116.21	116.06	0.57	0.013	138.3	1.21	1.28	0.34	11.89	**63.0**	
	MH-STM - (33)	SWALE OUTLET 1	0.000	0.00	0.000	0.313	0.00	11.89	95.21	82.7		3.1	**85.8**	375	14.57	116.03	115.89	0.96	0.013	179.3	1.57	1.55	0.16	12.05	**47.8**	
SUB(AB) 2	CB 15	MH-CB - (64)-MH-STM - (22)	0.084	0.90	0.076	0.076	10.00	10.00	104.26	21.9		0.0	**21.9**	250	7.47	116.00	115.97	0.40	0.013	39.3	0.78	0.79	0.16	10.16	**55.7**	
SUB(AB) 1	MH-CB - (64)	MH-STM - (22)	0.059	0.90	0.053	0.129	10.00	10.16	103.44	37.0		0.0	**37.0**	375	44.70	116.06	115.93	0.29	0.013	98.6	0.87	0.80	0.93	11.09	**37.5**	Cumulative AxR is added from MH-STM - (22) to the link between MH-STM - (26) and SWALE OUTLET 2 to account for the underground storage chambers.
ROOF B FLOW		MH-STM - (26)									3.5	3.5														
SUB(AB) 3	MH-STM - (26)	SWALE OUTLET 2	0.389	0.90	0.350	0.479	10.00	11.09	98.84	131.5		3.5	**135.0**	375	21.26	115.86	115.74	0.56	0.013	137.4	1.21	1.37	0.26	11.35	**98.2**	
SUB(BC) 1	CB - (40)	MH-STM - (41)	0.033	0.90	0.030	0.030	10.00	10.00	104.26	8.6		0.0	**8.6**	375	10.97	115.98	115.94	0.36	0.013	110.5	0.97	0.56	0.33	10.33	**7.8**	Cumulative AxR is added from MH-STM - (41) to the link between MH-STM - (44) and SWALE OUTLET 3 to account for the underground storage chambers.
ROOF C FLOW		MH-STM - (44)									2.5	2.5														
SUB(BC) 2	MH-STM - (44)	SWALE OUTLET 3	0.249	0.90	0.224	0.254	10.00	10.33	102.56	72.3		2.5	**74.8**	375	24.25	115.86	115.79	0.29	0.013	98.3	0.86	0.95	0.43	10.75	**76.1**	
SUB(C) 1	CB 35	CB 36	0.103	0.90	0.093	0.093	10.00	10.00	104.26	26.8		0.0	**26.8**	375	50.19	116.51	116.36	0.30	0.013	100.0	0.88	0.74	1.13	11.13	**26.8**	
SUB(C) 2	CB 36	CB 37	0.119	0.90	0.107	0.200	10.00	11.13	98.64	54.7		0.0	**54.7**	375	50.49	116.33	116.18	0.30	0.013	99.7	0.87	0.89	0.95	12.08	**54.9**	
SUB(C) 3	CB 37	MH-CB - (38)	0.076	0.90	0.068	0.268	10.00	12.08	94.42	70.4		0.0	**70.4**	375	50.31	116.15	116.00	0.30	0.013	99.9	0.88	0.94	0.89	12.97	**70.4**	
SUB(C) 4	MH-CB - (38)	SWALE OUTLET 4	0.050	0.90	0.045	0.313	10.00	12.97	90.82	79.0		0.0	**79.0**	375	20.97	115.97	115.90	0.33	0.013	105.7	0.93	1.01	0.34	13.31	**74.8**	


FALLOWFIELD ROAD
MOODIE DRIVE
3.1 L/s
3.5 L/s
2.5 L/s