JLR No.: 29803-000.1
Revision: 1

December 2, 2022

Prepared for:

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Assessment of Adequacy of Public Services

1927 Maple Grove Road

J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS

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1.0 Introduction

1.1 Background

In 2021, J.L. Richards & Associates Limited (JLR) was retained by Maple Grove Towns Inc. (MGTI) to prepare a Report that would assess the adequacy of public services in support of a Draft Plan of Subdivision Application for their property sited at 1927 Maple Grove Road. An Assessment of Adequacy of Public Services (AAPS) Report was issued in September 2021 and comments were subsequently issued by the City of Ottawa (City) in February and March 2022.

Subsequently, the AAPS Report was re-issued in September 2022 to address comments noted above. Since this second submission, comments were issued on November 14, 2022 by the City. Shortly after the second submission of the AAPS (September 2022), the City of Ottawa issued Technical Bulletin IWSTB-2022-01 which provided new roadway cross-section details for various rights-of-way (ROW). As a result, the September 2022 AAPS Report was revised to incorporate the new cross-sections from the IWSTB-2022-01 and to provide clarifications responses to comments issued on November 14, 2022.

This AAPS Report was also prepared to outline the design objectives and criteria, servicing constraints and high-level strategies for developing the subject lands with water, wastewater, storm and stormwater management services in accordance with the following:

- the November 2009 Servicing Study Guidelines for Development Applications in the City of Ottawa (City)
- the Ottawa Sewer Design Guidelines (2012)
- discussions held during the pre-consultation meeting with City staff
- subsequent discussions on the storm criteria and Email correspondences.

A copy of the original pre-consultation meeting notes (November 2, 2020) is included in Appendix A.

1.2 Site Description and Condition

The subject property is in the Kanata West area, within the urban limits of the City of Ottawa. The subject property, 1927 Maple Grove Road, is located approximately 285m east of Stittsville Main Street. As illustrated on Figure 1 (below), the property is mostly vegetated and includes a single-family house and a garage.

Figure 1: Site Location

MGTI proposes to redevelop the subject property with 38 townhouse units in 6 blocks within a 8920 m^2 (0.892 ha) parcel of land fronting Maple Grove Land. There is also the potential for two (2) additional units, one on either side of the roadway. The Conceptual Plan (Option 2) for the proposed development (prepared by Fotenn) is included in Appendix B. A topographical survey was recently prepared by Annis, O'Sullivan, Vollebekk Ltd., a copy of which is also included in Appendix B.

1.3 Existing Conditions an Infrastructure

A review of existing services was carried out along the frontage of the subject property to identify existing sewers and watermains. Based on the review of the Drawings obtained from the City of Ottawa (Appendix C), the following infrastructure has been identified to exist within both municipal right-of-way (R.O.W.):

Watermains:

- 305 mm diameter PVC watermain along Maple Grove installed in 2013

Sanitary Sewers:

- 375mm diameter sanitary sewer capped across from a pedestrian pathway approximately 80m east of site.

Storm Sewers:

- 2100mm diameter concrete storm sewer, approximately 90 m east of the site
- 375 mm diameter PVC sewer, approximately 90m east of the site.

1.4 Pre-Consultation, Permits and Approvals

A pre-consultation meeting was originally held between the MGTI, Fotenn, the MVCA and the City of Ottawa on November 2, 2020 (Appendix A) to clarify the design criteria and servicing constraints. A meeting was subsequently held on March 30, 2022 to discuss the storm discharge criterion for the subject properties that are to be serviced by the future Maple Grove trunk storm sewer (refer to Section 4.2 for details). The storm discharge criteria used for the preparation of this Report is presented in Section 4.1 (below) and discussed in Section 4.2.

Once the AAPS Report is approved, the development of the above-referenced property will be subject to the Draft Plan of Subdivision approval process with the City of Ottawa. At such time, the City of Ottawa Development Servicing Study Checklist and the preparation of supporting documents for the Application to the Ministry of the Environment, Conservation and Parks (MECP) will be completed for an Environmental Compliance Approval (ECA).

This AAPS and high-level drawings have been prepared in accordance with the following:

Ottawa Sewer Design Guidelines (October 2012) complete with the following Technical Bulletins

- ISTB-2012-01
- ISTDB-2014-01
- ISTDB-2016-01
- ISTDB-2018-01
- ISTDB-2018-04
- ISTDB-2019-01; and
- ISTDB-2019-02

City of Ottawa Water Distribution Guidelines complete with the following Technical Bulletins:

- ISTDB-2010-02
- ISTDB-2014-02
- ISTDB-2018-02; and
- ISTDB-2021-03

In addition, servicing of the 1927 Maple Grove Road property shall be developed in accordance with the following Studies:

- Kanata West Master Servicing Study (KWMSS) prepared by Stantec Consulting Ltd. And CCL/IBI Group, June 2006;
- Servicing Design Brief Poole Creek Village Phase 1 (Kanata West) prepared by IBI Group, 2014
- Design Brief – Pond 4, Kanata West, Mattamy Homes, City of Ottawa prepared by DSEL & JFSA, December 2014

2.0 Water Servicing

2.1 Water Supply

The subject site is located within the urban boundary of the City of Ottawa and will be serviced by the central water distribution system. Water supply to the subject property will originate from Pressure Zone 3W as shown in the 2013 Ottawa Infrastructure Master Plan.

In 2006, the Kanata West Master Servicing Study was completed for that area, which encompasses the subject property. As shown on the Watermain Final Concept Plan (refer to Appendix C), the subject property is to be serviced from the existing Maple Grove 300 mm diameter watermain.

2.2 Water Supply Design Criteria

Any additions to the City of Ottawa water distribution system must be designed in accordance with the Ottawa Design Guidelines (ODG) for Water Distribution (July 2010), and Technical Bulletins ISDTB-2014-02, ISTB-2018-02 and ISTB-2021-03. The proposed system will be designed to satisfy the pressure constraints listed in Table 1 (below) for the peak hour demand, maximum day demand plus fire flow, and maximum hourly demand.

In terms of the required fire flow (RFF), water supply within the municipal right-of-way (ROW) must achieve the guidance of the Fire Underwriters Survey (FUS). Table 1 (below) summarizes the design criteria for water distribution systems, which will serve as basis of the detailed design of the proposed watermain for the site.

Table 1: Water Design Criteria

Design Criteria	Design Value
Population > 500	
Residential average demand	280 L/cap/day
Residential maximum demand	2.5 x Avg
Residential peak hour	2.2 x Max Day
Density Single Family	3.4
Density Semi & townhouse	2.7
Density (apt) 1-bedroom	1.4
Density (apt) 2-bedroom	2.1
Density (apt) 3-bedroom	3.1
Population < 500	
Residential average demand	280 L/cap/day
Peaking Factors	MOE Table 3-3
Fire Flow Requirements	
Municipal ROW	F.U.S.
Pressure/Flow	
Peak hour	>275 kPa (40 psi)
Maximum day plus fire flow	>140 kPa (20 psi)
Minimum hour (maximum HGL)	<552 kPa (80 psi)

Water demands were calculated based on the above-noted design criteria (population less than 500) and have been summarized in Table 2. Appendix C includes the domestic demand calculation sheet.

Table 2: Theoretical Water Demands

Demand Scenario	Water Demand (L/s)
Average Day	0.35
Peak Hour	3.44
Minimum Hour	0.04

It should be noted that the above-noted demands have been calculated based on 40 townhouse units. As previously noted, this total includes 2 potential units which may not occur in the future.

2.3 Fire Flow Requirements

Various guidelines are used throughout North America to establish fire flow requirements for different types of buildings. Along municipal ROW within the City of Ottawa, the required fire flow (RFF) is to be calculated in accordance with the Guidelines entitled "Water Supply for Public Fire Protection (1999)" developed by the Fire Underwriters Survey (FUS) as well as Technical Bulletins listed in Section 2.2.

Given that the 3 m separation between the units is not provided with the new layout (November, 2022), the required fire flow (RFF) calculation was carried out in accordance with ISTB-2018-02 where one fire flow area corresponding to 20 contiguous townhouse units (now 19 contiguous units, per November 2022 layout) yielded a RFF of 20,000 L/min (333 L/s) as shown in the fire flow calculations and exposure sketch included in Appendix D. However, since the Townhouse Blocks have the minimum separation of 10 m between back of the units, the RFF was capped at 10,000 L/min (167 L/s) in accordance with TB-2014-02 (Appendix C).

2.4 Water Servicing and Simulation Results

Functional level servicing shows that the subdivision will be serviced by 200 mm diameter watermain up to the northern end limit of the subdivision. Looping to the future northern watermain is envisioned but will be implemented by others.

Boundary conditions (BC) were requested at the onset based on the domestic demands shown in Table 2 and the RFF of 10,000 L/min. The BC was used to assess headloss under; i) the peak hour demand, ii) maximum day plus fire flow, and iii) maximum hydraulic grade line (HGL) check.

Given that water servicing will consist of a system with a single connection to the existing system, headloss were calculated using the Hazen-Williams desktop calculation method. The operating pressures were assessed as follows (refer to Appendix C for Headloss Calculation Spreadsheet):

- Peak hour and maximum pressure: the headloss were estimated along the proposed ±125 m – 200 mm diameter watermain and was subtracted to the static elevation provided by the City. Static elevations of 156.7 m and 161.0 m for the peak hour and maximum pressure, respectively was provided.

- The maximum day plus fire flow: the headloss was estimated along the proposed ±92.0 m – 200 mm diameter watermain. This length represents the proposed 200 mm diameter watermain from the Maple Grove connection to the northern hydrant within the Site.

Headloss calculations included in Appendix C shows that the pressure and flow constraints under the water demand scenarios listed in Table 1 have been met with the proposed servicing. The pressure under peak hour was estimated at 478 kPa, the pressure under the maximum pressure condition at 520 kPa and the pressure under the maximum day plus fire flow was estimated at 251 kPa.

In regard to hydrant spacing, each townhouse block will be serviced by two (2) hydrants located within the prescribed distance of 75 m in accordance to ISTB-2018-02. Based on this technical Bulletin, these hydrants can provide an aggregate flow of 180 L/s. The exact hydrant locations will be confirmed at detailed design of the subdivision.

2.5 Water Servicing Conclusions

Based on the above calculated headloss, the proposed subdivision can be serviced by a 200 mm diameter watermain supplemented by two (2) hydrants. Headloss calculation under peak hour demand, maximum day plus fire flow and maximum hydraulic grade line (HGL) check showed that the pressure and flow constraints have been met.

3.0 Wastewater Servicing

3.1 Background

The subject property is within the Kanata West serviced area. Wastewater flows from the project site will be captured by a future sanitary sewer which is being currently designed by IBI on behalf of Claridge. This new sanitary sewer designed by IBI will discharge into the existing 375 mm diameter sanitary sewer on Maple Grove Road at Johnwoods Street. The captured flows will eventually outlet to the Kanata West Pumping Station and ultimately conveyed to the Robert O. Pickard Environmental Centre (ROPEC) for treatment.

The proposed sanitary sewer for 1927 Maple Grove Road was conceptually sized based on the City of Ottawa Sewer Design Guidelines ((OSDG) - (October 2012)) and associated Technical Bulletins. Key design parameters have been summarized in Table 3.

Table 3: Wastewater Servicing Design Criteria

Design Criteria	Design Value	Reference
Residential average flow	280 L per capita/day	ISTB-2018-01
Residential peaking factor	Harmon Formula x 0.8	City Section 4.4.1
Infiltration Allowance 0.05 L/s/ha (dry I/I) 0.28 L/s/ha (wet I/I)	0.33 L/s/ha	ISTB-2018-01
Minimum velocity	0.6 m/s	OSDG Section 6.1.2.2
Maximum velocity	3.0 m/s	OSDG Section 6.1.2.2
Manning Roughness Coefficient	0.013	OSDG Section 6.1.8.2
Minimum allowable slopes	Varies	OSDG Table 6.2, Section 6.1.2.2

3.2 Theoretical Sanitary Peak Flow

Wastewater flows conveyed to the local sanitary sewer were estimated based on the proposed density for townhouse units which is 2.7 person per unit and the theoretical unit flow of 280 L/capita/day. Based on this design criteria, a total combined peak wastewater flow of ±1.6 L/s was calculated. Table 4 summarizes the theoretical peak flows for the project site.

Table 4: Theoretical Peak Wastewater Flow

Design Criteria	Flow (L/s)
Theoretical Population: 108 (40 units)	
Theoretical Average Day Flow (Dry Weather)	0.33
Peaking Factor: 3.60 (Harmon)	
Peak Wastewater Flow (Dry Weather)	1.13
Dry & Wet I/I (0.33 L/s/ha 0.891 ha)	0.29
Total Theoretical Peak Flow	**1.55**

3.3 Proposed Sanitary Sewer Sizing

The wastewater analysis described in Section 3.2 shows that the proposed sanitary sewers must be sized to accommodate the peak wastewater flow of 1.55 L/s. An allocation for this site was provided as part of the KWMSS and confirmed as part of the Reconstruction of Maple Grove Road project. The sanitary sewer design sheet provided for the Maple Grove Road project (Appendix 'D') shows that the subject property was accounted for as part of a larger 20.03 ha. parcel with a population of 2044. The design basis was based on the previous OSDG of 350 L/p/day. Given the updated design parameters of 280 L/p/day prescribed in ISTB 2018-03 from the previous 350 L/cap/day, and the projected population of 93 people the sanitary sewer system on Maple Grove Road is expected to have adequate capacity to accommodate the flows generated from the subject site.

3.4 Wastewater Servicing Conclusions

The Project Site is tributary to the existing 375 mm diameter sanitary sewer located along Maple Grove Road. The theoretical peak wastewater flow of ±1.6 L/s was calculated based on the design criteria described in the Ottawa Sewer Design Guidelines and associated Technical Bulletins as summarized Table 4. The peak flow is consistent with the allocations previously made as part of the Maple Grove Road reconstruction.

4.0 Storm Servicing and Stormwater Management

4.1 Existing and Proposed Storm Servicing

Runoff generated from the subject site will be collected by proposed storm sewers that will outlet to a future trunk storm sewer on Maple Grove Road. This sewer located along the frontage of 1927 Maple Grove Road, will be designed by IBI on behalf of Claridge. The sewer will span from the 1981 Maple Grove Road property and will connect to the existing 2100 mm diameter storm sewer further east at the Johnwoods Street intersection. From this intersection, the captured runoff is conveyed along the existing Maple Grove Road trunk storm sewer to the Carp River, via Pond 4 or, via a diversion trunk storm sewer outletting to Poole Creek.

4.2 Synopsis of Studies

Storm servicing for the 1927 Maple Grove Road property is to be developed in accordance with a number of studies. A brief overview of relevant Studies follows:

Kanata West Master Servicing Study (KWMSS) (Stantec/IBI, 2006)

The KWMSS was prepared on behalf of the Kanata West Owner's Group to evaluate and investigate servicing requirements for a large mixed-use community (±725 ha) in Kanata West. In terms of storm and stormwater management servicing, the KWMSS recommended that the study area be serviced by seven (7) water quality/quantity stormwater management facilities located over the Kanata West Study Area.

Pond 4 was identified as the dedicated stormwater management facility to serve 267.97 ha of development, including the 1927 Maple Grove Road property. The KWMSS has identified the Maple grove trunk storm sewer as being the dedicated storm sewer servicing the subject property. Based on Drawing ST-PS of the KWMSS (refer to Appendix D), the 1927 Maple Grove Road property is included in Drainage Area A-1 with a Runoff Coefficient of C=0.6. As such, storm runoff from Area A-1 is to be conveyed to Pond 4 via the Maple Grove trunk storm sewer.

Servicing Design Brief Poole Creek Village (Phase 1), IBI Group, 2014.

As part of this Study, IBI and evaluated storm servicing needs for the area located west of Warmstone Drive while acknowledging the capacity constraints of the existing Maple Grove Road storm sewer and the need to provide a 1:5-year design capture. The area under review included the Fairwinds West, Poole Creek Village (Tartan), Bryanston Gate as well as the 1927 Maple Grove and 1981 Maple Grove properties. As part of the dual drainage work, IBI recommended that flows up to 85 L/s/ha be conveyed along Maple Grove Road to Pond 4 and that flows in excess of 85 L/s/ha be routed unattenuated to Pool Creek via the 2250 mm/2400 mm diameter diversion trunk storm sewer along Santolina Street/Warmstone Drive.

Design Brief for Pond 4 Kanata West

In 2014, a Design Brief was prepared by DSEL/JFSA supporting the design of Pond 4 while accounting for the diversion sewer that conveys the infrequent flows to Poole Creek (Appendix E for excerpts). As shown in Figure 3 of the Design Brief, Area A-1 was simulated with a 1:100-year capture without the need of on-site storage. The Design Brief consolidated previous dual drainage modelling that included the Maple grove trunk storm sewer, the detailed design information for Poole Creek Village, the Maple Grove Road trunk sewer overflow to Poole Creek, the Fairwinds West development, and Pond 4 detailed design information.

Coordination Meeting

On March 30, 2022, a meeting was held to discuss the storm discharge criterion for the subject properties that will be serviced by the future Maple Grove trunk storm sewer. The parties attending the meeting were the City of Ottawa, staff from IBI and JFSA and members from MGTI and JLR. At the meeting, IBI provided a brief overview of the operation of the storm sewer system. The following summary was provided at the meeting and subsequently confirmed (refer to Appendix F for Email):

- Frequent flows (up to 85 L/s/ha) from the area tributary to the future trunk storm sewer will be conveyed to Pond 4 for water quality and quantity control prior to its discharge into the Carp River. The control of 85 L/s/ha is achieved by an existing 850 mm diameter restrictor located at MH103 (per JFSA's Figure 3).

- During larger events, minor system flows in the Maple Grove trunk sewer exceeding 85 L/s/ha up to the 1:100-year peak flow will be conveyed along the Salina/Warmstone Trunk Overflow (2250 mm – 2400 mm) to Poole Creek unattenuated. Prior in discharging into Poole Creek, quality control is provided by means of an oil/grit separator (OGS).

Based on the discussions held at the meeting, the minor system flows from both the 1927 Maple Grove and 1981 Maple Grove properties is to be controlled on-site to the 1:100-year modelled flows in accordance with JFSA's hydrologic/hydraulic model. Shortly thereafter, JFSA provided the design criterion of the minor system flow for the areas serviced by the future trunk storm sewer system via email. This Email dated April 6, 2022, and included in Appendix F, provide the following unit flow rate for each of the design storms.

- The 100-year SCS 24Hr peak flow equivalent to 210.9 L/s/ha
- The 100-Year CHI 3Hr peak flow is equivalent to 241.1 L/s/ha

The above unit flow rates were based on the modelled area of 16.78 ha at an average imperviousness of 57% which is equivalent to a Runoff Coefficient of 0.60. The stormwater management calculations carried out as part of this AAPS Report were completed based on the lesser of the unit flow rate (i.e., 210.9 L/s), based on an area of 0.8916 ha.

4.3 Design Criteria

The functional servicing presented in this Report was developed based on the requirements specified in the KWMSS and the latest direction provided by JFSA in terms of minor system allowance (see Section 4.2) in the future Maple Grove trunk storm sewer. The storm servicing strategy for the 1927 Maple Grove property is as follows:

- The storm sewer system within the 1927 Maple Grove property to be sized based on the minimum 1:2-year storm capture based on a Tc of 10 minutes and a Runoff Coefficient (C) reflecting the proposed development, being 0.65 for the proposed product.

- Given that the storm discharge criterion provided by JFSA, the minor system flow allowance is substantially greater than the minimum 1:2-year capture from the OSDG.

Thus, sizing of the on-site storm sewers and corresponding release rate to the future Maple Grove storm sewer system must meet the unit flow rate of 210.9 L/s/ha and/or 241.1 L/s/ha under the 1:100-year SCS 24-hour and the 1:100-year Chicago 3-hour storm, respectively. Given that desktop calculations have been used to establish on-site storage volume requirements, the lesser of the unit flow allowance (i.e., 210.9 L/s/ha) was used to establish the maximum allowable peak flow for the subject property.

- Given that the assumed proposed C-Factor for this development (0.65) exceeds the assumed C-Factor of 0.60 by JFSA, onsite storage is to be provided by means of roadway sags to detain flows exceeding the 1:100-year peak flow (210.9 L/s/ha) prior to discharge in the future Maple Grove Trunk storm sewer. No Major overland flow shall leave the site during the 1:100-year storm unless subtracted from the allowable peak flow. Once the Draft Plan and building product is finalized, a detailed C-Factor calculation will be carried out at detailed design.

- Water quality control for the will be provided by Pond 4 based on lands having a C-Factor of 0.60. If the C-Factor is found to exceed 0.60, a review of water quality compliance will be carried out at detailed design.

4.4 Storm Servicing

The general storm and stormwater servicing constraints used for this site are listed in Table 5 below.

Table 5: Storm Servicing Criteria

General Design Criteria
The calculated peak flows were estimated with the Rational Method and the City of Ottawa Intensity-Duration-Frequency (IDF) curves.
The allowable peak flow calculated based on the lesser of the unit flow rate (210.9 L/s) established by JFSA.
Flows in excess of the allowable peak flow to be detained on-site by means of roadway sags
Peak flows estimated based on an inlet time of ten (10) minutes, as per the Technical Bulletin ISDTB-2012-4.
The C-Factor to be calculated based on 0.90 for all hard surfaces and 0.20 for all landscaped areas.
No major overland flow to leave site during the 1:100-year event, unless deducted from the allowable.
Provide measures to ensure that site preparation and construction is in accordance with the current Best Management Practices for Erosion and Sediment Control.

4.5 Proposed Stormwater Management Strategy

4.5.1 Minor system and Allowable Peak Flow

The overall minor system allowance was calculated at 188.04 L/s (Appendix F) based on the 1:100-year unit flow capture rate of 210.9 L/s/ha. Based on the design criterion by JFSA, the 1:100-year minor system flows from the subject property should be limited to 188.04 L/s prior to discharge in the proposed Maple Grove storm sewer system.

Split lot drainage will be implemented for the townhouse blocks. A sawtooth design will be utilized in rear yards, with landscape catch basins and perforated pipe system per City of Ottawa Standard Detail S29. Similarly, the roadway system will be designed using a sawtooth profile such to integrate two (2) roadway sags. The 1:100-year flow allowance of 188.04 L/s will be captured within the rear yard sewer system or cascade overland to the ROW while maintaining adequate freeboard to the openings in the units. Catch basins introduced at roadway sags will be sized to capture minor system flows from the front and rear yards and will be limited with the rear yard CBs to 188.04 L/s. Flows exceeding 188.04 L/s will be detained in the proposed roadway sags. The functional level servicing is depicted in drawing CS1.

4.5.2 Major system

Road sags were introduced within the ROW to collect and direct runoff towards catch basins. Surface storage will be achieved using the street sags and inlet control devices (ICDs) in the catch basin. The ICDs and minor system pipes will be sized to ensure that the captured flows do not exceed the allowable peak flow of 188.04 L/s while excess flows up to the 1:100-year will be accommodated by roadway sags. The emergency overland spill will be directed towards Maple Grove Road. Functional level is presented in Drawing CG1.

Storage Requirements
The storage volume requirement for the project site was calculated based on the Modified Rational Method (MRM) assuming a conservative C-Factor of 0.65 following the guidance of the Studies listed in Section 4.2.

The desktop calculations were carried out using the allowable peak flow of 188.04 L/s and based on a C-Factor of 0.8125, which reflects a 25% increase over the C-Factor of 0.65. As per the MRM calculations, various critical time steps ranging from 10 minutes to 45 minutes were used to establish the critical storage volume requirements. Based on this calculation, a 1:100-year volume of 103 m^3 was calculated (refer to Appendix F for details).

To achieve the on-site storage of 103 m^3, two (2) roadway sags are proposed as depicted in Drawing CG1. Based on a single sag with 0.34 m depth, an overall storage volume for a single sag was calculated at ±65 m^3 using the design sheet developed by JFSA on behalf of the City of Ottawa. (Appendix F). Thus, the two (2) roadway sags shown on Drawing CG1 using the same configuration would, therefore, exceed the above-noted storage volume of 103 m^3. At detailed design, storage volume will be assessed for both roadway sags using Civil 3D and the longitudinal slope and static depth will be revised accordingly to meet the storage volume requirement. The storage sag volume using Civil 3D is generally greater than the spreadsheet approach.

4.5.3 Infiltration

4.5.3.1 Infiltration Requirements

Infiltration requirements within the urbanized area in the Carp River was initially reviewed as part of the Carp River Sub-Watershed Study (2004). This document recommended that infiltration of the landscaped areas be increased by 25% over the existing condition to compensate for the hard surfaces such as roadways.

The more recent document entitled “Kanata West Master Servicing Study (2006)”, referred to in this Report as the KWMSS, provided guidance with respect to infiltration, which also reflects the recommendation made as part of the Carp River Sub-watershed Study (2004). Figure 5.4 and Section 5.6 of the KWMSS (refer to Appendix G) sets the requirements for infiltration for the subject site as follows:

- An infiltration of 70 mm to 100 mm per year should be targeted for the project site as per Figure 5.4. Thus, an average infiltration of 85 mm/year should be targeted under post-development conditions. Per the footnote on Figure 5.4, the target of 85 mm/year is inclusive of the 25% increase based on existing soil types and recharge potential within the project site.; and
- Section 5.6 reads as follows “the infiltration target of 85 mm/year has been established to compensate for those areas that cannot provide infiltration (i.e., Roadway Corridors)”.

4.5.3.2 Infiltration Assessment

As discussed during a meeting held with the City on October 26, 2022, infiltration capacity of the site and water balance will be reviewed at detailed design once the Draft Plan and building product have been finalized. It should be noted that the Geotechnical Report (EXP, September 2022) has noted depth of inferred bedrock to range between 1.6 m to 2.7 m throughout the Site (refer to Table IV of the EXP Report, under separate cover). In addition, groundwater levels were reported to be approximately 2 m below existing ground as shown in Figures 3 to 9 of EXP’s Report (refer to EXP’s Appendix). Recently, the Owner has expanded EXP’s scope of work to include the following:

- Carry out percolation rate tests to assess the infiltration capability of the native soils within the property, and
- Carry out manual groundwater level measurements.

Upon the completion of the above, EXP’s hydrogeologist will review the data, along with the Geotechnical Report. The review of this data will enable the hydrogeologist to assess the site’s specificity and constraints under existing condition and to review the post-development condition in light of the KWMSS recommendations (Section 5.6 and Figure 5.4).

5.0 Erosion and Sedimentation Control

Erosion and sedimentation control measures, as outlined in the Ontario Ministry of Natural Resources (MNR) Guidelines on Erosion and Sediment Control for Urban Construction Sites, will be implemented to trap sediment on site. At a minimum, the following erosion and sedimentation control measures could be implemented during construction (refer to Drawing CS1):

- Supply and installation of a silt fence barrier, as per OPSD 219.110, if required;
- Supply and installation of filter fabric between the frame and cover of catch basins and maintenance holes adjacent to the project area during construction, to prevent sediment from entering the sewer system. The filter fabric is to be inspected regularly and corrected as required;
- Stockpiling of material during construction is to be located offsite;

The proposed erosion control measures shall conform to the following documents:

- "Guidelines on Erosion and Sediment Control for Urban Construction Sites" published by Ontario Ministries of Natural Resources, Environment, Municipal Affairs, and Transportation & Communication, Association of Construction Authorities of Ontario and Urban Development Institute, Ontario, May 1987.
- "MTO Drainage Manual", Chapter F: "Erosion of Materials and Sediment Control", Ministry of Transportation & Communications, 1985.
- "Erosion and Sediment Control" Training Manual by Ministry of Environment, Spring 1998.
- Applicable Regulations and Guidelines of the Ministry of Natural Resources.

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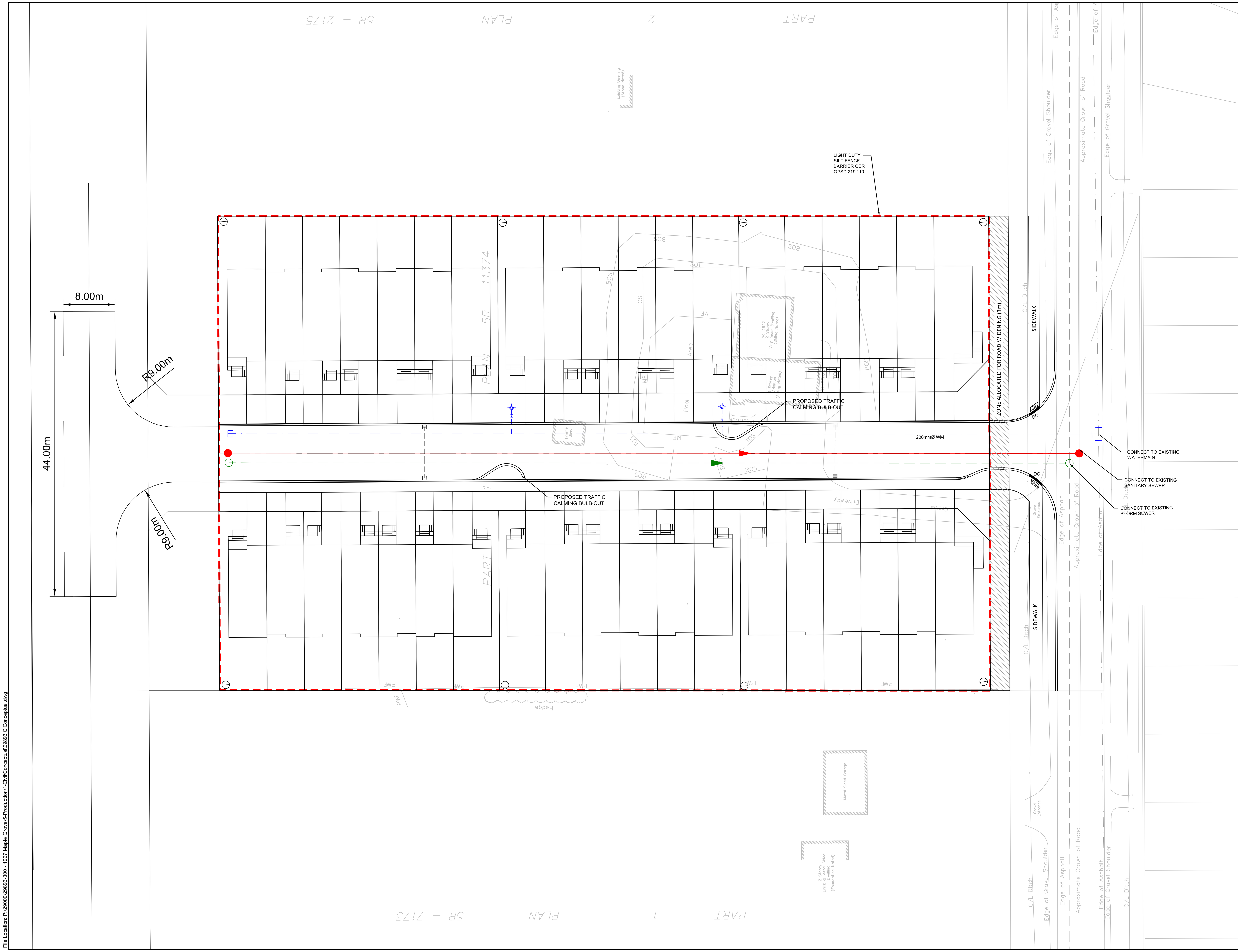

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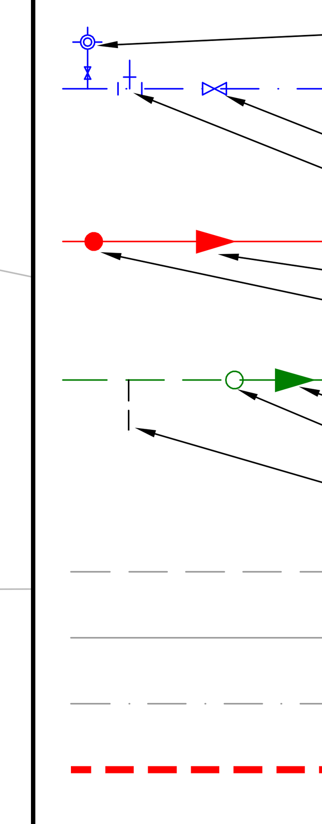

No.	ISSUE / REVISION	DD/MM/YY
04	ISSUED FOR CITY REVIEW	02/12/22
03	ISSUED FOR CITY REVIEW	23/09/22
02	ISSUED TO LANDSCAPE ARCHITECT	01/09/22
01	ISSUED FOR CITY REVIEW	07/09/21

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SCALE: 1:250

CLIENT:

CONSULTANT: www.jlrichards.ca

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CONSULTANT:

PROFESSIONAL STAMP

PROFESSIONAL STAMP

PROJECT:

MAPLE GROVE TOWNS INC.

1927 MAPLE GROVE ROAD, OTTAWA

DRAWING:

CONCEPTUAL SITE SERVICING PLAN

DESIGN: AT
DRAWN: TB
CHECKED: KF
JLR #: 29893

DRAWING #: CS1

PLOT DATE: December 2, 2022 12:37:31 PM

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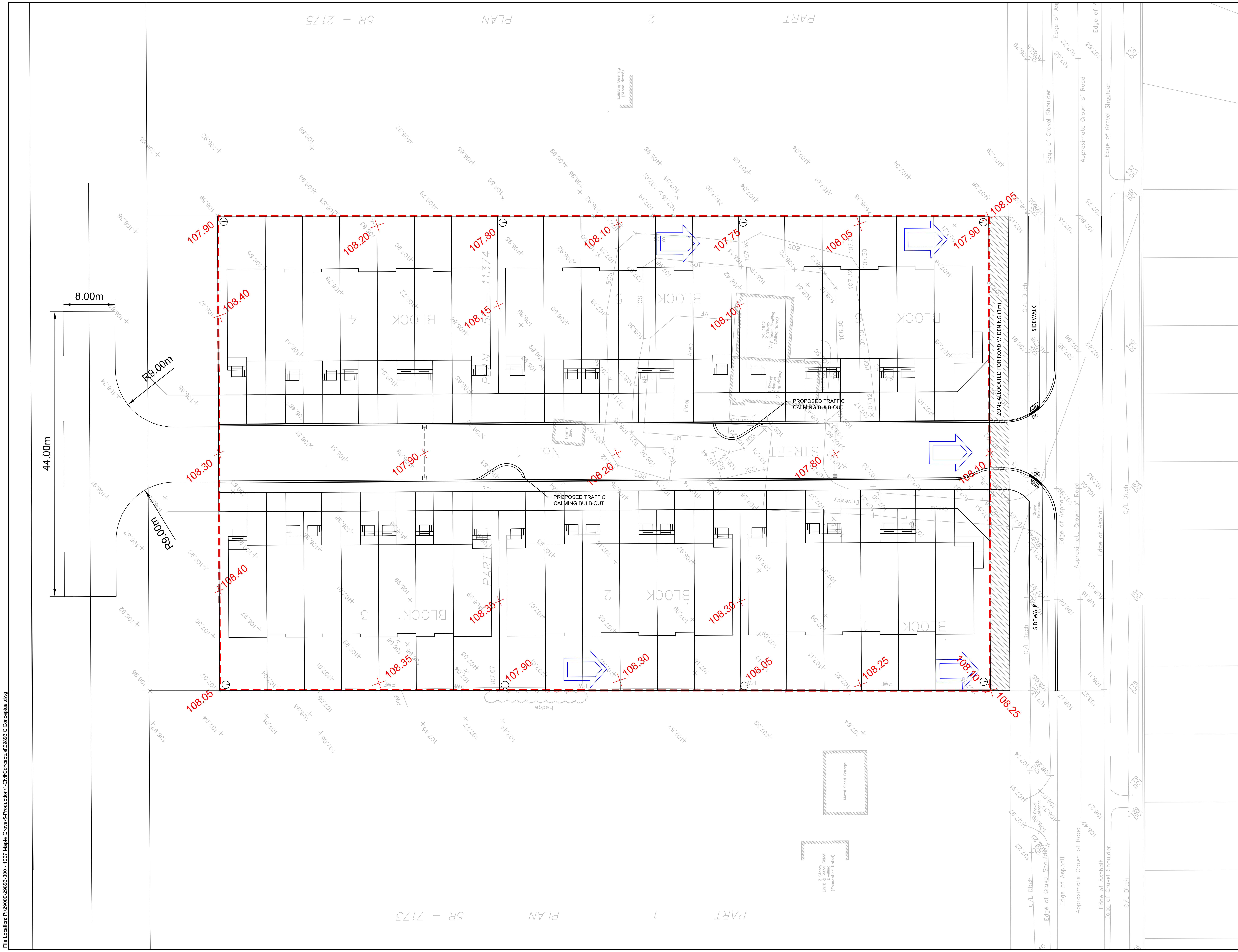


LEGEND

107.80 CONCEPTUAL ELEVATION

108.10 NYLOPLAST DRAINS AND ELEVATIONS

EMERGENCY OVERLAND FLOW DIRECTION

LIGHT DUTY SILT FENCE BARRIER AS PER OPSD 219.110

No.	ISSUE / REVISION	DD/MM/YY
04	ISSUED FOR CITY REVIEW	02/12/22
03	ISSUED FOR CITY REVIEW	23/09/22
02	ISSUED TO LANDSCAPE ARCHITECT	01/09/22
01	ISSUED FOR CITY REVIEW	07/09/21

VERIFY SHEET SIZE AND SCALES. THE BAR TO THE RIGHT IS 25mm IF THIS IS A FULL SIZE DRAWING. 0 25mm

SCALE: 1:250

CLIENT:

CONSULTANT: www.jlrichards.ca

J.L.Richards ENGINEERS · ARCHITECTS · PLANNERS

CONSULTANT:

PROFESSIONAL STAMP

PROFESSIONAL STAMP

PROJECT:

MAPLE GROVE TOWNS INC.

1927 MAPLE GROVE ROAD, OTTAWA

DRAWING:

CONCEPTUAL SITE GRADING PLAN

DESIGN: AT
DRAWN: TB
CHECKED: KF
JLR #: 29893

DRAWING #: CG1

PLOT DATE: December 2, 2022 12:37:41 PM

File Location: P:\29000\29893-000 - 1927 Maple Grove\5-Production\1-Civil\Conceptual\29893 C Conceptual.dwg

Appendix A

Summary of Pre-Consultation meetings

1927 Maple Grove
Pre-Consultation Comments

Planning Comments

1. This is a pre-consultation for a Major Zoning By-law Amendment and Plan of Subdivision application. Application form, timeline and fees can be found here.
2. Cash-in-lieu of parkland and associated appraisal fee will be required as a condition of approval as per the Parkland Dedication Bylaw.
3. Option P2 is preferred. However, recognizing this is dependent on the construction of a public street by a different developer, both options are acceptable.
4. Please include a pedestrian walkway connection on the north end of the site for P1 to access the future park.
5. The proposal is within the Kanata West Concept Plan area. Please review the KWCP for policy directions and also contact the Kanata West Owners Group for any cost sharing requirements.
6. Please provide a sidewalk in front of the site on Maple Grove.
7. What is the plan for the hydro pole in front of the site?
8. Services on Maple Grove will have to be extended to reach the site. Adjacent developers including Claridge at 1981 Maple Grove (Vincent Denomme vincent.denomme@claridgehomes.com) and Formasian at 1919 Maple Grove (Jayant Gupta jayant@110architects.ca) are jointly responsible for street urbanization/services. Please contact representative from Claridge and Formasian to discuss about private cost sharing contribution, and whether it may be applicable to this site.
9. The site contains archeology potential, an archeology study is required.
10. Please consult with the Ward Councillor prior to submission.

Urban Design Comments

1. Please provide a sidewalk on west side of the local street to align with access to the park
2. Option 2 allows for better access to the park to the north and is therefore the preferable option from a design perspective.
3. Please ensure future illustrations account for the required widening on Maple Grove Road.
4. The proposed 2.5 m setback on Maple Grove should be increased to a minimum of 3 meters to allow for tree planting.
5. Consideration/coordination with the two abutting landowners (west and east) should be taken to allow for private street connections in the future. These would consist of 8 m wide blocks as shown on the attached illustration.

Engineering Comments

- The Servicing Study Guidelines for Development Applications are available at the following link: https://ottawa.ca/en/city-hall/planning-and-development/information-developers/development-application-review-process/development-application-submission/guide-preparing-studies-and-plans
- Record drawings and utility plans are available for purchase from the City's Information Centre. Contact the City's Information Centre by email at informationcentre@ottawa.ca or by phone at (613) 580-2424 x44455
- Stormwater quantity control criteria – be consistent with the criteria specified in the Pond 4 final report and/or in the Kanata west Master servicing Study.
- Stormwater quality control – Consult with the Conservation Authority (MVCA) for their requirements. Include the correspondence with the MVCA in the stormwater/site servicing report.
- When calculating the composite runoff coefficient (C) for the site (post development), please provide a drawing showing the individual drainage area and its runoff coefficient.
- When using the modified rational method to calculate the storage requirements for the site, the underground storage should not be included in the overall available storage. The modified rational method assumes that the restricted flow rate is constant throughout the storm which, in this case, underestimates the storage requirement prior to the 1:100-year head elevation being reached. Alternately, if you wish to include the underground storage, you may use an assumed average release rate equal to 50% of the peak allowable rate. Otherwise, disregard the underground storage as available storage or provide modeling to support the design.
- Engineering plans are to be submitted on standard A1 size (594mm x 841mm) sheets.
- Phase 1 ESA and Phase 2 ESA must conform to clause 4.8.4 of the Official Plan that requires that development applications conform to Ontario Regulation 153/04.
- Boundary conditions are required to confirm that the required fire flows can be achieved as well as availability of the domestic water pressure on the City street in front of the development. Use Table 3-3 of the MOE Design Guidelines for Drinking-Water System to determine Maximum Day and Maximum Hour peaking factors for 0 to 500 persons and use Table 4.2 of the Ottawa Design Guidelines, Water Distribution for 501 to 3,000 persons. Please provide the following information to the City of Ottawa via email to request water distribution network boundary conditions for the subject site. Please note that once this information has been provided to the City of Ottawa it takes approximately 5-10 business days to receive boundary conditions.
    - Type of Development and Units
    - Site Address
    - A plan showing the proposed water service connection locations.
    - Average Daily Demand (L/s)
    - Maximum Daily Demand (L/s)

    - Peak Hour Demand (L/s)
    - Fire Flow (L/min)
      [Fire flow demand requirements shall be based on Fire Underwriters Survey (FUS) Water Supply for Public Fire Protection

      1999]

      Exposure separation distances shall be defined on a figure to support the FUS calculation and required fire flow (RFF).

    - Hydrant capacity shall be assessed to demonstrate the RFF can be achieved. Please identify which hydrants are being considered to meet the RFF on a fire hydrant coverage plan as part of the boundary conditions request.
- If the proposed development is going to be processed under a subdivision application with public streets, required storage needs to be provided at the street sags.

Transportation Comments

1. TIA will not be required.
2. ROW protection on Maple Grove between Huntmar and Stittsville Main is 26m even.
3. Geometric Road Design (GRD) drawings will be required with the first submission of underground infrastructure and grading drawings. These drawings should include such items as, but is not limited to:
    a. Road Signage and Pavement Marking for the subdivision;
    b. Intersection control measure at new internal intersections; and
    c. Location of depressed curbs and TWSIs;
    d. More details can be provided upon request
4. Residential roads are to be designed for 30km/h operating speed.
5. Include traffic calming measures on roads within the limits of their subdivision to limit vehicular speed and improve pedestrian safety. Traffic calming measures shall reference best management practices from the Canadian Guide to Neighbourhood Traffic Calming, published by the Transportation Association of Canada, and/or Ontario Traffic Manual, and/or the City of Ottawa's Draft Traffic Calming Design Guidelines. These measures may include either vertical or horizontal features (such measures shall not interfere with stormwater management and overland flow routing), including but not limited to:
    a. intersection or mid block narrowings, chicanes, medians;
    b. speed humps, speed tables, raised intersections, raised pedestrian crossings;
    c. road surface alterations (for example, use of pavers or other alternate materials, provided these are consistent with the City's Official Plan polices related to Design Priority Areas);
    d. pavement markings/signage; and

    e. temporary/seasonal installations such as flexi posts or removable bollards.
6. Urbanize the north side of Maple Grove with curb and sidewalk (along the frontage).
7. Noise Impact Studies required:
        i. Detailed before registration
    b. Road

Environment Comments

In terms of preference, between the two options I prefer the one with the ability to provide the most urban tree canopy. This is something they can model in their TCR, EIS or planning rational.

In terms of the EIS, the only trigger is potential species at risk. If they identify special concern species (like the eastern wood-pewee) then they should address this species from a significant wildlife perspective.

Forestry Comments

1. A Tree Conservation Report (TCR) must be supplied for review along with the suite of other plans/reports required by the City; an approved TCR is a requirement of Site Plan or Plan of Subdivision approval
2. Any removal of privately-owned trees 10cm or larger in diameter require a tree permit issued under the Urban Tree Conservation Bylaw; the permit is based on the approved TCR
3. The TCR must list all trees on site by species, diameter and health condition.
4. The TCR must address all trees with a critical root zone that extends into the developable area.
5. If trees are to be removed, the TCR must clearly show where they are and document the reason they can not be retained
6. All retained trees must also be shown and all retained trees within the area impacted by the development process must be protected as per the City guidelines listed on Ottawa.ca
7. Trees with a trunk that crosses/touches a property line are considered co-owned by both property owners; permission from the adjoining property owner must be obtained prior to the removal of co-owned trees
8. The City does encourage the retention of healthy trees wherever possible; please ask your design/planning team to find opportunities for retention wherever

possible if the trees are healthy and will contribute to the design/function of the site. For more information on the process or help with tree retention options, contact Mark Richardson mark.richardson@ottawa.ca

9. The removal of City-owned trees will require the permission of Forestry Services who will also review the submitted TCR; note that Forestry Services may ask for compensation for any City-owned tree that has to be removed.

Please refer to the links to "Guide to preparing studies and plans" and fees for general information. Additional information is available related to building permits, development charges, and the Accessibility Design Standards. Be aware that other fees and permits may be required, outside of the development review process. You may obtain background drawings by contacting informationcentre@ottawa.ca.

These pre-con comments are valid for one year. If you submit a development application(s) after this time, you may be required to meet for another pre-consultation meeting and/or the submission requirements may change. You are as well encouraged to contact us for a follow-up meeting if the plan/concept will be further refined.

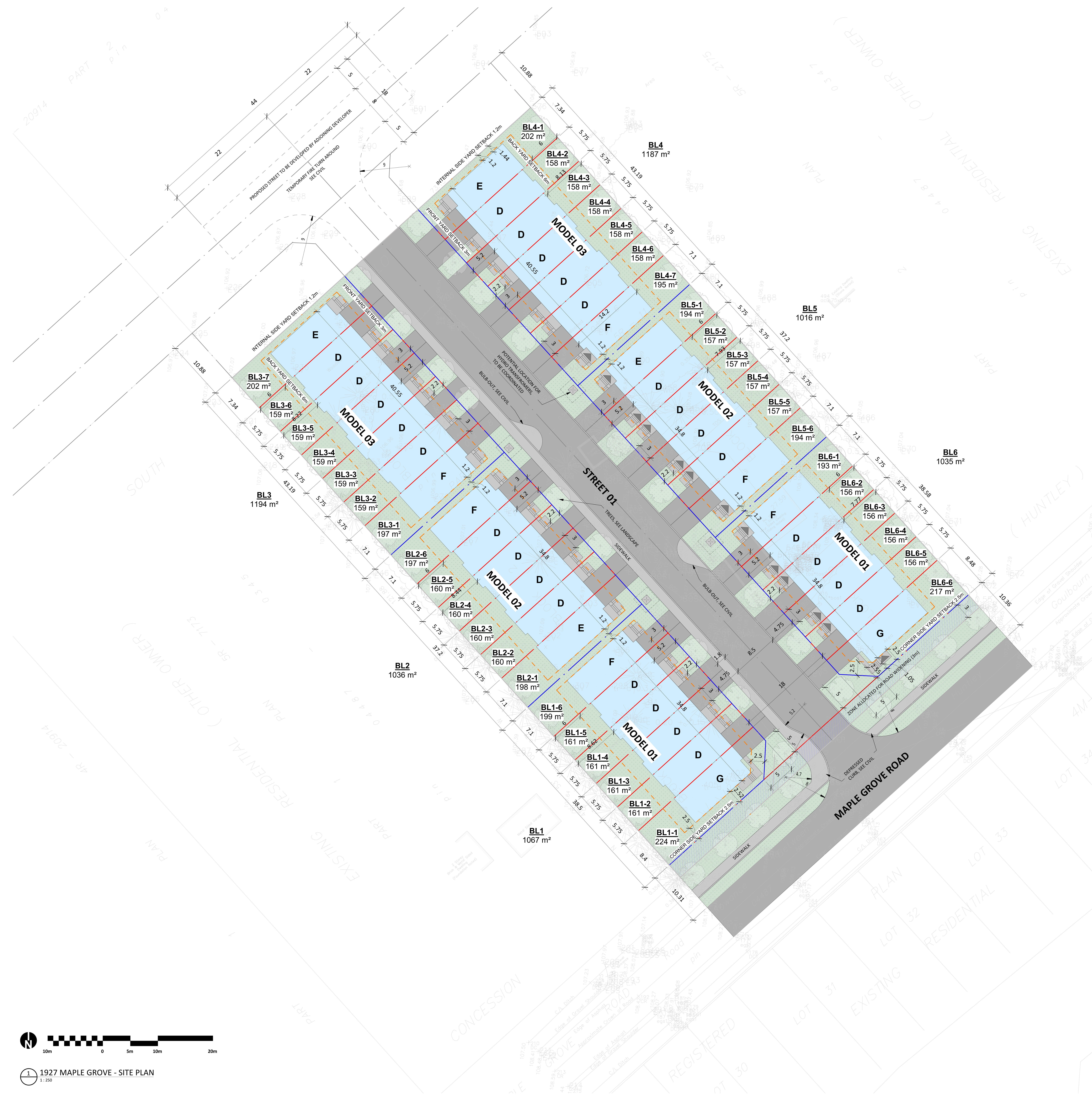
Please contact me at stream.shen@ottawa.ca or at 613-580-2424 extension 24488 if you have any questions.

Sincerely,

Stream Shen MCIP RPP
Planner II
Development Review - West

Appendix B

Concept Plan and
Topographical Survey



1 1927 MAPLE GROVE - SITE PLAN
1 : 250

C:\Users\ppomerleau\Documents\21009_MAPLE-GROVE_CENTRAL_R21_ppomerleauUW453.rvt

SITE PLAN LEGEND

- LOT LINE
- SETBACKS
- EXISTING BUILDING
- NEW BUILDING
- GRASS
- ASPHALT
- SIDEWALK
- NEW TREE, SEE LANDSCAPE

SITE INFORMATION & DEVELOPMENT STATISTICS

LOT		PIN
		04487 - 0345
SITE AREA		
TOTAL SITE AREA:		~8,915 sq m (0.89ha)
UNITS		
TOWNHOUSES: 38 UNITS		
ZONING		R3YY
	REQUIRED	PROVIDED
MINIMUM LOT WIDTH		
TOWNHOUSE	6 m	5.75 m
MINIMUM LOT AREA		
TOWNHOUSE	137 sq m	160 sq m
MAXIMUM BUILDING HEIGHT	12 m	~9.8 m
SETBACKS		
MINIMUM FRONT YARD:	3 m	5.2 m
MINIMUM CORNER SIDE YARD:	2.5 m	2.5 m
MINIMUM INTERIOR SIDE YARD:	1.2 m	1.2 m
MINIMIM REAR YARD:	6 m	8.24 m
PARKING RATES		
R9 - TOWNHOUSES:	1 p/unit = 38	38 (GARAGES)
VISITOR:	0	38 DRIVE AISLES
GROSS FLOOR AREA		
TOWNHOUSE D:		225 sq m
TOWNHOUSE E:		236 sq m
TOWNHOUSE F:		236 sq m
TOWNHOUSE G:		236 sq m
TOTAL MODEL 01 (GDDDDF)		1,372 sq m
TOTAL MODEL 02 (EDDDDF)		1,372 sq m
TOTAL MODEL 03 (EDDDDDF)		1,597 sq m

NOTE

1. ASSUMES TYPICAL RESIDENTIAL FLOOR HEIGHT OF 3m.
2. THE BASE PLAN (LOT LINES, EXISTING ROADS AND SURROUNDING AREAS) IS BASED ON THE TOPOGRAPHICAL PLAN OF SURVEY OF ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
3. DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0,3048.

PROJECT
MAPLE GROVE DEVELOPMENT
1927, MAPLE GROVE ROAD, STITTSVILLE, ON K2S 1B9

OWNER
LATITUDE Homes
1202, CARP ROAD, STITTSVILLE, ON K2S 1B9

ARCHITECTURAL
PMA ARCHITECTES
(418) 651-8954
INFO@PMAARCHITECTES.COM
3070, CHEMIN DES QUATRE-BOURGEOIS QUÉBEC (QC) G1W 2K4
PMAARCHITECTES.COM

PLANNER
FOTENN Planning + Design
396 Cooper Street, Suite 300, Ottawa ON K2P 2H7
613.730.5709 www.fotenn.com

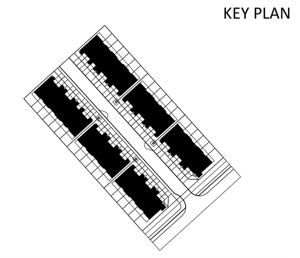
SURVEYOR
ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
SURVEYING
14 CONCOURSE GATE, SUITE 500, NEPEAN, ON K2E 7S6

CIVIL
J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS
1565 CARLING AVENUE, SUITE 700, OTTAWA, ON K1Z 8R1

GEOTECHNICAL
exp.
2650, QUEENSVIEW DRIVE, SUITE 100, OTTAWA, ON K2B 8H6

ENRIVONMENTAL CONSULTANT
Bowfin Environmental Consulting
168, MONTREAL ROAD, CORNWALL, ON K6H 1B3

KEY PLAN



ARCHITECT SEAL

REVISIONS

NO	DESCRIPTION	DATE
5	FOR CITY REVIEW	2022-11-24
4	FOR COORDINATION	2022-11-23
3	FOR COORDINATION	2022-09-12
2	FOR COORDINATION	2022-09-07
1	FOR DEVELOPMENT APPLICATION	2021-09-17

NOTE

IT IS THE RESPONSIBILITY OF THE APPROPRIATE CONTRACTOR TO CHECK AND VERIFY ALL DIMENSIONS ON THE SITE AND TO REPORT ALL ERRORS AND/OR OMISSIONS TO THE ARCHITECT. ALL CONTRACTORS MUST COMPLY WITH ALL PERTINENT CODES AND BY-LAWS. DO NOT SCALE DRAWINGS.

FOR COORDINATION
DO NOT USE FOR CONSTRUCTION
2022-11-24

DATE 2022-11-24	DESIGNED PP
	DRAWN PP
PROJECT No 21009	CHECKED PM

SHEET TITLE
SITE PLAN

SHEET No
A101

TOPOGRAPHICAL PLAN OF SURVEY OF

PART OF LOT 1
CONCESSION 1
GEOGRAPHIC TOWNSHIP OF HUNTLEY
CITY OF OTTAWA

Prepared by Annis, O'Sullivan, Vollebekk Ltd.

Scale 1 : 250

10 7.5 5.0 2.5 0 5 10 Metres

Metric

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

Surveyor's Certificate

I CERTIFY THAT :

1. This survey and plan are correct and in accordance with the Surveys Act, the Surveyors Act and the regulations made under them.
2. The survey was completed on the 21st day of December, 2020.

Jan 7/21
Date

T. Hartwick
Ontario Land Surveyor

Notes & Legend

	Denotes	
—□—	"	Survey Monument Planted
—■—	"	Survey Monument Found
SIB	"	Standard Iron Bar
SSIB	"	Short Standard Iron Bar
IB	"	Iron Bar
(WIT)	"	Witness
Meas.	"	Measured
(AOG)	"	Annis, O'Sullivan, Vollebekk Ltd.
(PI)	"	Plan 4R-20914
(P2)	"	(857) Plan April 10, 1984
	"	Deciduous Tree
	"	Coniferous Tree
BH		Borehole
FH	"	Fire Hydrant
WV	"	Water Valve
W	"	Well Cap
MH-ST	"	Maintenance Hole (Storm Sewer)
MH-S	"	Maintenance Hole (Sanitary)
HM	"	Hydro Meter
VC	"	Valve Chamber (Watermain)
S	"	Sign
UP	"	Utility Pole
AN	"	Anchor
AC	"	Air Conditioner
MB	"	Mail Box
— P —	"	Approximate Underground Power
— OHW —	"	Overhead Wires
CBRW	"	Concrete Block Retaining Wall
CSP	"	Corrugated Steel Pipe
INV	"	Invert
T/G	"	Top of Grate
Ø	"	Diameter
+ 65.00	"	Location of Elevations
+ 65.00	"	Top of Wall Elevation
BOS	"	Bottom of Slope
TOS	"	Top of Slope
PWF	"	Post & Wire Fence
RF	"	Rail Fence
C/L	"	Centreline
CH	"	Cedar Hedge
——	"	Property Line

ASSOCIATION OF ONTARIO LAND SURVEYORS
PLAN SUBMISSION FORM
2150130
THIS PLAN IS NOT VALID UNLESS IT IS AN EMBOSSED ORIGINAL COPY ISSUED BY THE SURVEYOR In accordance with Regulation 1026, Section 29 (3).

SITE AREA = 8917.7 m²
0.89 hectares

Bearings are grid, derived from Can-Net 2016 Real Time Network GPS observations, MTM Zone 9 (76°30' West Longitude) NAD-83 (original).

For bearing comparisons, a rotation of 0°25'20" counter-clockwise was applied to bearings on Plan 4R-20914.

ELEVATION NOTES

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

UTILITY NOTES

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

ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
14 Concourse Gate, Suite 500
Nepean, Ont. K2E 7S6
Phone: (613) 727-0850 / Fax: (613) 727-1079
Email: Nepean@aovltd.com
Ontario Land Surveyors

Job No. 21202-20 1927MapleGroveRd PrtLt1Con1Hnt T F | DG

Appendix C

Hydraulic Network Analysis
Information and Calculations

Water Demand Calculations
1927 Maple Grove (JLR 29893-001)

Unit Breakdown	**No.**	**Person Per Unit (Table 4.1)**
Singles	0	3.4
Semi-detached or Townhouse	40	2.7
Back to Back	0	2.7
Terrace Stacked Units	0	2.7
High Density Residential	0	1.8
Totla Unit Count =	40	
Total Population	108	ppl
Average Day Consumption Rate	280	L/c/d
Average Day Demand	**0.35**	**L/s**
Maximum Day Peaking Factor	6.51	x Avg Day (Table 3-3 MOE)
Maximum Day Demand	**2.28**	**L/s**
Peak Hour Peaking Factor	9.82	x Max Day (Table 3-3 MOE)
Peak Hour Demand	**3.44**	**L/s**
Minimum Hour Peaking Factor	0.10	x Avg Day (Table 3-1 MOE)
Minimum Hour Demand	**0.04**	**L/s**

1927 Maple Grove

Maple Grove Towns Inc.

29893-001

<u>Boundary Conditions (March 12, 2021 Email from the City):</u>

Water Demand Scenario	Demands (L/s)	Head (m) Maple Grove
Peak Hour	0.35	156.7
Maximum HGL	0.04	161.0
MXDY + FF	3.44	154.1

Note: The supply elevations under the maximum day demand plus fire flow estimated by the City based on RFF of 10,000 L/min

Headloss Calculations (Hazen Williams Equation)

Calculate headloss in a given pipe length based on flows and C value

HL = 10.675 * L * Q^1.852 / (C^1.856 * D ^4.8704)

Where,

HL = Headloss (m)

L - Length (m)

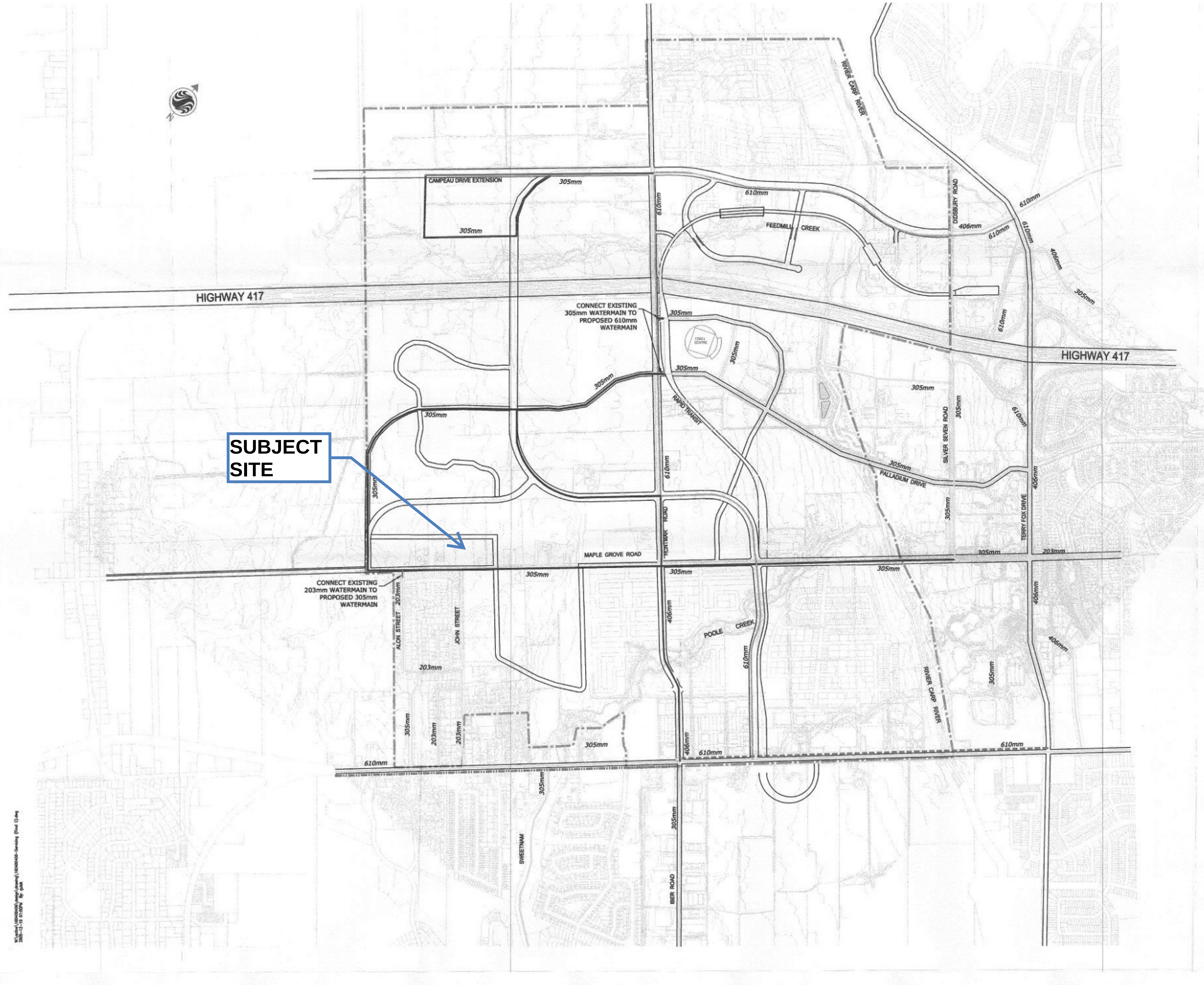
Q - Flow (m^3/s)

C - Hazen Williams "C"

D - Main Diameter (m)

Lengths used in the Headloss Calculations	
Pk HR & Max HGL (200 mm WM) :	124.7
Max Day plus FF (at northern hydr	92.0

Water Demand Condition	Flow - Q (L/s)	Flow - Q (m^3/s)	Length (m) to U/S 200 mm	Length (m) to U/S hydrant	C	D (m)	HeadLoss (m)	HGL (m) @ Maple Grove	Calculated HGL (m)	Elevation (m)	Pressure @ Node (m)	Pressure @ Node (kPa)	Requirement	Criteria Acheived?
Average Day	0.35	0.00035	124.7	124.7	110	0.200								
Maximum Day	2.28	0.00228	124.7	124.7	110	0.200								
Peak Hour	3.44	0.00344	124.7	124.7	110	0.200	0.01507	156.700	156.685	108	48.685	478	275	Yes
Maximum HGL	0.04	0.00004	124.7	124.7	110	0.200	0.00000	161.000	161.000	108	53.000	520	552	Yes
Maximum Day Plus Fire (Q = 2.28 L/s + 167 L/s)	169.28	0.16928	92.0	124.7	110	0.200	20.55963	154.100	133.540	108	25.540	251	140	Yes



Stantec Consulting Ltd.
1505 Laperriere Avenue
Ottawa ON Canada
K1Z 7T1
Tel. 613.722.4420
Fax. 613.722.2799
www.stantec.com

Stantec

Legend

KANATA–WEST CONCEPT PLAN BOUNDARY
EXISTING WATERMAIN
EXISTING 610mm WATERMAIN TO BE UPGRADED TO 914mm
EXISTING 610mm WATERMAIN TO BE UPGRADED TO 762mm
PROPOSED 610mm DIA. WATERMAIN
PROPOSED 406mm DIA. WATERMAIN
PROPOSED 305mm DIA. WATERMAIN
PROPOSED 203mm DIA. WATERMAIN

Notes

INTERNAL WATERMAIN SIZE ARE EXPECTED TO VARY FROM 152mm TO 305mm.

CCL/IBI

Revision		By	Appd.	Date
5	REVISED FOR DEC.21/05 SUBMISSION	GBU	SJP	DEC.21/05
4	REVISED AS PER CITY COMMENTS (Sept.16/05)	GBU	MAF	OCT.28/05
3	REVISED WATER DISTRIBUTION NETWORK	GBU	S.J.P.	AUG 09/05
2	REVISED POND 1 AREA	NI	MAF	JUNE 09/05
1	REVISED LOTTING FOR TARTAN AND MATTAMY	BCB	SJP	JAN.18/05

File Name: Dwn. Chkd. Dsgn. Date

Seals

Client/Project

Kanata West Concept Plan
Master Servicing Study

Ottawa, Ontario

Title

Watermain
Final Concept

Project No. 60400406
Scale 1:7500 0 75 225 375m

Drawing No. WM-1
Sheet 2 of 7
Revision 5

FUS Fire Flow Calculations

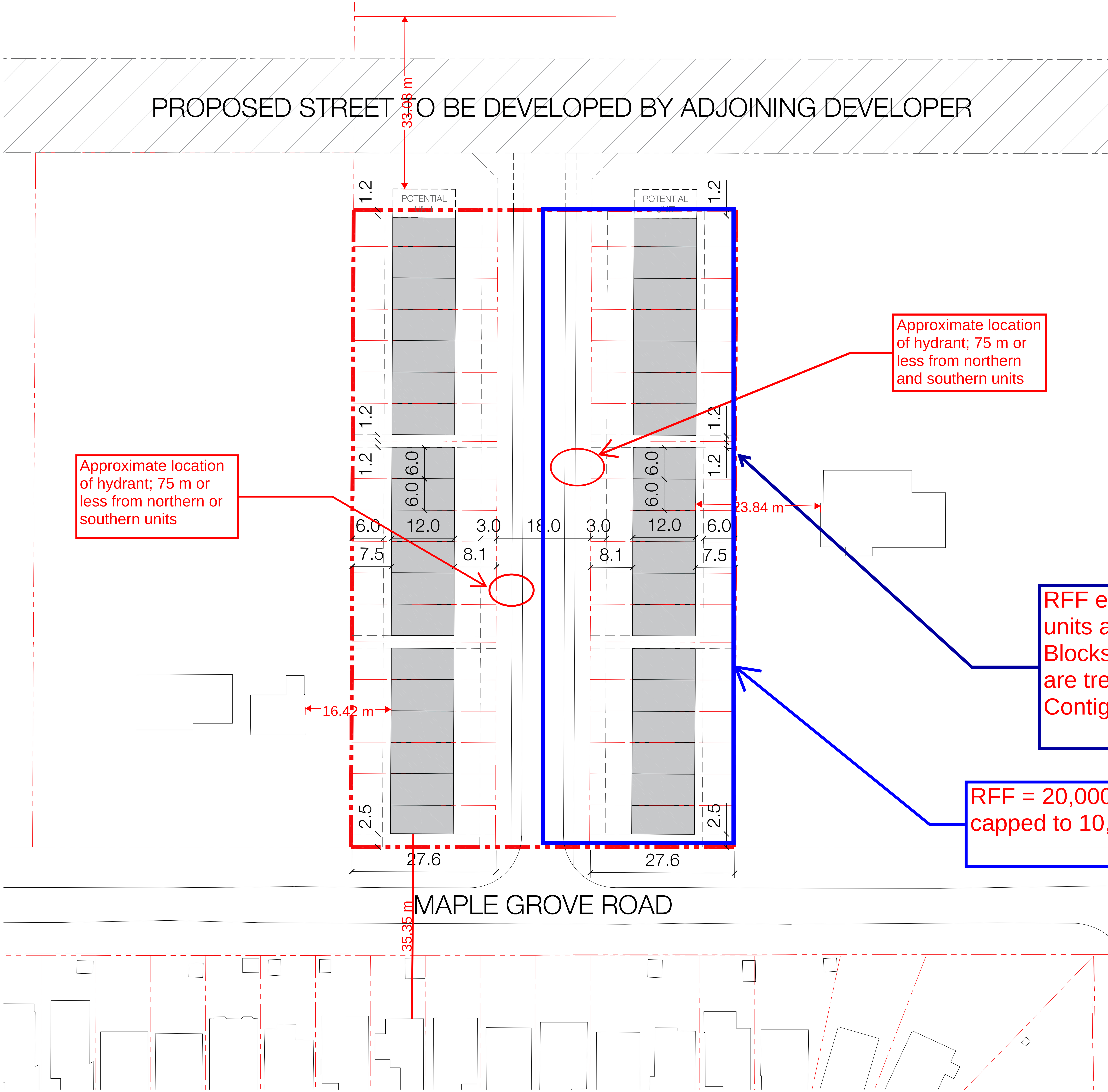
1927 Maple Grove - Row Townhouse

(JLR 29893-001)

OPTION 2 Concept

Step	Parameter	Value		Note
A	Type of Construction	Wood Frame		
	Coefficient (C)	1.5		
B	Ground Floor Area	1440	m^2	Includes 3 Blocks of TH (20 units) as the seperation between the Blocks is 2.4 m (less than 3 m), blocks to be treated as contiguous
C	Height in storeys	2	storeys	Basements are excluded.
	Total Floor Area	2880	m^2	
D	Fire Flow Formula	$F=220C\sqrt{A}$		
	Fire Flow	17710	L/min	
	Rounded Fire Flow	18000	L/min	Flow rounded to nearest 1000 L/min.
E	Occupancy Class	Limited Combustible		Residential buildings have a limited combustible occupancy.
	Occupancy Charge	-15%		
	Occupancy Increase or Decrease	-2700		
	Fire Flow	15300	L/min	No rounding applied.
F	Sprinkler Protection	None		
	Sprinkler Credit	0%		
	Decrease for Sprinkler	0	L/min	
G	*North Side Exposure*			
	Exposing Wall:	Wood Frame		
	Exposed Wall:	Wood Frame		
	Length of Exposed Wall:	12.0	m	
	Height of Exposed Wall:	2	storeys	
	Length-Height Factor	24.0	m-storeys	
	Separation Distance	33.03	m	
	North Side Exposure Charge	5%		
	East Side Exposure			
	Exposing Wall:	Wood Frame		
	Exposed Wall:	Wood Frame		
	Length of Exposed Wall:	18.0	m	
	Height of Exposed Wall:	2	storeys	
	Length-Height Factor	36.0	m-storeys	
	Separation Distance	23.84	m	
	East Side Exposure Charge	8%		
	South Side Exposure			
	Exposing Wall:	Wood Frame		
	Exposed Wall:	Wood Frame		
	Length of Exposed Wall:	12.0	m	
	Height of Exposed Wall:	2	storeys	
	Length-Height Factor	24.0	m-storeys	
	Separation Distance	35.25	m	
	South Side Exposure Charge	5%		
	West Side Exposure			
	Exposing Wall:	Wood Frame		
	Exposed Wall:	Wood Frame		
	Length of Exposed Wall:	12.0	m	
	Height of Exposed Wall:	2	storeys	
	Length-Height Factor	24.0	m-storeys	
	Separation Distance	16.42	m	
	West Side Exposure Charge	12%		
	Total Exposure Charge	30%		The total exposure charge is below the maximum value of 75%.
	Increase for Exposures	4590	L/min	
H	Fire Flow	19890	L/min	
	Rounded Fire Flow	20000	L/min	Flow rounded to nearest 1000 L/min.
City Cap	**Required Fire Flow (RFF)**	**10000**	**L/min**	The City of Ottawa's cap does apply since there is the 10 m minimum separation between the back of the units and no side flankage.
		167	**L/s**	

Fire Underwriters Survey (FUS) Fire Flow Calculations
In accordance with City of Ottawa Technical Bulletin ISTB-2018-02 dated March 21, 2018



SITE INFORMATION

ZONING	R3YY
SITE AREA	
Total Site Area:	8,916m^2
HEIGHT	
Residential:	4 Storeys (12m)
PARKING RATES	REQUIRED
Residential:	1.0 p/unit

SETBACKS	F.Y.	S.Y.	I.S.Y.	R.Y.
	3m	2.5m	1.2m	6m

DEVELOPMENT STATISTICS

RESIDENTIAL UNITS
Townhouses: 38

PARKING	Required	Provided
Residential:	38	38

NOTES

1. Assumes typical Residential floor height of 3m.

2. The base plan (lot lines, existing roads and surrounding areas) is based on the City's Open Data and aerial images. The site area is approximate and all dimensions need to be confirmed by a proper survey.

1927 MAPLE GROVE ROAD

CONCEPT PLAN 2

LEGEND

PROPOSED BUILDING
PROPERTY BOUNDARY
SETBACKS

0 5m 10m 15m 30m

No.	REVISION	DATE	BY
1	DRAFT	2020.03.05	CB

CLIENT
ZAYOUN GROUP INC

FOTENN
Planning + Design

396 Cooper Street, Suite 300, Ottawa ON K2P 2H7
613.730.5709 www.fotenn.com

DESIGNED	RP
REVIEWED	RP
DATE	2020.03.05

P2

Appendix D

Background Documents
Sanitary

Stantec Consulting Ltd.
1505 Laperriere Avenue
Ottawa ON Canada
K1Z 7T1
Tel. 613.722.4420
Fax. 613.722.2799
www.stantec.com

CCL/IBI
Cumming Cockburn Limited
1770 WOODWARD DR., OTTAWA (613)225-1311

Legend

ULTIMATE MAJOR DRAINAGE LIMIT
SUBCATCHMENT AREAS
PROPOSED TRUNK SEWER
PROPOSED FORCEMAIN
TEMPORARY FORCEMAIN
PROPOSED STITTSVILLE PUMPING STATION AND FORCEMAIN
EXISTING TRUNK SEWER
MAJOR DRAINAGE SPLIT
NODES
EXISTING PUMPING STATION AND FORCEMAIN (TO BE DECOMMISSIONED)
44 25.54 ha. INPUT POINT AND AREA IN HECTARES
EXISTING PUMPING STATION GRAVITY OUTLET

	Revision	By	Appd.	Date
5	REVISED FOR DEC.21/05 SUBMISSION	G.B.U.	S.J.P.	05:12:21
4	REVISED TRUNK SEWER FROM 16 TO KWPS	R.W.W.	R.W.W.	05:10:05
3	ARROWS FOR EXIST. PUMP STATIONS ADDED	R.W.W.	R.W.W.	05:08:09
2	REPORT JUNE 2005	R.W.W.	R.W.W.	05:06:07
1	REPORT APR. 2005	R.W.W.	R.W.W.	05:04:20

File Name:

Dwn. Chkd. Dsgn. Date

Seals

Client/Project

Kanata West Concept Plan
Master Servicing Study

Ottawa, Ontario

Title

Preferred Waste-Water Option

Project No. 60400406

Scale 1:7500 (0 75 225 375m)

Drawing No. S-I

Sheet 7 of 7

Revision 5

SANITARY SEWER DESIGN SHEET
PROJECT : Kanata West Servicibility Study
LOCATION : CITY OF OTTAWA

PAGE 1 OF 1
PROJECT: 3598-LD-03
DATE: April 2005
DESIGN: JIM
FILE: 3598LD.sewers.XLS

MODEL 1 ULTIMATE (population based criteria..ICI simultaneous peaking)

LOCATION: STREET	FROM MH	TO MH		TOTAL AREA (Ha)	RESIDENTIAL: APPLIC AREA (Ha)	UNIT/Ha	TOTAL UNITS	POPULATION INDIV	POPULATION ACCUM	PEAK FACTOR	PEAK FLOW (l/s)	EMPLOYMENT/RETAIL/BUSINESS PARK/OPEN SPACES: APPLIC AREA (Ha)	ACCUM AREA (Ha)	TOTAL AREA (Ha)	FLOW RATE (l/Ha/d)	PEAK FLOW INDIV (l/s)	PEAK FLOW ACCUM (l/s)	PEAK FLOW TOTAL (l/s)	INFILTRATION: AREA (Ha) INDIV	AREA (Ha) CUMUL	TOTAL CUMUL	PEAK FLOW (l/s)	TOTAL FLOW (l/s)	PROPOSED SEWER: CAPACITY l/s	VELOCITY (full) m/s	LGTH. (m)	PIPE (mm)	GRADE %	AVAIL. CAP. (%)	HARMON PF	ACTUAL q/Q	va/Vf	ACTUAL VELOCITY (m/s)
Campeau Drive Trunk Sewer	1	2	Area 1 (PBP)	38.11								38.11	38.11		35000	23.16	23.16		38.11	38.11													
			Area 2 (PBP)	27.29								27.29	65.40		35000	16.58	39.74		27.29	65.40													
			Area 3 Ext Employment	14.05								14.05	79.45		50000	12.20	51.94		14.05	79.45													
			Area 4 HP Employment	10.93								10.93	90.38	90.38	50000	9.49	61.42	61.42	10.93	90.38	90.38	25.31	86.73	283.79	1.27	525.0	525	0.40	69.44%		0.306	0.730	0.927
	2	3	Area 5 Residential	29.19	29.19	19	555	1664	1664	3.65	24.58			90.38				61.42	29.19	29.19										3.65			
			Area 9 Ext Employment	8.45							24.58	8.45	8.45	98.83	50000	7.34	7.34	68.76	8.45	128.02	128.02	35.85	129.18	286.61	0.98	700.0	600	0.20	54.93%		0.451	0.830	0.815
	14	3	Area 6/8 Ext Employment	16.65								16.65	16.65		50000	14.45	14.45		16.65	16.65													
			Area 7 HP Employment	5.48								5.48	22.13	22.13	50000	4.76	19.21	19.21	5.48	22.13	22.13	6.20	25.41	148.74	0.91	910.0	450	0.25	82.92%		0.171	0.630	0.571
	3	4							1664	3.65	24.58	0.00	0.00	120.96		0.00	0.00	87.97	0.00	0.00	150.15	42.04	154.59	392.29	1.06	300.0	675	0.20	60.59%	3.65	0.394	0.790	0.839
	4A	4	Area 10 Residential	27.86	27.86	19	529	1588	1588	3.66	23.55								27.86	27.86	27.86	7.80	31.36	148.74	0.91	750.0	450	0.25	78.92%	3.66	0.211	0.660	0.598
	4	5	14 Mixed Use	4.13	1.76	50	88	263	3515	3.38	48.17	2.37	2.37	123.33	35000	1.44	1.44	89.41	4.13	4.13	182.14	51.00	188.58	392.29	1.06	450.0	675	0.20	51.93%	3.38	0.481	0.840	0.892
	Queensway	5	Area 13 Community Retail	6.35								6.35	6.35	6.35	35000	3.86	3.86	3.86	6.35														
			Area 11/12 Mixed Use	11.80	5.02	50	251	752	752	3.88	11.81	6.79	13.14	13.14	35000	4.12	7.98	7.98	11.80	18.15	18.15	5.08	24.88	43.88	0.87	420.0	250	0.50	43.31%	3.88	0.567	0.880	0.762
	5	5A	Area 15 Community Retail	3.88				0	4267	3.31	57.19	3.88			35000	2.36			3.88											3.31			
First Line Road Sewer			Area 44	25.54							57.19	25.54	29.42	165.89	35000	15.52	17.88	115.27	25.54	29.42	229.71	64.32	236.77	519.43	1.14	300.0	750	0.20	54.42%		0.456	0.830	0.945
				229.71							57.19							115.27				64.32	236.77										
Signature Ridge		5A	Area 100 Residential	90.20	90.20	19	1714	5141	5141	3.23	67.35	0.00																		3.23			
Signature Ridge		5A	Area 100 Non-Residential	4.88							67.35	4.88	4.88	4.88	50000	4.24	4.24	4.24	95.08	95.08	95.08	26.62	98.21										
Intersticial Lands & Broughton/Richardson		5A																					65.00										
Total To SRPS	**5A**	**SRPS**		**324.79**	**154.02**		**3136**		**9409**		**124.54**	**170.77**						**119.51**			**324.79**	**90.94**	**399.98**	**580.53**	**1.27**	**30.0**	**750**	**0.25**	**31.10%**	2.98	0.689	0.940	1.197
Palladium Drive Trunk Sewer	6	7	Area 32 (PBP)	57.03								57.03	57.03		50000	49.51	49.51		57.03	57.03													
			Area 32A Park	8.34								8.34	65.37		0	0.00	49.51		8.34	65.37													
			Area 33/34 Ext Employment	54.85								54.85	120.22	120.22	50000	47.61	97.12	97.12	54.85	120.22	120.22												
	7	8	Area 37 Mixed Use	36.70	15.60	50	780	2340	2340	3.53	33.47	21.10	21.10	141.32	50000	18.32	18.32	115.44	36.70	36.70	156.92	43.94	192.85	455.83	1.23	925.0	675	0.27	57.69%	3.53	0.423	0.810	1.000
				156.92	15.60		780		2340		33.47	141.32						115.44	156.92		156.92	43.94	192.85							3.53			
Corel Centre Etc. (Existing Sewer) *		16	Area 35 HP Employment	6.05								6.05	6.05		30000	3.15	3.15		6.05														
		16	Area 36 (Corel Centre)																			30.00											
		16	Area 38 Exten Employment	20.15								20.15	26.20	26.20	14400	5.04	8.19	8.19	20.15	26.20	26.20	7.34	45.52			*Existing*							
First Line Road Sewer	15	16	Area 40 Employment	14.59								14.59	14.59		35000	8.87	8.87		14.59	14.59													
			Area 41 Employment	11.97								11.97	26.56		35000	7.27	16.14		11.97	26.56													
			Area 42 Employment	20.66								20.66	47.22		35000	12.55	28.69		20.66	47.22													
			Area 43 Employment	28.89								28.89	76.11	76.11	35000	17.55	46.25	46.25	28.89	76.11	76.11	21.31	67.56	224.35	1.00	525.0	525	0.25	69.89%		0.301	0.730	0.733
Carp River Trunk	16	8	Nothing To Add	102.31	15.60		780		2340	3.53	33.47	102.31	102.31	102.31	0	0.00	54.44	54.44	0.00	102.31	102.31	28.65	113.08	286.61	0.98	400.0	600	0.20	60.54%	3.53	0.395	0.790	0.776
Carp River Trunk	8	10A	Nothing To Add	259.23	15.60		780		2340		33.47	0.00	0.00	243.63		0.00	0.00	169.87	0.00	139.01	259.23	109.92	305.93	579.95	1.05	550.0	825	0.15	47.25%	3.53	0.528	0.860	0.904
Marle Grove Road Trunk Sewer	9	10	Area 18/19 Exist. Residential	23.34	23.34	19	443	1330	1330										23.34	23.34										3.72			
			Area 22/26/27 Residential	79.32	79.32	30	2380	7139	8469	3.03	103.82								79.32	102.66	102.66	28.74	132.56	405.11	1.39	775.0	600	0.40	67.28%	3.03	0.327	0.740	1.027
Hazeldean/Huntmar Trunk Sewer	11	12	Area 16/20 Residential	99.01	99.01	19	1881	5644	5644	3.20	73.06								99.01	99.01										3.20			
			Area 16/20 Commercial	33.50								33.50	33.50	33.50	50000	29.08	29.08	29.08	33.50	132.51													
			Area 16/20 Open Space	14.13								14.13							14.13	146.64													
			Area 17 Ex. Commercial	3.44							73.06	3.44	36.94	36.94	35000	2.09	31.17	31.17	3.44	150.08	150.08	42.02	146.26	554.82	1.50	775.0	675	0.40	73.64%		0.264	0.700	1.051
	12	10	Area 21 Exist. Employment	10.89								10.89	10.89	10.89	50000	9.45	9.45		10.89	10.89													
			Area 19A Exist Residential	6.63	6.63	19	126	378									9.45		6.63	17.52													
			Area 23/24 Community Retail	17.61								17.61	28.50	28.50	35000	10.70	20.15	51.32	17.61	35.13													
			Area 28/30 Residential	27.10	27.10	30	813	2439	8460	3.03	103.72	0.00	0.00	65.44				51.32	27.10	62.23	212.31	59.45	214.49	519.43	1.14	950.0	750	0.20	58.71%	3.03	0.413	0.800	0.911
Marle Grove Road Trunk Sewer	10	10A	Area 39 Mixed Use	21.13	8.98	50	449	1347				12.15	12.15	77.59	35000	7.38	7.38	58.71	21.13														
			Area 29 Residential	15.00	15.00	30	450	1350	19627	2.66	211.54							58.71	15.00	36.13	351.10	98.31	368.56	669.89	1.21	1000.0	825	0.20	44.98%	2.66	0.550	0.870	1.056
Carp River Trunk Sewer	13	10A	Area 25 Community Retail	20.24								20.24	20.24	20.24	35000	12.30	12.30	12.30	20.24														
			Area 31 residential	38.72	38.72	30	1162	3485	3485	3.39	47.80							12.30	38.72	58.96	58.96	16.51	76.61	320.17	1.10	1000.0	600	0.25	76.07%	3.39	0.239	0.680	0.746
		10A	Area 31A (PBP)	0.75								0.75	0.75	0.75	50000	0.65	0.65	0.65	0.75	0.75	0.75	0.21	0.86	36.69	0.72	100.0	250	0.35	97.65%		0.023	0.340	0.246
Pumping Station 2 to KWPS	**10A**	**KWPS**		**670.04**	**313.70**		**8484**		**25451**		**292.82**	**356.34**						**241.53**			**670.04**	**224.95**	**759.29**	**1273.71**	**1.43**	**30.0**	**1050**	**0.20**	**40.39%**	2.55	0.596	0.900	1.283
STUDY TOTALS				994.83	467.72		11620		34860			527.11																		2.41			

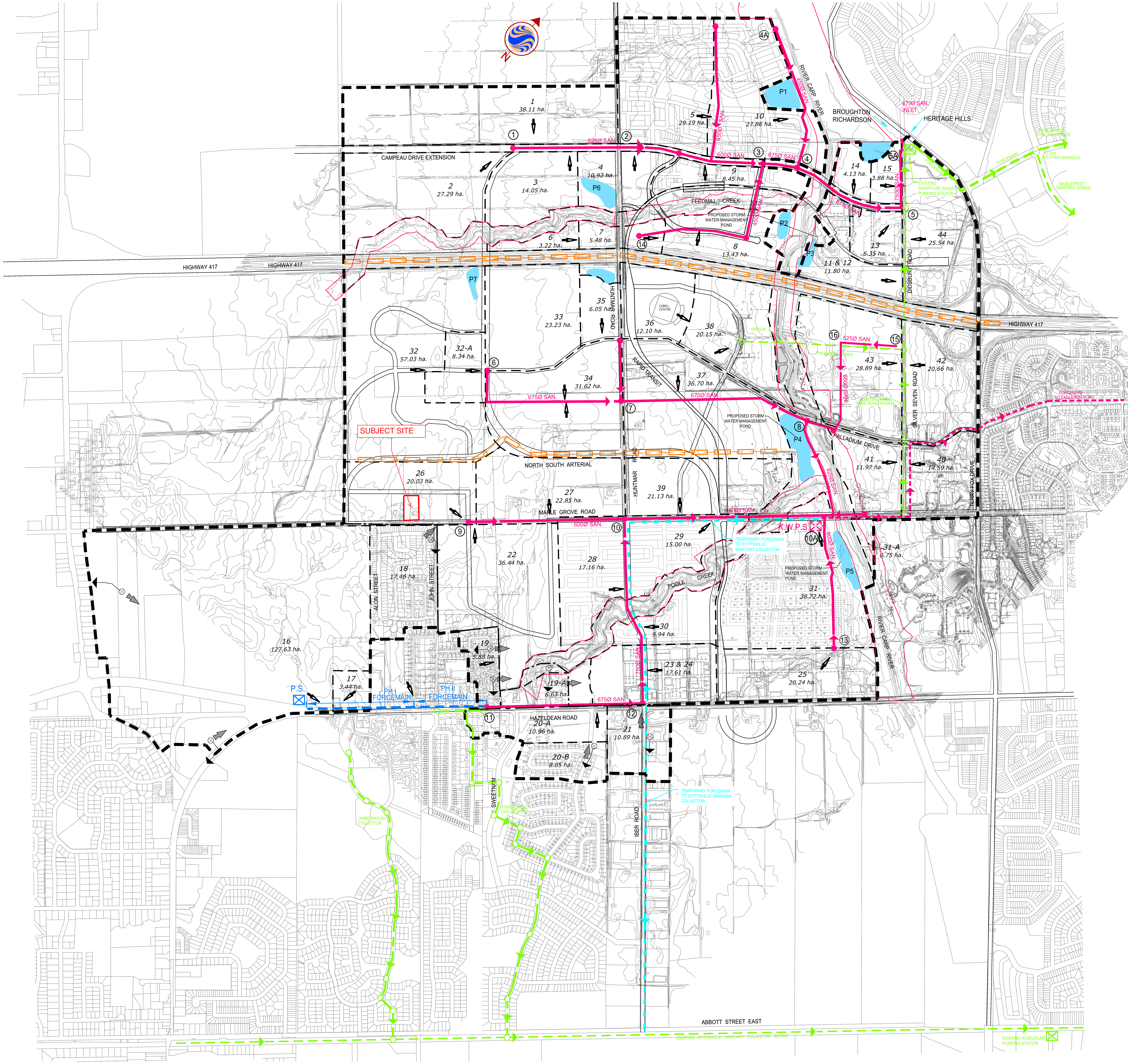
Average Daily Per capita Flow Rate = 350 l/cap/d
Infiltration Allowance Flow Rate = 0.28 l/sec/Ha
Residential Peaking Factor = 1+(14/(4+(P^0.5))), P=Pop. in 1000's, Max of 4
Population density per unit = 3.00
Peak For Employment/Retail/Business Park = 1.50
Mixed-Uses Assumes: 15% Community Retail, 42.5% Business Park and 42.5% Residential

Revision No. 1:	April 01, 2005	Revision No. 6:	Oct. 14, 2005
Revision No. 2:	April 11, 2005	Revision No. 7:	Nov. 10, 2005
Revision No. 3:	April 21, 2005	Revision No. 8:	Nov. 11, 2005
Revision No. 4:	June 07, 2005	Revision No. 9:	Apr. 19, 2006
Revision No. 5:	August 10, 2005		

CCL/IBI

Stantec

FIG. 4.2-1



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CCL/IBI
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Legend

	ULTIMATE MAJOR DRAINAGE LIMIT
	SUBCATCHMENT AREAS
	PROPOSED TRUNK SEWER
	PROPOSED FORCEMAIN
	TEMPORARY FORCEMAIN
	PROPOSED STITTSVILLE PUMPING STATION AND FORCEMAIN
	EXISTING TRUNK SEWER
	MAJOR DRAINAGE SPLIT
1	NODES
	EXISTING PUMPING STATION AND FORCEMAIN (TO BE DECOMMISSIONED)
44 25.54 ha.	INPUT POINT AND AREA IN HECTARES
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File Name: Dwn. Chkd. Dsgn. Date

Seals

Client/Project

Kanata West Concept Plan
Master Servicing Study

Ottawa, Ontario

Title

Preferred Waste-Water
Option

Project No. 60400406

Scale 1:7500 (0 75 225 375m)

Drawing No. S-I

Sheet 7 of 7

Revision 5

SANITARY SEWER CALCULATION SHEET

Ottawa

Manning's n=0.013

LOCATION			RESIDENTIAL AREA AND POPULATION							COMM		INDUST		INSTIT		C+I+I	INFILTRATION				PIPE						
STREET	FROM M.H.	TO M.H.	AREA (ha)	UNITS	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA (mm)	SLOPE (%)	CAP. (FULL) (l/s)	RATIO Q act/Q cap	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
MAPLE GROVE ROAD																											
			20.03	601	2044	20.03	2044										20.03	20.03									
	104A	105A	17.46	332	1129	37.49	3173	3.42	43.96								17.46	37.49	10.497	54.46	100.0	375	0.25	87.67	0.62	0.79	0.83
	105A	106A	0.70	0	0	38.19	3173	3.42	43.96								0.70	38.19	10.693	54.65	101.0	375	0.25	87.67	0.62	0.79	0.83
			0.40		39	38.59	3212										0.40	38.59									
			0.40	0	0	38.99	3212										0.40	38.99									
	106A	107A	15.72	310	1012	54.71	4224	3.31	56.64								15.72	54.71	15.319	71.96	70.0	450	0.20	127.50	0.56	0.80	0.82
	107A	108A	0.72	0	0	55.43	4224	3.31	56.64								0.72	55.43	15.520	72.16	63.0	450	0.20	127.50	0.57	0.80	0.82
	108A	109A	6.72	140	427	62.15	4651	3.27	61.61								6.72	62.15	17.402	79.01	80.0	450	0.40	180.32	0.44	1.13	1.09
	109A	1090A	1.74		168	63.89	4819	3.26	63.64								1.74	63.89	17.889	81.53	80.0	450	0.40	180.32	0.45	1.13	1.10
	1090A	110A				63.89	4819	3.26	63.64								0.00	63.89	17.889	81.53	83.0	450	0.40	180.32	0.45	1.13	1.10
			0.76	19	58	64.65	4877										0.76	64.65									
	110A	Ex. 88	13.79	310	946	78.44	5823	3.18	75.01								13.79	77.68	21.750	96.76	22.0	600	0.40	388.33	0.25	1.37	1.14
	Ex. 88	Ex. 89	0.78	0	0	79.22	5823	3.18	75.01								0.78	78.46	21.969	96.98	101.3	600	0.40	388.33	0.25	1.37	1.14
			4.07	158	427	83.29	6250										4.07	82.53									
			4.08	141	381	87.37	6631										4.08	86.61									
			3.08	100	270	90.45	6901										3.08	89.69									
			2.52			92.97	6901																				
			6.29	202	554	99.26	7455										6.29	95.98									
	Ex. 89	Ex. 89A	1.50	61	165	100.76	7620	3.07	94.76								1.50	97.48	27.294	122.05	72.8	600	0.40	388.33	0.31	1.37	1.21
	Ex. 89A	Ex. 90				100.76	7620	3.07	94.76								0.00	97.48	27.294	122.05	47.2	600	0.40	388.33	0.31	1.37	1.21
	Ex. 90	Ex. 91				100.76	7620	3.07	94.76								0.00	97.48	27.294	122.05	112.7	600	0.62	483.47	0.25	1.71	1.42
			0.82	0	0	101.58	7620										0.82	98.30									
			1.04	36	123	102.62	7743										1.04	98.52									
			1.76	71	242	104.38	7985										1.76	100.28									
			24.00	563	1845	128.38	9830										24.00	124.28									
			2.40	58	198	130.78	10028										2.40	126.68									
			5.29	105	357	136.07	10385										5.29	131.97									
	Ex. 91	Ex. 92	1.47	34	116	137.54	10501	2.93	124.64								1.47	133.44	37.363	162.00	96.1	825	0.28	759.56	0.21	1.42	1.12
	Ex. 92	Ex. 93				137.54	10501	2.93	124.64								0.00	133.44	37.363	162.00	88.9	825	0.51	1025.11	0.16	1.92	1.39
	Ex. 93	Ex. 94				137.54	10501	2.93	124.64								0.00	133.44	37.363	162.00	96.4	825	0.50	1015.01	0.16	1.90	1.39

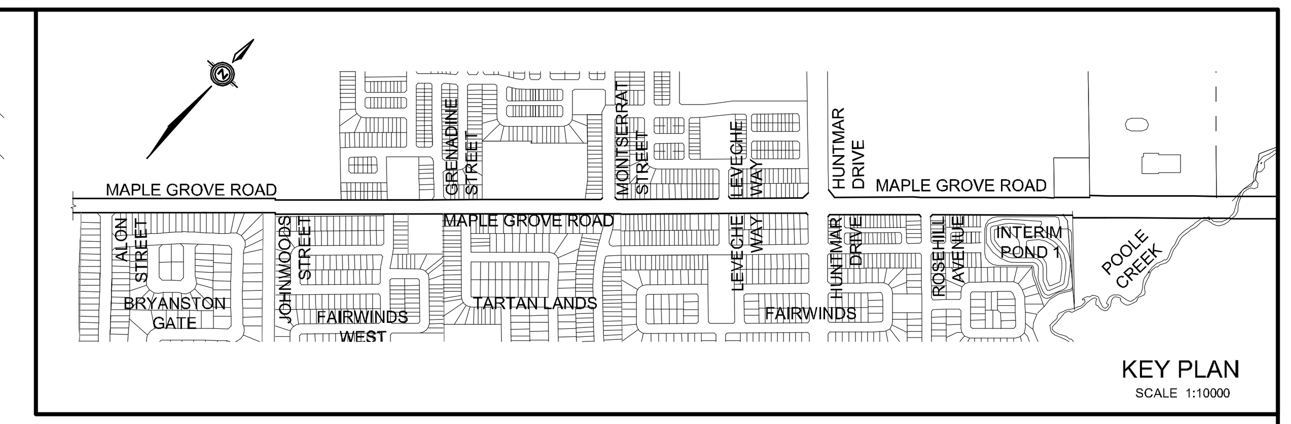
LICENSED PROFESSIONAL ENGINEER
Z. LI
PROVINCE OF ONTARIO

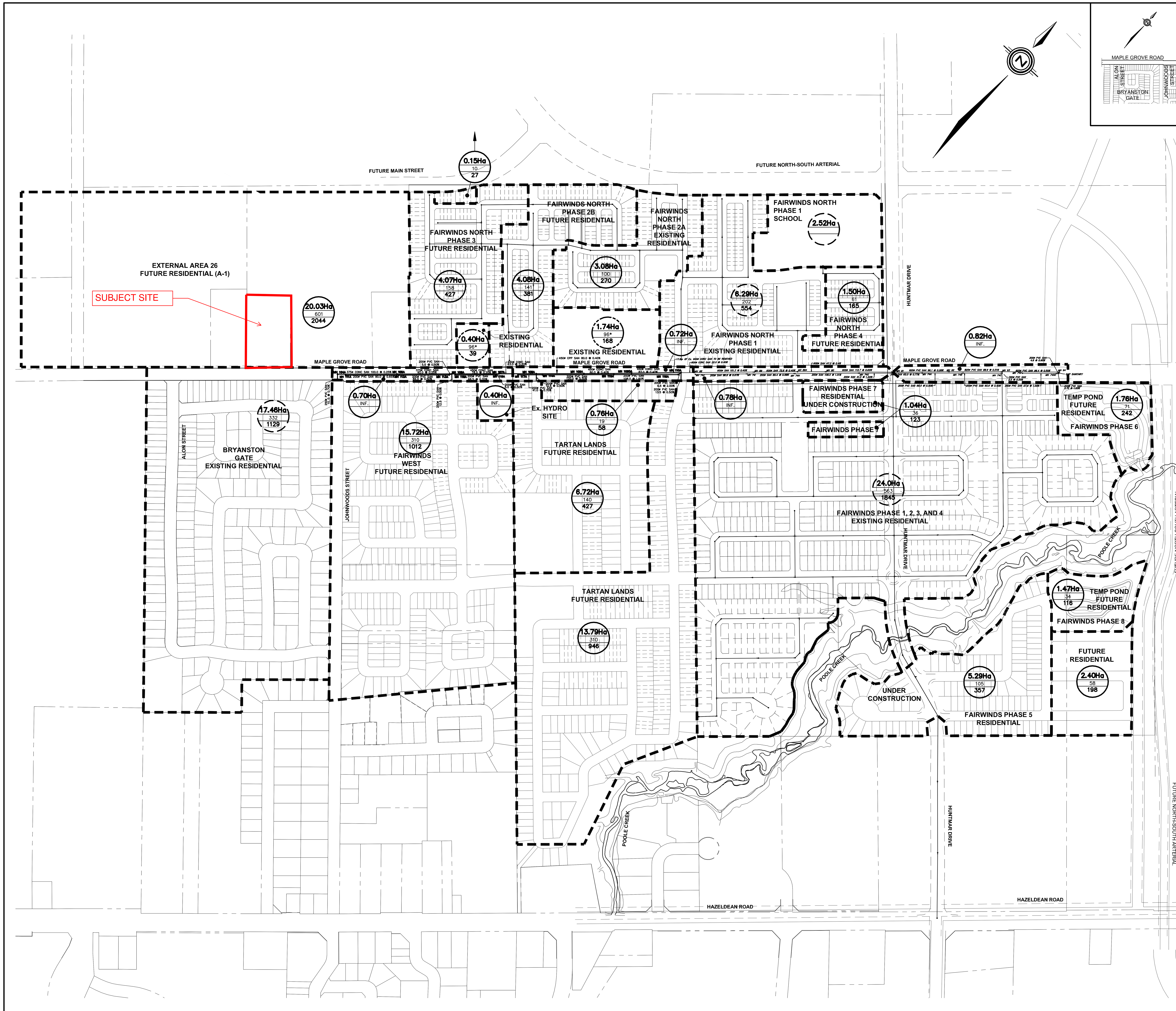
DESIGN PARAMETERS

Average Daily Flow =	350	l/p/day	Industrial Peak Factor = as per MOE Graph		
Commercial/Institution Flow =	50000	L/ha/da	Extraneous Flow =	0.280	L/s/ha
Industrial Flow =	35000	L/ha/da	Minimum Velocity =	0.760	m/s
Max Res. Peak Factor =	4.00		Manning's n =	0.013	
Commercial/Institution peak Factor =	1.50		Townhouse coeff=	2.7	
			Single house coeff=	3.4	

Designed: K.M.	PROJECT: RECONSTRUCTION OF MAPLE GROVE ROAD
Checked: Z.L.	LOCATION: City of Ottawa
Dwg. Reference: Sanitary Drainage Plan, Dwg. No. 6	File Ref: 10-451
Date: May, 2012	Sheet No. 1 of 1

451_san3.xls





LEGEND

0.82Ha / 20 / 111	DRANAGE AREA IN HECTARES UNIT NUMBER POPULATION (3.4 PERSON PER UNIT FOR SINGLE HOUSE). (2.7 PERSON PER UNIT FOR TOWNHOUSE).
0.40Ha / 96* / 39	EXISTING DRAINAGE AREA IN HECTARES DENSITY (PERSONS PER HA) POPULATION
0.82Ha / 111	EXTERNAL DRANAGE AREA IN HECTARES POPULATION
	SANITARY SEWER TRIBUTARY BOUNDARY
	SANITARY TRIBUTARY SUB–CATCHMENT BOUNDARY

TOPOGRAPHIC INFORMATION

TOPOGRAPHIC INFORMATION PROVIDED BY J.D. BARNES LIMITED, PROJECT No. 05–10–439–02, SURVEY DATED JULY 6, 2011 AND PROJECT No. 0X–10–628–02, SURVEY DATED NOVEMBER 27, 2008 AND PROJECT No. 06–10–561–01–00, SURVEY DATED FEBURARY 24, 2008 AND PROJECT No. 06–10–628–00, SURVEY DATED MARCH 29, 2007.

TOPOGRAPHIC INFORMATION PROVIDED BY STANTEC CONSULTING LIMITED AND WEBSTER & SIMMONDS SURVEYING LIMITED, FILE No. 161610660–111 RECEIVED ON DECEMBER 1, 2006.

LEGAL INFORMATION

CALCULATED R–PLAN PROVIDED BY J.D. BARNES LIMITED, PROJECT No. 07–10–820–00, SURVEY DATED MAY 30, 2012.

BENCH MARK No. 0011988U502

TOWNSHIP: STITTSVILLE

CONCRETE CULVERT ALONG HAZELDEAN ROAD, 1.3 KM NE OF ROAD INTERSECTION WITH MAIN ST NORTH. BRASS CAP SET ON TOP OF CONC CULVERT, SOUTH SIDE OF THE ROAD, 30 CM WEST OF EASTERLY EXTREMITY, 30 CM NORTH OF THE SOUTH FACE, SLIGHTLY BELOW ROAD LEVEL.

ELEVATION = 106.039 m

No.	BY	DATE	DESCRIPTION	BY
5	Z.L.	13–01–04	REVISED SANITARY BETWEEN MH's 105A–107A, 112A–106A, 115A–116A, 118A–119A	
4	Z.L.	12–05–31	4th SUBMISSION	
3	Z.L.	12–04–13	3rd SUBMISSION	
2	Z.L.	12–02–14	2nd SUBMISSION	
1	Z.L.	11–08–02	1st SUBMISSION	

Ottawa CITY OF OTTAWA

PROJECT No. 10–451

LICENSED PROFESSIONAL ENGINEER Z. LI 13-01-04 PROVINCE OF ONTARIO

SANITARY DRAINAGE PLAN

MATTAMY (FAIRWINDS NORTH) LIMITED

RECONSTRUCTION OF MAPLE GROVE ROAD

DSEL
david schaeffer engineering ltd
120 Iber Road, Unit 203
Stittsville, ON K2S 1E9
Tel. (613) 836-0856
Fax. (613) 836-7183
www.DSEL.ca

DRAWN BY: W.L./C.M.	CHECKED BY: K.M.	DRAWING NO.	SHEET NO.
DESIGNED BY: K.M.	CHECKED BY: Z.L.		6
SCALE: 1:3000	DATE: JULY 2011		

Appendix E

Background Documents
Storm

DESIGN BRIEF

FOR

POND 4
KANATA WEST

MATTAMY HOMES

CITY OF OTTAWA

PROJECT NO.: 12-644

AUGUST 9, 2013

REVISED DECEMBER 10, 2014

4TH SUBMISSION

DESIGN BRIEF

FOR
POND 4

KANATA WEST

MATTAMY HOMES

PROJECT NO: 12-644

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Background: Rationale for Report Update

*This report is an update of the August 2014 "**Design Brief for Pond 4, Kanata West, Mattamy Homes**". The previous version of this report has been updated to address City comments. Some minor changes have been made to the storm sewer data for the proposed Fairwinds West and Poole Creek Village subdivisions.*

DESIGN BRIEF

FOR
POND 4

KANATA WEST

MATTAMY HOMES

CITY OF OTTAWA

PROJECT NO.: 12-644

1.0 INTRODUCTION

This design brief is submitted in support of Pond 4, located in the Kanata West Community. Pond 4 discharges to the Carp River just upstream of the Carp River Palladium Drive crossing, and downstream of the Poole Creek confluence with the Carp River, as depicted in ***Figure 1***. Pond 4 will service a mixed-use area serviced by north and south trunk sewers and bound by Poole Creek, Palladium Drive and the Kanata West Community Boundary, as depicted in ***Figure 3***.

Design parameters for Pond 4 are contained in the ***Kanata West Master Servicing Study (KWMSS)*** completed by Stantec Consulting Ltd. on June 16, 2006.

As set out in the ***KWMSS***, Pond 4 is intended to satisfy various stormwater management requirements, including quality control and quantity control. The criteria are based on the ***Carp River Watershed / Subwatershed Study*** completed by Robinson Consultants in November 2004. A diversion pipe from the south trunk sewer to Poole Creek through the Fairwinds West and Poole Creek developments is also required to respect the limited capacity of the partially-installed downstream south trunk sewer and to compensate for the lack of a safe overland flow route for major system flows to Poole Creek.

The current Pond 4 design is to be constructed in two stages – interim and ultimate conditions. Areas to be developed under interim conditions are identified in ***Figure 2***, and areas to be developed under ultimate conditions are identified in ***Figure 3***. Note that for the purposes of modelling interim conditions conservatively, it has been assumed that the post-development drainage area to the north trunk will also drain to Pond 4 under pre-development conditions.

This design brief is prepared to provide technical support for the detailed design of Pond 4, as well as to demonstrate conformance with the overall design requirements of the City of Ottawa, Ministry of the Environment, background studies and general industry practice. Modelling files submitted with this report are a consolidated model of the most recent applicable information from the existing / proposed Fairwinds Community (per DSEL / JFSA / Stantec), the proposed Poole Creek Village development (per IBI Group), and the ***KWMSS*** (per Stantec).

1.1 Existing Conditions

Pond 4 is within the Carp River Watershed and is subject to the regulations associated with the Mississippi Valley Conservation Authority (MVCA). Pond 4 is adjacent to the Carp River and outlets directly to it.

The existing ground elevation at the Pond 4 site varies between approximately 96.00 m at the west limits to 93.50 m at the east limits, near the Carp River.

There is an existing interim pond located on the south side of Maple Grove Road in the existing Fairwinds South Phase 1 development, which currently services existing residential development in the Fairwinds Community. Once Pond 4 is commissioned, the interim pond will be decommissioned.

1.2 Interim Conditions

Quality and some quantity control are provided by the partially-constructed Pond 4, discharging to the Carp River. Only the south trunk and drainage areas tributary to the south trunk will be constructed; areas tributary to the future north trunk sewer will remain undeveloped (refer to ***Figure 2***). Orifice and weir controls will be installed in the diversion pipe through Fairwinds West and Poole Creek Village to divert flows to Poole Creek. The interim pond design is presented in ***Drawing 3***. Details of the various pond components will follow in ***Section 6.0*** for interim conditions.

1.3 Ultimate Conditions

Quality and some quantity control are provided by the fully-constructed Pond 4, discharging to the Carp River. Both the south and north trunk sewers and all drainage areas tributary to Pond 4 will be constructed under these ultimate conditions (refer to ***Figure 3***). Orifice and weir controls will be installed in the diversion pipe through Fairwinds West and Poole Creek Village to divert flows to Poole Creek. Note that the south forebay will be relocated and the main cell expanded under ultimate conditions. The ultimate pond design is presented in ***Drawing 4***. Details of the various pond components will follow in ***Section 7.0*** for ultimate conditions. Note that the proposed ultimate pond design may change in the future once the Carp River model is finalized and at the appropriate development stage for the north trunk, and is presented in this study for reference rather than final approval.

1.4 Summary of Pre-consultation

The following provides a summary of the pre-consultation meetings:

1.4.1 City of Ottawa, July 15, 2013

A meeting occurred with staff from the City of Ottawa, Mattamy, Tartan, DSEL and JFSA to discuss moving forward with the Kanata West Pond 4 design, approval and construction.

1.5 Required Permits / Approvals

Pond 4 is subject to the following approvals:

1.5.1 City of Ottawa

The City of Ottawa is required to approve the engineering design drawings and reports for Pond 4. The City of Ottawa must review and sign off on the design and forward to the Ministry of the Environment (MOE) as a direct submission review.

1.5.2 Ontario Ministry of the Environment (MOE)

The MOE is required to review the engineering design and issue an Environmental Compliance Approval (ECA) for the Stormwater Management Facility. The following ECA was issued for the construction of Kanata West Pond 4 and the decommissioning of the existing interim pond on the south side of Maple Grove Road:

- ECA #4298-9Q6HQ3 issued on October 31, 2014

A copy of this ECA is enclosed in ***Appendix A*** for reference.

A permit to take water (PTTW) is required during the construction of Pond 4 and will be issued by the MOE. The following PTTW was issued for the construction of Kanata West Pond 4.

- PTTW #0263-9N6RE8 issued on October 10, 2014.

A copy of this ECA is enclosed in ***Appendix A*** for reference.

1.5.3 Mississippi Valley Conservation Authority (MVCA)

A permit is required for the new outlet from Pond 4 to the Carp River. The MVCA has been provided with a copy of the Pond 4 design. A permit application has been submitted and is under review at the MVCA.

A weir is to be installed at the outlet of the proposed diversion pipe to prevent outflow to Poole Creek during the 25 mm storm, maintain 2-year outflows from the diversion pipe below the pre-development flow of 1.38 m^3/s, and prevent 100-year flood levels on Poole Creek from backing up into the diversion pipe (including a reasonable freeboard), in accordance with the MVCA draft plan conditions for Fairwinds West and the associated July 2013 ***Impact of Outflows from Proposed Diversion Pipe on Poole Creek*** memo (refer to ***Appendix H***). The diversion pipe outfall design by IBI Group should also consider potential localized impacts on erosion and fluvial geomorphic processes in Poole Creek.

2.0 GUIDELINES, PREVIOUS STUDIES, AND REPORTS

2.1 Existing Studies, Guidelines, and Reports

The following studies were utilized in the preparation of this report.

- ***Mississippi Valley Flood Plain Mapping Study***
  Cumming-Cockburn and Associates Limited, December 1983

- ***Stormwater Planning and Design Manual***
  Ministry of the Environment, March 2003
  *(**SWMP Design Manual**)*

- ***City of Ottawa Official Plan***
  City of Ottawa, adopted by Council 2003

- ***Carp River Watershed / Subwatershed Study***
  Robinson Consultants, November 2004

- ***Kanata West Master Servicing Study***
  Stantec Consulting Ltd, June 16, 2006
  *(**KWMSS**)*

- ***Existing Conditions HEC-RAS Model of Poole Creek***
  Greenland International Consulting Limited, March 2009

- ***Carp River Model Calibration Validation Exercise – Final Report***
  Greenland International Consulting Limited, July 2011

- ***Design Brief for the Reconstruction of Maple Grove Road***
  David Schaeffer Engineering Ltd, May 31, 2012

- ***Ottawa Sewer Design Guidelines***
  City of Ottawa, October 2012

- ***Impact of Outflows from Proposed Diversion Pipe on Poole Creek***
  J.F. Sabourin and Associates Inc., July 2013

- ***Poole Creek Regulation Mapping Study***
  MVCA, October 2013

- ***Stormwater Management Report for the Fairwinds West Subdivision***
  J.F. Sabourin and Associates Inc., August 2014

2.2 Findings of the Kanata West Master Servicing Study

The ***KWMSS*** established the stormwater control criteria, the pond location and the general stormwater management scheme. Pond 4, discharging to the Carp River, is to be designed with the following characteristics:

1. Quality control will be provided by a permanent pool in accordance with MOE Level 2 – Normal protection (70% TSS removal).

2. Quantity control: Post-development peak flows not to exceed pre-development levels for all storms up to the 10-year event.

3. A sediment forebay shall be provided.

4. Emergency overflow conveyance will be provided to safely pass emergency overflows.

Note that while a normal level of protection will be provided by Pond 4 prior to discharge to the Carp River in accordance with the ***KWMSS***, enhanced protection (80% TSS removal) is required for those flows discharging to Poole Creek via the diversion pipe. This is provided by a weir control installed at the diversion pipe outlet to direct the full "first flush" flows - in this case, the 25 mm storm flows - to Pond 4 for treatment. This approach is supported by MVCA as per the correspondence presented in ***Appendix H***.

Additionally, as requested by MVCA and in accordance with ***KWMSS*** requirements, baseflow augmentation will be provided by a 200 mm diameter circular vertical orifice controlling the first 0.2 m of active storage volume (greater than or equal to 10% of the 100-year active storage). A summary of the required Pond 4 characteristics is provided in ***Table 2***.

2.3 Proposed Deviations from the Master Servicing Study

The Pond 4 design contains deviations from the ***KWMSS***. Firstly, an interim (partially-constructed) pond was introduced to support the development of the drainage areas to the south trunk sewer, prior to construction of the north trunk sewer and developments. The size of the ultimate conditions (fully constructed) pond, servicing the north and south developments, was increased to account for the recently updated October 2012 *City of Ottawa Sewer Design Guidelines*, wherein those development lands serviced by the north trunk under ultimate conditions are to have 5-year minor system capture rates, and 10-year capture rates on arterial roads (contrary to the more restrictive capture rates specified in the ***KWMSS***). A summary of deviations in both interim and ultimate conditions inlet pipe dimensions and flows is presented in ***Table 1A***.

Table 1A
KWMSS Deviations – Inlet Pipe Dimensions and Flows

Item	KWMSS	Current Design Interim	Current Design Ultimate
South Trunk Inlet Pipe Dimensions	2550 mm @ 0.3% slope	2550 mm @ 0.3% slope	2550 mm @ 0.3% slope
North Trunk Inlet Pipe Dimensions	2250 mm @ 0.4% slope	N/A	To be resized at a future design stage
South Trunk Inlet Pipe Downstream Invert	91.26 m	91.503 m	91.777 m
North Trunk Inlet Pipe Downstream Invert	91.22 m	N/A	To be resized at a future design stage
10-Year, 12-hour SCS			
South Trunk Inflow to Pond	12.372 m^3/s	11.804 m^3/s	11.804 m^3/s
North Trunk Inflow to Pond	7.337 m^3/s	N/A	10.734 m^3/s
Major Inflow to Pond	1.682 m^3/s	4.965 m^3/s	0.520 m^3/s
100-Year, 12-hour SCS			
South Trunk Inflow to Pond	14.320 m^3/s	15.245 m^3/s	15.245 m^3/s
North Trunk Inflow to Pond	7.680 m^3/s	N/A	16.631 m^3/s
Major Inflow to Pond	3.570 m^3/s	8.424 m^3/s	0.760 m^3/s

The ***KWMSS*** Pond 4 outlet controls consist of a 350 mm diameter quality control orifice at an invert of 93.20 m, and a 30 m long broad-crested quantity control weir at an invert of 94.20 m. Although the ***KWMSS*** specifies a requirement for a baseflow augmentation volume equal to or greater than 10% of the 100-year active storage volume, the ***KWMSS*** Pond 4 design does not include a dedicated baseflow augmentation orifice. The Pond 4 design was therefore revised at the request of MVCA to include a 200 mm diameter baseflow augmentation orifice at an invert of 93.20 m, and to raise the 350 mm diameter quality control orifice by 0.20 m to an invert of 93.40 m. Furthermore, the top of a 9 m x 3 m drop inlet structure set at 94.20 m will act as a quantity control weir for the 2- to 10-year events in the updated pond design, and the 30 m long quantity control weir will be raised by 0.40 m to an invert of 94.60 m in accordance with the City's request that flow only spill over the 30 m long quantity control weir for events exceeding the 10-year level.

The proposed interim and ultimate conditions pond designs respect the ***KWMSS*** 100-year pond level of 94.74 m, and the ***KWMSS*** 10- and 100-year pond outflows of 17.282 m^3/s and 20.015 m^3/s, respectively. A summary of the deviations in both interim and ultimate conditions Pond 4 operating conditions under restrictive downstream conditions is presented in ***Table 1B***.

Table 1B
KWMSS Deviations – Pond Operating Conditions

Item	KWMSS	Current Design Interim	Current Design Ultimate
Drainage Area	267.97 ha	278.288 ha	278.288 ha
Imperviousness	59%	37%	62%
Permanent Pool	20,187m^3	14,471m^3 required 29,736m^3 provided	22,078m^3 required 53,815m^3 provided
Extended Detention	10,719m^3	11,132m^3 required 22,288m^3 provided	11,132m^3 required 43,005m^3 provided
10-Year, 12-hour SCS			
Pond Level	94.69 m	94.584 m	94.552 m
Flow Augmentation Outflow	N/A	0.039 m^3/s	0.037 m^3/s
Quality Orifice Outflow	0.208 m^3/s	0.121 m^3/s	0.111 m^3/s
Quantity Weir 1 Outflow	N/A	10.293 m^3/s	9.044 m^3/s
Quantity Weir 2 Outflow	17.074 m^3/s	0 m^3/s	0 m^3/s
100-Year, 12-hour SCS			
Pond Level	94.74 m	94.714 m	94.739 m
Flow Augmentation Outflow	N/A	0.047 m^3/s	0.049 m^3/s
Quality Orifice Outflow	0.218 m^3/s	0.155 m^3/s	0.159 m^3/s
Quantity Weir 1 Outflow	N/A	15.937 m^3/s	17.114 m^3/s
Quantity Weir 2 Outflow	19.797 m^3/s	1.916 m^3/s	2.585 m^3/s

A comparison of 10- and 100-year total outflow hydrographs from Pond 4 is presented in ***Appendix B***, including Pond 4 outflows as per the ***KWMSS***; pond outflows used in the July 2011 ***Carp River Model Calibration Validation Exercise – Final Report*** by Greenland; and the proposed pond outflows under interim and ultimate conditions. As may be seen in ***Appendix B***, the shape and timing of the outflow hydrographs are generally consistent between the ***KWMSS***, the July 2011 validated model, and interim and ultimate conditions. Furthermore, the peak interim and ultimate conditions flows are consistently lower than the ***KWMSS*** and the July 2011 validated model.

3.0 DRAINAGE ANALYSIS

Pond 4 has been designed to operate under interim conditions and under ultimate conditions. The pond design characteristics and requirements under interim conditions and ultimate conditions are presented in ***Table 2*** and ***Table 3***, respectively.

Table 2
Interim Conditions - Pond Design Characteristics

Item	Approximated Design Criteria	Comments
Drainage Area	278.288 ha	Including pond block; Refer to ***Figure 2***
Imperviousness	37%	
Required Permanent Pool Volume	14,471 m^3	Based on 52 m^3/ha[1]
Required Quality Control Volume	11,132 m^3	Based on 40 m^3/ha
Allowable Release Rates	10-Year: 17.282 m^3/s 100-Year: 20.015 m^3/s	To match the simulated ***KWMSS*** release rates

(1) Interpolated for 37% imperviousness. For normal protection level for wet pond, as per Table 3.2 of the ***SWMP Design Manual***.

Table 3
Ultimate Conditions - Pond Design Characteristics

Item	Approximated Design Criteria	Comments
Drainage Area	278.288 ha	Including pond block; Refer to ***Figure 3***
Imperviousness	62%	
Required Permanent Pool Volume	22,078 m^3	Based on 79.33 m^3/ha [1]
Required Quality Control Volume	11,132 m^3	Based on 40 m^3/ha
Allowable Release Rates	10-Year: 17.282 m^3/s 100-Year: 20.015 m^3/s	To match the simulated ***KWMSS*** release rates

(1) Interpolated for 62% imperviousness. For normal protection level for wet pond, as per Table 3.2 of the ***SWMP Design Manual***.

Furthermore, the design of the facility has been completed in general conformance with the ***City Guidelines*** and the ***SWMP Design Manual***.

4.0 SUBDIVISION DRAINAGE

An area of 278.288 ha draining to Pond 4, as presented in ***Figures 2*** and ***3*** (under interim and ultimate conditions, respectively), will be serviced by a conventional storm sewer system. ***Appendix G*** presents major and minor system drainage plans for those areas that are existing or at the detailed design stage, including Bryanston Gate (which contains the proposed Hartin Street development), Fairwinds North, Fairwinds South, Fairwinds West, the Hydro Site, and

Poole Creek Village. All other proposed development lands are modelled as lumped drainage areas in accordance with the ***KWMSS***; greater detail of the proposed major and minor system drainage routes for these lands will be provided at the preliminary or detailed design stages for each individual development.

A Rational Method design sheet was prepared for ultimate conditions (refer to ***Appendix E***) in order to estimate minor system flows in the south trunk sewer based on the City of Ottawa IDF relationship and selected runoff coefficients.

The minor system and pond operation has been modelled using XPSWMM based on the peak inflows calculated with the DDSWMM and SWMHYMO programs. Note that the storm sewer design in the consolidated XPSWMM model is as provided by DSEL (south trunk and Fairwinds developments), IBI Group (Poole Creek Village) and Stantec (Fairwinds South Phase 1, and north trunk as per the June 2006 ***KWMSS***).

Note that while the DDSWMM program is most appropriate for use in modelling urban drainage, the undeveloped land draining to Pond 4 under interim conditions (106.31 ha future development to the north trunk) was instead modelled using SWMHYMO. Digital modelling files are attached, and DDSWMM and SWMHYMO input files and an XPSWMM model schematic are presented in ***Appendix A***. Additionally, note that a temporary 1800 mm x 1500 mm ditch inlet will be installed in the Maple Grove Road ditch just west of Poole Creek in order to capture the 100-year existing flows in the ditch that are cut off by the proposed access road, prior to the urbanization of Maple Grove Road.

The performance of Pond 4 was analyzed in XPSWMM under both free outfall and restrictive downstream conditions at the outlet of the facility for all storms, where restrictive downstream conditions are based on the 100-year flood level of 94.20 m (as per the ***Mississippi Valley Flood Plain Mapping Study***, Cumming-Cockburn and Associates Limited, December 1983). Note that as the peak on the Carp River is expected to pass after the peak in the pond, the conservative assumption has been made that the water levels in the pond and on the Carp River rise at the same rate under restrictive conditions. As such, all active storage volume above the permanent pool is still available for quantity control under restrictive conditions, but no pond outflow is possible until the water level in the pond exceeds the maximum level on the Carp River (i.e. 94.20 m). The restrictive downstream condition of 94.20 m at Pond 4 is incorporated into the stage-storage-discharge curve, as per Table C-6 of ***Appendix C*** and D-6 of ***Appendix D***.

The DDSWMM / SWMHYMO / XPSWMM models have been used to verify that the target release rates for the 10- and 100-year storms are achieved, and that the storage volumes provided are adequate based on the detailed pond design. Simulated peak inflows, release rates, pond levels and storage volumes for the 25 mm 3-hour Chicago design storm, the 2- to 100-year 12-hour SCS design storms, and the July 1st, 1979, August 4th 1988 and August 8th 1996 historical event are presented in ***Tables 4*** and ***5*** for free outfall and restrictive downstream conditions, respectively.

Table 4A
SWM Pond Inflow, Outflow and Storage Summary
(Interim, Free Outfall Conditions)

Event	Minor Inflow (m^3/s)	Major Inflow (m^3/s)	Total Inflow (1) (m^3/s)	Pond Outflow (m^3/s)	Pond Level (m)	Storage Used (2) (m^3)
25 mm 3-Hour Chicago	6.343	1.365	7.708	0.669	94.240	23301
2-Year, 12-Hour SCS	6.857	2.459	9.316	3.940	94.391	27305
5-Year, 12-Hour SCS	10.107	3.929	14.037	8.161	94.520	30822
10-Year, 12-Hour SCS	11.804	4.965	16.770	10.393	94.578	32451
25-Year, 12-Hour SCS	13.582	6.333	19.915	13.032	94.633	33982
50-Year, 12-Hour SCS	14.466	7.454	21.920	15.508	94.674	35123
100-Year, 12-Hour SCS	15.245	8.424	23.669	17.859	94.709	36108
July 1st, 1979 Event	15.829	9.751	25.580	20.600	94.746	37169
August 4th, 1988 Event	15.517	8.268	23.785	19.137	94.726	36604
August 8th, 1996 Event	13.147	5.817	18.963	12.860	94.630	33894

(1) Taken as a direct summation of major and minor flows to the SWM Facility.
(2) Active storage volume only.

Table 4B
SWM Pond Inflow, Outflow and Storage Summary
(Ultimate, Free Outfall Conditions)

Event	Minor Inflow (m^3/s)		Major Inflow (m^3/s)	Total Inflow (1) (m^3/s)	Pond Outflow (m^3/s)	Pond Level (m)	Storage Used (2) (m^3)
	South Trunk	North Trunk					
25 mm 3-Hour Chicago	6.343	5.198	0.472	12.012	0.252	94.075	37331
2-Year, 12-Hour SCS	6.857	6.323	0.308	13.488	1.762	94.304	48189
5-Year, 12-Hour SCS	10.107	8.859	0.441	19.407	6.237	94.465	56369
10-Year, 12-Hour SCS	11.804	10.734	0.520	23.058	9.173	94.547	60618
25-Year, 12-Hour SCS	13.582	13.573	0.618	27.772	13.245	94.637	65386
50-Year, 12-Hour SCS	14.466	15.588	0.695	30.749	16.565	94.690	68147
100-Year, 12-Hour SCS	15.245	16.631	0.760	32.636	19.667	94.733	70436
July 1st, 1979 Event	15.829	18.050	0.954	34.833	23.540	94.783	73074
August 4th, 1988 Event	15.517	17.196	1.029	33.741	22.457	94.769	72358
August 8th, 1996 Event	13.147	11.511	0.918	25.575	15.182	94.669	67054

(1) Taken as a direct summation of major and minor flows to the SWM Facility.
(2) Active storage volume only.

Table 5A
SWM Pond Inflow, Outflow and Storage Summary
(Interim, Restrictive Downstream Conditions)

Event	Minor Inflow (m^3/s)	Major Inflow (m^3/s)	Total Inflow (1) (m^3/s)	Pond Outflow (m^3/s)	Pond Level (m)	Storage Used (2) (m^3)
25 mm 3-Hour Chicago	6.343	1.365	7.708	0.837	94.268	24046
2-Year, 12-Hour SCS	6.857	2.459	9.316	4.031	94.403	27614
5-Year, 12-Hour SCS	10.107	3.929	14.037	8.227	94.527	31022
10-Year, 12-Hour SCS	11.804	4.965	16.770	10.453	94.584	32627
25-Year, 12-Hour SCS	13.582	6.333	19.915	13.119	94.638	34116
50-Year, 12-Hour SCS	14.466	7.454	21.920	15.687	94.679	35275
100-Year, 12-Hour SCS	15.245	8.424	23.669	18.055	94.714	36248
July 1st, 1979 Event	15.829	9.751	25.580	20.623	94.748	37243
August 4th, 1988 Event	15.517	8.268	23.785	19.245	94.730	36710
August 8th, 1996 Event	13.147	5.817	18.963	12.943	94.634	34026

(1) Taken as a direct summation of major and minor flows to the SWM Facility.
(2) Active storage volume only.

Table 5B
SWM Pond Inflow, Outflow and Storage Summary
(Ultimate, Restrictive Downstream Conditions)

Event	Minor Inflow (m^3/s)		Major Inflow (m^3/s)	Total Inflow (1) (m^3/s)	Pond Outflow (m^3/s)	Pond Level (m)	Storage Used (2) (m^3)
	South Trunk	North Trunk					
25 mm 3-Hour Chicago	6.343	5.198	0.472	12.012	0.000	94.190	42537
2-Year, 12-Hour SCS	6.857	6.323	0.308	13.488	1.821	94.318	48861
5-Year, 12-Hour SCS	10.107	8.859	0.441	19.407	6.294	94.473	56778
10-Year, 12-Hour SCS	11.804	10.734	0.520	23.058	9.192	94.552	60907
25-Year, 12-Hour SCS	13.582	13.573	0.618	27.772	13.390	94.643	65704
50-Year, 12-Hour SCS	14.466	15.588	0.695	30.749	16.745	94.695	68420
100-Year, 12-Hour SCS	15.245	16.631	0.760	32.636	19.907	94.739	70723
July 1st, 1979 Event	15.829	18.050	0.954	34.833	23.620	94.786	73235
August 4th, 1988 Event	15.517	17.196	1.029	33.741	22.614	94.773	72566
August 8th, 1996 Event	13.147	11.511	0.918	25.575	15.286	94.673	67271

(1) Taken as a direct summation of major and minor flows to the SWM Facility.
(2) Active storage volume only.

Based on the DDSWMM / SWMHYMO / XPSWMM models, the peak 10- and 100-year inflows to the SWM facility are approximately 16.770 m^3/s and 23.669 m^3/s, respectively, under interim conditions, and approximately 23.058 m^3/s and 32.636 m^3/s, respectively, under ultimate conditions.

4.1 Minor and Major System Drainage

In accordance with current City of Ottawa standards, it is understood that the north trunk sewer will be designed to provide a 5-year level of service (10-year on arterial roads) based on City of Ottawa IDF curves. Conversely, the partially installed south trunk provides a capacity of 85 L/s/ha, in accordance with now outdated City of Ottawa Guidelines. A 5-year level of service is provided to Bryanston Gate (which contains the proposed Hartin Street development), Fairwinds North, Fairwinds South, Fairwinds West, the Hydro Site, the future Claridge development (A-1), and Poole Creek Village, which are all serviced by the south trunk sewer. Note that excess 100-year flows in Fairwinds North are stored on-site to avoid crossing Maple Grove Road, and the 100-year flows on Maple Grove Road west of the future north-south arterial are captured to the minor system to compensate for the lack of a safe overland flow route. Future developments draining to the south trunk east of Huntmar Drive are limited to the 85 L/s/ha capacity of the south trunk sewer per the ***KWMSS***.

DDSWMM drainage area characteristics for those future development areas draining to the north trunk or to the south trunk east of Huntmar Drive are as per the ***KWMSS***. Poole Creek Village is as modelled by IBI Group, with inflow hydrographs and storm sewer data provided to DSEL / JFSA on April 11th and 14th, 2014 (including the design of the diversion pipe within Poole Creek Village and the outfall to Poole Creek). The DDSWMM length parameter for Bryanston Gate, Maple Grove Road and the Fairwinds Community was measured along the centreline of the road for street sub-catchments, and along the centreline of the drainage swale or path for other subcatchments. The corresponding DDSWMM width parameter was set as equal to or twice the measured length, for catchments that drain to the road/swale from one or two directions, respectively.

The DDSWMM model depression storage and infiltration parameters are as per the October 2012 City of Ottawa Sewer Design Guidelines. The percent imperviousness values of lumped and future drainage areas were calculated based on the runoff coefficient (C) provided by DSEL or from the June 2006 KWMSS, where C = 0.7 x imperviousness ratio + 0.2. Percent imperviousness values of detailed drainage areas within the Fairwinds Community and on Maple Grove Road were calculated based on the actual development layout; impervious areas were taken as fully effective in the front and half effective in the rear of each lot to account for indirectly connected roof drainage.

The 105.83 ha natural area draining to Pond 4 under interim conditions (future development to the north trunk sewer) was modelled in SWMHYMO with an SCS Curve Number of 84 based on an average of those used in the March 2009 existing conditions Carp River XPSWMM model by Greenland Engineering for areas 115P, 116P and 206F. A time to peak of 0.76 hours was calculated using distance and elevations from Google Earth, where time to peak is equal to two-thirds of the Bransby-Williams 85/10 time of concentration.

4.2 Diversion Pipe to Poole Creek

A diversion pipe to Poole Creek through the Fairwinds West and Tartan lands is proposed in order to respect the capacity of the downstream south trunk sewer while still providing a 5-year level of service to the upstream Claridge, Bryanston Gate, Fairwinds West and Poole Creek Village developments. The excess 100-year flows from these developments are also captured to the minor system upstream of the diversion pipe in order to compensate for the lack of a safe overland flow route for major system flows to Poole Creek.

Outflows from the diversion pipe, as shown in ***Figures 2*** and ***3***, are controlled by:

i) A 0.85 m diameter orifice at an invert of 98.33 m at the Maple Grove Road trunk sewer outlet of MH 103, to divert flows from the south trunk to Poole Creek via the diversion pipe.

ii) A 12 m long weir at an invert of 100.29 m at the outlet of the diversion pipe, as designed by IBI Group. The purposed of the weir is to prevent outflow to Poole Creek during the 25 mm storm, maintain 2-year outflows from the diversion pipe below the pre-development flow of 1.38 m^3/s, and prevent 100-year flood levels on Poole Creek from backing up into the diversion pipe (including a reasonable freeboard), in accordance with the MVCA draft plan conditions for Fairwinds West and the associated July 2013 ***Impact of Outflows from Proposed Diversion Pipe on Poole Creek*** memo (refer to ***Appendix H***).

iii) A 0.818 m diameter orifice at an invert of 98.466 at the outlet of MH 131 (by IBI Group) to control outflows to Maple Grove Road from Poole Creek Village.

Controls (i) and (ii) were selected to allow the greatest amount of flow to be directed to Pond 4 without having a negative impact on the existing downstream developments under any considered design storm, historical event or climate change stress test.

Further to item (ii) and as noted in the correspondence presented in ***Appendix H***, the 80% TSS removal required by the Ministry of the Environment for an enhanced level of protection for Poole Creek at the diversion pipe outfall is not a target to be met for every storm, but a long-term statistical average. Therefore, some storm events, and in particular large rainfall events, will not have 80% TSS removal; this is balanced by the TSS removal for smaller, more frequent storm events. Nonetheless, as shown in ***Table 6***, less than 20% of the rainfall volume for a 100-year design storm will discharge to Poole Creek via the diversion pipe.

Table 6
Runoff Volumes Discharging to Poole Creek from the Diversion Pipe

Volume	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Total Outflow (m^3) (1)	0	2414	5697	10295	13853	17033
Total Outflow (mm) (2)	0.0	2.6	6.2	11.2	15.0	18.5
Rainfall (mm)	41.7	49.6	66.8	78.6	88.0	96.0
Outflow/Rainfall (%)	0.0	5.3	9.2	14.2	17.1	19.2

(1) Based on the 12-hour SCS Type II Design Storm.
(2) Based on 92.20 ha upstream drainage area (Bryanston Gate, Fringewood North, Claridge, Fairwinds West, and Poole Creek Village).

To validate these results in relation to historical rainfall records, a review of the City of Ottawa Guidelines and the Ottawa International Airport rainfall data (1967-2003) shows seven "significant" rainfall events with maximum return periods greater than a 10-year; one greater than 10-year, three greater than 25-year, and three greater than 100-year (based on varying durations). Furthermore, more than 80% of average annual rainfall occurs during events of less than 25 mm. As such, based on the retention of runoff from the 25 mm design storm and the supporting data presented in ***Table 6*** for less frequent events, 80% TSS removal is provided by the proposed diversion pipe design on an average annual basis. These results are similar to that of a conventional stormwater management pond.

It is expected that erosive conditions in Poole Creek downstream of the diversion pipe will improve under proposed conditions as a result of the diversion of the 25 mm storm runoff and the majority of the 2- to 100-year storm runoff volumes to Pond 4.

4.3 Assumptions and Sources of Data Used

The following parameters and assumptions used in the analysis are based on City of Ottawa standards and generally accepted stormwater management design guidelines.

- SWM Model: *DDSWMM (v. 2.1), SWMHYMO (v. 5.02) and XPSWMM (v. 10)*
- Minor System Design: *1:5 year*; *1:10 year on arterial roads*
- Major System Design: *1:100 year*
- Max. Allowable Flow Depth: *30 cm above gutter*
- Extent of Major System: *Must be contained within the municipal right-of-way*.
- Model Parameters: *Fo = 76.2 mm/hr, Fc = 13.2 mm/hr, DCAY = 4.14/hr, D.Stor.Imp. = 1.57 mm, D.Stor.Per. = 4.67 mm (as per October 2012 City of Ottawa Guidelines).*
- Imperviousness: *Detailed Area Imperviousness: based on development layout and taken and fully effective in the front lot portion and half effective in rear lot portion of each house.*
  Lumped Area Imperviousness: Based on runoff coefficient (C) where Percent Imperviousness = (C - 0.2) / 0.7 x 100%.
- Design Storms: *3-hour Chicago and 12-hour SCS Type II Design Storms. Maximum intensity averaged over 10 minutes. Based on October 2012 City of Ottawa Sewer Design Guidelines.*
- Historical Events: *July 1st, 1979, August 4th, 1988 and August 8th, 1996 historical events per October 2012 City of Ottawa Sewer Design Guidelines.*
- Manning's Roughness: *0.013 for concrete pipes (free flow)*.
- Minor System Losses: *Minor losses were entered as exit losses at the end of pipes entering manholes, based on the angle between that inlet pipe and the outlet pipe.*
- Freeboard in HGL analysis: *0.3 m between underside of footing elevation and 100-year hydraulic gradeline in the Fairwinds Community; match June 2006 KWMSS 100-year HGL elevations at other nodes.*
- Downstream HGL: *100-year Carp River floodplain elevation of 94.2 m as per the Mississippi Valley Flood Plain Mapping Study (Cumming-Cockburn and Associates Limited, December 1983).*

4.4 Major System Conveyance

As per current City standards, the total 100-year depth of water (static and dynamic) on the street must be retained within the right-of-way and should not exceed 30 cm. For the Fairwinds Community, conformance with this standard is demonstrated in the following reports:

i) ***Mattamy Homes Fairwinds Subdivision Stormwater Management Report and Temporary Stormwater Management Facility Design Brief*** (Stantec, October 2006).

ii) ***Stormwater Management Report for Fairwinds Phase 2 of the Mattamy Homes Subdivisions*** (JFSA, May 2007).

iii) ***Stormwater Management Report for Fairwinds Phase 3 of the Mattamy Homes Subdivisions*** (JFSA, May 2007).

iv) ***Stormwater Management Report for Phases 2B, 3 and 4 of Fairwinds North of the Mattamy Homes Subdivision*** (JFSA, April 2012).

v) ***Stormwater Management Report for the Fairwinds West Subdivision*** (JFSA, August 2014).

Water depths within the existing Bryanston Gate subdivision may exceed 30 cm, but identifying or addressing existing issues within this subdivision are not within the scope of this analysis. The 100-year flows on Maple Grove Road west of the future North-South Arterial are captured to the minor system, and therefore the depth of water on the street will not exceed 30 cm. Water depth within Poole Creek Village is to be addressed by IBI Group.

The depth of water on the street in the remaining future development areas will be assessed at the preliminary or detailed design stages for each individual development. No action is proposed to address water depths under the climate change stress test in future developments at this time, as information available at the detailed design stage is required to properly define and address any issues that may occur.

4.5 Hydraulic Gradeline Analysis

The following parameters and assumptions used in the minor system and hydraulic gradeline analysis was completed for the proposed system using the XPSWMM program based on the 100-year 3-hour Chicago and 12-hour SCS design storms, and for the July 1st 1979, August 4th 1988 and August 8th 1996 historical events. The minor system performance was analysed for both free outfall and restrictive downstream conditions, where restrictive conditions were modelled based on the follow static 100-year flood levels:

i) 99.99 m at the diversion pipe outfall to Poole Creek as per the in progress update to the October 2013 ***Poole Creek Regulation Mapping Study*** by MVCA.

ii) 96.69 m at the existing Fairwinds South outfall to Poole Creek as per ***Existing Conditions HEC-RAS Model of Poole Creek***, provided by Greenland International Consulting Limited, March 2009. Note that the March 2009 value was used as it is more conservative than the 96.31 m flood level reported in the October 2013 ***Poole Creek Regulation Mapping Study*** by MVCA.

iii) 94.20 m at the Pond 4 outfall to Carp River as per the June 2006 KWMSS (based on ***Mississippi Valley Flood Plain Mapping Study***, Cumming Cockburn and Associates Limited, December 1983). Note that this Regulatory flood level is less than the 94.37 m 100-year existing water level reported in the July 2011 ***Carp River Model Calibration Validation Exercise – Final Report*** by Greenland. The sensitivity analysis below has confirmed that a 100-year flood level of 94.37 m will not negatively impact the 100-year hydraulic gradeline results in the upstream Fairwinds Community; however, the Regulatory flood level of 94.20 m will still be used in the Pond 4 XPSWMM model as the we understand that the Carp River flood mapping exercise is still on-going.

Refer to ***Appendix A*** for an XPSWMM schematic model and to ***Appendix E*** for the composite (worst-case) hydraulic gradeline results under interim and ultimate conditions.

The 100-year flow will surcharge most parts of the minor system; however, a freeboard of 0.3 m between the hydraulic gradeline and the underside of footing has been provided throughout the Fairwinds Community developments. Freeboard provided within the Poole Creek Village development is to be assessed separately by IBI Group based on the consolidated model.

KWMSS Conformance

The north trunk sewer is to be resized to meet hydraulic gradeline requirements based on City requirements for 5-year capture (10-year on arterial roads), rather than the ***KWMSS*** 85 L/s/ha trunk sewer sizing. It will be confirmed that a freeboard of 0.3 m between the hydraulic gradeline and the underside of footing elevations is provided in future developments to the north and south trunks at the preliminary or detailed design stages for each individual development.

The hydraulic gradeline elevations simulated at the south trunk ***KWMSS*** nodes N4A01 (Claridge Homes) and N4A02 (Bryanston Gate) are less than the ***KWMSS*** values of 105.86 m and 105.65 m, respectively, for all considered events. Furthermore, the KWMSS hydraulic gradeline elevation of 105.86 m at Bryanston Gate is more than 0.3 m below the lowest existing underside of footing elevation of 106.21 m in the vicinity (as per the Bryanston Gate Subdivision Grading Plan shown in ***Appendix I***).

Based on the 100-year flow generated in the existing Bryanston Gate subdivision, and the plan and profile of the “pre-development” outlet pipes along Maple Grove Road (i.e. prior to the extension of the Maple Grove Road trunk sewer) presented in ***Appendix I***, it was determined in XPSWMM that the 100-year flow would have surcharged above the ground level of 107.50 m under these “pre-development conditions”. This analysis was undertaken at the City’s request to demonstrate that the proposed interim and ultimate conditions offer an improvement to the hydraulic gradeline in the existing Bryanston Gate subdivision.

Sediment Accumulation and Climate Change Sensitivity Tests

Several tests were undertaken to assess the hydraulic gradeline results under potential future conditions. Firstly, hydraulic gradeline elevations were simulated for the 100-year 3-hour Chicago storm and 100-year 12-hour SCS Type II storm, wherein the south trunk sewer inlet pipes to Pond 4 is 25% blocked by sediment accumulation in the portion of the sewer submerged by the Pond 4 permanent pool, as calculated in Table F-1 of ***Appendix F***. A freeboard of 0.3 m between the hydraulic grade line and the underside of footings is still provided throughout the Fairwinds Community under these conditions, as presented in Tables F-2 and F-3 of ***Appendix F***.

Tables F-4 and F-5 of ***Appendix F*** present the climate change stress test results for the hydraulic gradeline analysis based on a 20% increase in the 100-year storms, as per the October 2012 City of Ottawa Sewer Design Guidelines, as well as 25% blockage of the submerged portion of the inlet pipes to Pond 4 to represent a worst-case scenario. Under these conditions, seven lots in Fairwinds South have freeboards between 0 m and -0.23 m (i.e. between the underside of footing and basement floor elevation) as a result of limitations in the existing local sewer. Based on previous discussions with the City, we understand that no retroactive action is required since the deficiencies in freeboard may be attributed to the existing / approved drainage systems rather than to the impact of Pond 4.

Additionally, four lots in Fairwinds West have freeboards between 0 m and -0.23 m, and eight lots in Fairwinds West have freeboards of less than -0.23 m (i.e. above the basement floor elevation) during the 100-year + 20% stress test due to the orifice and weir controls installed for the diversion pipe to Poole Creek.

However, another climate change stress test scenario is presented in Tables F-6 and F-7 of ***Appendix F***, where the 100-year + 20% stress test is applied to all drainage areas except the existing Bryanston Gate subdivision. Per discussions with the City, the opportunity for an increased level of service up to the 100-year level is already provided to the existing Bryanston Gate subdivision as a result of the proposed Pond 4 system. As before, 25% blockage of the submerged portion of the inlet pipes to Pond 4 was also modelled under these conditions. Under this 100-year + 20% stress test, the hydraulic gradeline in the Fairwinds West subdivision is reduced such that only one lot has a freeboard of -0.003 m, and no other lots in Fairwinds West have a freeboard of less than 0 m or -0.23 m.

Stress test results within future developments will be addressed at the detailed design stage for each development.

Carp River Flood Level Sensitivity Test

The July 2011 ***Carp River Model Calibration Validation Exercise – Final Report*** by Greenland indicates a 100-year flood level at the Pond 4 outlet of 94.37 m under existing conditions and 94.02 m under future conditions. We understand that modelling of the Carp River is on-going, and as such neither of these flood levels are considered official. Furthermore, the future 100-year flood level of 94.02 m estimated in July 2011 is less than the Regulatory level of 94.20 m used to define restrictive downstream conditions in the present study. However, a sensitivity analysis has been undertaken to evaluate the potential performance of Pond 4 and the upstream storm sewer system based on the July 2011 existing 100-year flood level of 94.37 m, in order to demonstrate some factor of safety in the proposed design as it relates to restrictive downstream conditions.

For the 100-year 3-hour Chicago and 100-year 24-hour SCS Type II design storms, accounting for 25% sediment blockage of the submerged portions of the inlet pipes to Pond 4, a 0.3 m freeboard is still provided to all existing and proposed lots in the Fairwinds Community for a downstream Carp River 100-year flood level of 94.37 m.

Furthermore, for the 100-year + 20% 3-hour Chicago and 24-hour SCS Type II climate change stress tests, accounting for 25% sediment blockage of the submerged portions of the inlet pipes to Pond 4, the number of lots in the Fairwinds Community with a freeboard between 0 m and -0.23 m (i.e. between the underside of footing and basement floor elevations), and with a freeboard of less than -0.23 (i.e. above the basement floor elevations) remains consistent between the Regulatory flood level of 94.20 m and the July 2011 level of 94.37 m.

The 100-year pond level of 94.83 m under the July 2011 restrictive downstream conditions does not have a 0.3 m freeboard to the top of berm at 95.05 m. However, it should be noted that the maximum pond level of 94.87 m during the July 1st, 1979 historical event does not exceed the top of berm. Furthermore, the 10- and 100-year pond release rates do not exceed the allowable release rates of 17.282 m^3/s and 20.015 m^3/s, respectively, as defined by the July 2006 Kanata West Master Servicing Study.

It may therefore be concluded based on the results of this sensitivity analysis that although the proposed Pond 4 is designed based on the official Regulatory flood level of 94.20 m, it has regard for a potential flood level of 94.37 m nonetheless.

5.0 POND OPERATING CHARACTERISTICS

Pond 4 has been designed in accordance with the requirements of the ***KWMSS***, ***City of Ottawa Sewer Design Guidelines*** and the ***SWMP Design Manual***, and includes the following features (refer to ***Table 1***):

Sediment Forebay	➢ to improve sediment removal prior to entering the pond
Permanent Pool	➢ to buffer storm flows and trap pollutants
Quality Control	➢ to provide water quality control
Quantity Control Storage	➢ to attenuate post-development flows to the allowable release rates
Emergency Overflow Conveyance	➢ to provide for passage of emergency flows

As previously noted, the proposed pond also includes a baseflow augmentation component, as requested by MVCA. Details of the sizing of the pond components vary under interim and ultimate conditions, and are described in ***Section 6.0*** for interim conditions, and ***Section 7.0*** for ultimate conditions. The grading of the pond and the details are illustrated on the engineering drawings provided.

The operating characteristics of the pond are defined in the following sections, and are summarized below in ***Tables 7*** and ***8*** for free outfall and restrictive downstream conditions, respectively.

Table 7A
Summary of SWM Pond 4 Operating Characteristics
(Interim, Free Outfall Conditions)

Pond Component	Pond Inflow (1) (m^3/s)	Lower Elevation (m)	Upper Elevation (m)	Volume Used (2) (m^3)	Allowable Outflow (3) (m^3/s)	Pond Outflow (m^3/s)
Permanent Pool	N/A	90.700	93.200	29736	N/A	N/A
Quality Control	N/A	93.200	93.728	11132	N/A	0.064
Extended Detention	N/A	93.728	94.200	22288	N/A	0.279
25 mm 3-Hour Chicago	7.708	93.200	94.240	23301	N/A	0.669
2-Year, 12-Hour SCS	9.316	94.240	94.391	27305	N/A	3.940
5-Year, 12-Hour SCS	14.037	94.391	94.520	30822	N/A	8.161
10-Year, 12-Hour SCS	16.770	94.520	94.578	32451	17.282	10.393
25-Year, 12-Hour SCS	19.915	94.578	94.633	33982	N/A	13.032
50-Year, 12-Hour SCS	21.920	94.633	94.674	35123	N/A	15.508
100-Year, 12-Hour SCS	23.669	94.674	94.709	36108	20.015	17.859
July 1st, 1979 Event	25.580	94.709	94.746	37169	N/A	20.600
August 4th, 1988 Event	23.785	94.709	94.726	36604	N/A	19.137
August 8th, 1996 Event	18.963	94.709	94.630	33894	N/A	12.860

(1) Pond inflow taken as a direct summation of major and minor flows to the pond.
(2) Volumes used are active storage only for all pond components except the permanent pool.
(3) Based quantity control of 10- and 100-year release rates to match ***KWMSS***.

Table 7B
Summary of SWM Pond 4 Operating Characteristics
(Ultimate, Free Outfall Conditions)

Pond Component	Pond Inflow (1) (m^3/s)	Lower Elevation (m)	Upper Elevation (m)	Volume Used (2) (m^3)	Allowable Outflow (3) (m^3/s)	Pond Outflow (m^3/s)
Permanent Pool	N/A	90.700	93.200	53815	N/A	N/A
Quality Control	N/A	93.200	93.480	11132	N/A	0.056
Extended Detention	N/A	93.480	94.200	43005	N/A	0.279
25 mm 3-Hour Chicago	12.012	93.200	94.075	37331	N/A	0.252
2-Year, 12-Hour SCS	13.488	94.075	94.304	48189	N/A	1.762
5-Year, 12-Hour SCS	19.407	94.304	94.465	56369	N/A	6.237
10-Year, 12-Hour SCS	23.058	94.465	94.547	60618	17.282	9.173
25-Year, 12-Hour SCS	27.772	94.547	94.637	65386	N/A	13.245
50-Year, 12-Hour SCS	30.749	94.637	94.690	68147	N/A	16.565
100-Year, 12-Hour SCS	32.636	94.690	94.733	70436	20.015	19.667
July 1st, 1979 Event	34.833	94.733	94.783	73074	N/A	23.540
August 4th, 1988 Event	33.741	94.733	94.769	72358	N/A	22.457
August 8th, 1996 Event	25.575	94.733	94.669	67054	N/A	15.182

(1) Pond inflow taken as a direct summation of major and minor flows to the pond.
(2) Volumes used are active storage only for all pond components except the permanent pool.
(3) Based quantity control of 10- and 100-year release rates to match ***KWMSS***.

Table 8A
Summary of SWM Pond 4 Operating Characteristics
(Interim, Restrictive Downstream Conditions)

Pond Component	Pond Inflow (1) (m^3/s)	Lower Elevation (m)	Upper Elevation (m)	Volume Used (2) (m^3)	Allowable Outflow (3) (m^3/s)	Pond Outflow (m^3/s)
Permanent Pool	N/A	90.700	93.200	29736	N/A	N/A
Quality Control	N/A	93.200	93.728	11132	N/A	0.064
Extended Detention	N/A	93.728	94.200	22288	N/A	0.279
25 mm 3-Hour Chicago	7.708	93.200	94.268	24046	N/A	0.837
2-Year, 12-Hour SCS	9.316	94.268	94.403	27614	N/A	4.031
5-Year, 12-Hour SCS	14.037	94.403	94.527	31022	N/A	8.227
10-Year, 12-Hour SCS	16.770	94.527	94.584	32627	17.282	10.453
25-Year, 12-Hour SCS	19.915	94.584	94.638	34116	N/A	13.119
50-Year, 12-Hour SCS	21.920	94.638	94.679	35275	N/A	15.687
100-Year, 12-Hour SCS	23.669	94.679	94.714	36248	20.015	18.055
July 1st, 1979 Event	25.580	94.714	94.748	37243	N/A	20.623
August 4th, 1988 Event	23.785	94.714	94.730	36710	N/A	19.245
August 8th, 1996 Event	18.963	94.714	94.634	34026	N/A	12.943

(1) Pond inflow taken as a direct summation of major and minor flows to the pond.
(2) Volumes used are active storage only for all pond components except the permanent pool.
(3) Based quantity control of 10- and 100-year release rates to match ***KWMSS***.

Table 8B
Summary of SWM Pond 4 Operating Characteristics
(Ultimate, Restrictive Downstream Conditions)

Pond Component	Pond Inflow [(1)] (m^3/s)	Lower Elevation (m)	Upper Elevation (m)	Volume Used [(2)] (m^3)	Allowable Outflow [(3)] (m^3/s)	Pond Outflow (m^3/s)
Permanent Pool	N/A	90.700	93.200	53815	N/A	N/A
Quality Control	N/A	93.200	93.480	11132	N/A	0.056
Extended Detention	N/A	93.480	94.200	43005	N/A	0.279
25 mm 3-Hour Chicago	12.012	93.200	94.190	42537	N/A	0.000
2-Year, 12-Hour SCS	13.488	94.190	94.318	48861	N/A	1.821
5-Year, 12-Hour SCS	19.407	94.318	94.473	56778	N/A	6.294
10-Year, 12-Hour SCS	23.058	94.473	94.552	60907	17.282	9.192
25-Year, 12-Hour SCS	27.772	94.552	94.643	65704	N/A	13.390
50-Year, 12-Hour SCS	30.749	94.643	94.695	68420	N/A	16.745
100-Year, 12-Hour SCS	32.636	94.695	94.739	70723	20.015	19.907
July 1st, 1979 Event	34.833	94.739	94.786	73235	N/A	23.620
August 4th, 1988 Event	33.741	94.739	94.773	72566	N/A	22.614
August 8th, 1996 Event	25.575	94.739	94.673	67271	N/A	15.286

(1) Pond inflow taken as a direct summation of major and minor flows to the pond.
(2) Volumes used are active storage only for all pond components except the permanent pool.
(3) Based quantity control of 10- and 100-year release rates to match ***KWMSS***.

With the proposed controls described in the following sections, the above modelling results show that the actual provided 10- and 100-year release rates do not exceed the allowable release rates simulated in the ***KWMSS*** under interim and ultimate conditions.

The 100-year pond levels of 94.714 m and 94.739 m under interim and ultimate conditions, respectively, do not exceed the 94.74 m level simulated in the ***KWMSS***, and a 0.3 m freeboard above this pond level is provided to the top of berm around the pond at 95.05 m. Note that the maximum pond level is 94.786 m during the July 1st 1979 historical event under ultimate conditions. The elevation of the top of berm (95.05 m) is above this pond level, and thus the water is contained within the pond.

6.0 INTERIM POND COMPONENTS

Refer to the provided ***Drawings 3 and 5 to 8*** for detailed design drawings and cross-sections of Pond 4 and components under interim conditions.

6.1 Sediment Forebay

The proposed facility has been equipped with two sediment forebay(s) in order to improve the pollutant removal by allowing the larger particles to settle out prior to entering the main cell of the pond. Only the south forebay will receive flows requiring treatment under interim conditions, wherein only the south trunk sewer and areas tributary to it are developed. The south forebay has been designed with a length-to-width ratio of greater than 2:1 and does not exceed one-third of the permanent pool area, as required in the ***SWMP Design Manual***.

Pond 4 obtains flows through one 2550 mm diameter circular pipe at 0.3% slope to the south forebay. The forebay has been sized to meet the greater of the settling and dispersion criteria, as stated in the ***SWMP Design Manual***. Calculations for the minimum dispersion length, settling length and the average velocity have been included in Calculation Sheet C-2 of ***Appendix C*** for interim conditions. Note that the forebay does not quite meet average velocity requirements based on the average forebay width, as the width of the forebay has been set to meet City of Ottawa specifications for cleaning and maintenance, such that the access roads are no more than 15.0 m from the centre of the forebay.

The forebay has been provided with a permanent pool of 2.0 m depth to minimize the potential for re-suspension. In accordance with City of Ottawa criteria, the forebay has been graded at 5:1 above the permanent pool level. A permeable forebay berm has been set 0.30 m below the permanent pool water level.

6.2 Permanent Pool

In accordance with the ***SWMP Design Manual***, the pond should have a permanent pool depth between 1.0 and 3.0 metres. The proposed facility has been designed with a permanent pool depth of 2.5 m.

The permanent pool has been sized in accordance with the requirements of the ***SWMP Design Manual***, Table 3.2, normal protection level for wet pond, 37% imperviousness, as follows:

$$(92.00\ m^3/ha - 40\ m^3/ha) \times 278.288\ ha = 14{,}471\ m^3$$

The proposed facility has a permanent pool volume of 29,736 m^3 under interim conditions.

The slopes in the permanent pool will be graded with side slopes of 4:1, with minor localized variations. As per the City of Ottawa design criteria, the side slopes adjacent to the maintenance access road have been graded at 5:1 maximum. In general, and in accordance with the City of Ottawa criteria, the grading above and below the permanent pool elevation has been graded at 5:1 in order to address public safety concerns. The proposed pond grading is shown on ***Drawing 3***.

6.3 Baseflow Augmentation

As requested by MVCA, a vertical circular 200 mm diameter baseflow augmentation orifice at an invert of 93.20 m is provided to control the first 0.2 m of active storage to a drawdown time of 2.4 days under interim conditions. The calculation of baseflow augmentation drawdown time is provided in Tables C-3 and C-4 of ***Appendix C*** for interim conditions

The baseflow augmentation volumes provided are greater than or equal to 10% of the 100-year active storage volume, where:

$$4044\ m^3 \text{ baseflow augmentation} : 3625\ m^3 \text{ 10\% of 100-year active storage}$$

This is consistent with the conceptual design of the ***KWMSS***.

6.4 Quality Control

The extended detention storage has been sized based on 40 m^3/ha in accordance with the ***SWMP Design Manual*** requirements. If the required quality control volume was based on the entire 278.288 ha drainage area to be conservative, it would be calculated as follows:

$$40\ m^3/ha \times 278.288\ ha = 11{,}132\ m^3$$

This required quality control volume is contained within the extended detention volume under interim conditions.

6.5 Extended Detention

A vertical circular 350 mm diameter quality control orifice at an invert of 93.40 m will provide a drawdown time of 2.8 days for the required 11,132 m^3 of water quality volume. The calculation of extended detention drawdown time is provided in Tables C-3 and C-4 of ***Appendix C*** for interim conditions. The pond will operate with a maximum extended detention storage depth of 1.00 m at an elevation of 94.20 m (22,288 m^3) under interim conditions, as per the ***KWMSS***.

The extended detention component has been provided with side slopes of 5:1 with minor localized variations, as illustrated in ***Drawings 5, 5A and 5B***. Side slopes of 5:1 have been applied to the pond area for 3 m on either side of the permanent pool. The extended detention outlet is illustrated on ***Drawings 6* to 8**.

6.6 Quantity Control

Quantity control for the 2- to 10-year events under interim conditions will be provided by the top of a 9 m x 3 m drop inlet structure, acting as a 24 m long perimeter weir at an invert of 94.20 m. This weir was added to the design at the City's request to ensure that flow will only spill over the broad-crested weir described below for events exceeding the 10-year level. Note that the elevation of this quantity control weir is equal to the 100-year flood level of 94.20 m at the pond outlet (as per the ***Mississippi Valley Flood Plain Mapping Study***, Cumming-Cockburn and Associates Limited, December 1983).

Quantity control above the 10-year level will be provided under interim conditions by a 30 m long weir (as specified in the ***KWMSS***) set in the pond berm at an elevation of 94.60 m.

Calculations in support of the quantity control weirs are provided in Table C-5 (free outfall conditions), Table C-6 (restrictive downstream conditions) and Calculation Sheet C-1 of ***Appendix C*** for interim conditions. The details of the quantity control weirs are provided in ***Drawing 3***.

6.7 Conveyance of Emergency Overflows

In the event of a blockage or a storm greater than the design horizon, the 30 m long quantity control weir is sufficiently sized to act as an emergency overflow weir.

6.8 Access Road

Access roads, including reinforced grass service roads, have been provided in each facility in order to facilitate routine inspection and maintenance activities. The access road has a constant cross slope of 2%.

7.0 ULTIMATE POND COMPONENTS

Refer to the provided ***Drawings 4 to 8*** for detailed design drawings and cross-sections of Pond 4 and components under ultimate conditions.

7.1 Sediment Forebay

The proposed facility has been equipped with two sediment forebay(s) in order to improve the pollutant removal by allowing the larger particles to settle out prior to entering the main cell of the pond. The forebays have been designed with length-to-width ratios of greater than 2:1 and do not exceed one-third of the permanent pool area, as required in the ***SWMP Design Manual***.

Pond 4 obtains flows through two inlet pipes: one 2550 mm diameter circular pipe at 0.3% slope to the south forebay and one pipe (to be designed by others) to the north forebay. The forebays have been sized to meet the greater of the settling and dispersion criteria, as stated in the ***SWMP Design Manual***. Calculations for the minimum dispersion length, settling length and the average velocity have been included in Calculation Sheets D-2 and D-3 of ***Appendix D*** for ultimate conditions. Note that the forebays do not quite meet average velocity requirements based on the average forebay widths, as the widths of the forebays have been set to meet City of Ottawa specifications for cleaning and maintenance, such that the access roads are no more than 15.0 m from the centre of the forebays.

The forebays have been provided with a permanent pool of 2.0 m depth to minimize the potential for re-suspension. In accordance with City of Ottawa criteria, the forebays have been graded at 5:1 above the permanent pool level. A permeable forebay berm has been set 0.30 m below the permanent pool water level.

7.2 Permanent Pool

In accordance with the ***SWMP Design Manual***, the pond should have a permanent pool depth between 1.0 and 3.0 metres. The proposed facility has been designed with a permanent pool depth of 2.5 m.

The permanent pool has been sized in accordance with the requirements of the ***SWMP Design Manual***, Table 3.2, normal protection level for wet pond, 62% imperviousness, as follows:

$$(119.33\ m^3/ha - 40\ m^3/ha) \times 278.288\ ha = 22{,}078\ m^3$$

The proposed facility has a permanent pool volume of 53,815 m^3 under ultimate conditions.

The slopes in the permanent pool will be graded with side slopes of 4:1, with minor localized variations. As per the City of Ottawa design criteria, the side slopes adjacent to the maintenance access road have been graded at 5:1 maximum. In general, and in accordance with the City of Ottawa criteria, the grading above and below the permanent pool elevation has been graded at 5:1 in order to address public safety concerns. The proposed pond grading is shown on ***Drawing 4***.

7.3 Baseflow Augmentation

As requested by MVCA, a vertical circular 200 mm diameter baseflow augmentation orifice at an invert of 93.20 m is provided to control the first 0.2 m of active storage to drawdown times of 4.7 days under ultimate conditions. The calculation of baseflow augmentation drawdown time is provided in Tables D-3 and D-4 of ***Appendix C*** for ultimate conditions.

The baseflow augmentation volumes provided are greater than or equal to 10% of the 100-year active storage volume, where:

7930 m^3 baseflow augmentation : 7072 m^3 10% of 100-year active storage

This is consistent with the conceptual design of the ***KWMSS***.

7.4 Quality Control

The extended detention storage has been sized based on 40 m^3/ha in accordance with the ***SWMP Design Manual*** requirements. The required quality control volume for the 278.288 ha drainage area is calculated as follows:

$$40\ m^3/ha \times 278.288\ ha = 11{,}132\ m^3$$

The quality control volume is contained within the extended detention volume under ultimate conditions.

7.5 Extended Detention

A vertical circular 350 mm diameter quality control orifice at an invert of 93.40 m will provide a drawdown time of 4.9 days for the required 11,132 m^3 of water quality volume. The calculation of extended detention drawdown time is provided in Tables D-3 and D-4 of ***Appendix D*** for ultimate conditions. The pond will operate with a maximum extended detention storage depth of 1.00 m at an elevation of 94.20 m (43,005 m^3) under ultimate conditions, as per the ***KWMSS***.

The extended detention component has been provided with side slopes of 5:1 with minor localized variations, as illustrated in ***Drawings 5B, 5E and 5F***. Side slopes of 5:1 have been applied to the pond area for 3 m on either side of the permanent pool. The extended detention outlet is illustrated on ***Drawings 6*** **to** ***8***.

7.6 Quantity Control

Quantity control for the 2- to 10-year events under ultimate conditions will be provided by the top of a 9 m x 3 m drop inlet structure, acting as a 24 m long perimeter weir at an invert of 94.20 m. This weir was added to the design at the City's request to ensure that flow will only spill over the broad-crested weir described below for events exceeding the 10-year level. Note that the elevation of this quantity control weir is equal to the 100-year flood level of 94.20 m at the pond outlet (as per the ***Mississippi Valley Flood Plain Mapping Study***, Cumming-Cockburn and Associates Limited, December 1983).

Quantity control above the 10-year level will be provided under interim conditions by a 30 m long weir (as specified in the ***KWMSS***) set in the pond berm at an elevation of 94.60 m.

Calculations in support of the quantity control weirs are provided in Table D-5 (free outfall conditions), Table D-6 (restrictive downstream conditions) and Calculation Sheet D-1 of ***Appendix D*** for ultimate conditions. The details of the quantity control weirs are provided in ***Drawing 4***.

7.7 Conveyance of Emergency Overflows

In the event of a blockage or a storm greater than the design horizon, the 30 m long quantity control weir is sufficiently sized to act as an emergency overflow weir.

7.8 Access Road

Access roads, including reinforced grass service roads, have been provided in each facility in order to facilitate routine inspection and maintenance activities. The access road has a constant cross slope of 2%.

8.0 POND OUTFALL VELOCITIES

The 9 m x 3 m drop inlet structure discharges to Poole Creek by twin 3.0 m x 1.2 m rectangular concrete culverts at a 0.5% slope. The velocities in this pipe were calculated based on the highest flow of the interim and ultimate conditions, and of free outfall and restrictive downstream conditions. Free flow velocities in the pipes were calculated using Manning's equation with a Manning's roughness coefficient of 0.013, as presented in ***Table 9***.

Table 9
Pond Outfall Pipe Flows, Depths and Velocities

Event	Critical Condition	Pond Level (m)	Pond Outflow (m^3/s)	Pipe Outflow (m^3/s)	Pipe Flow Depth (m)	Pipe Velocity (m/s)
25 mm 3-Hour Chicago	Interim, Restr. Outfall	94.268	0.837	0.837	0.114	1.221
2-Year, 12-Hour SCS	Interim, Restr. Outfall	94.403	4.031	4.031	0.307	2.187
5-Year, 12-Hour SCS	Interim, Restr. Outfall	94.527	8.227	8.227	0.490	2.799
10-Year, 12-Hour SCS	Interim, Restr. Outfall	94.584	10.453	10.453	0.575	3.030
25-Year, 12-Hour SCS	Ultimate, Restr. Outfall	94.643	13.390	12.948	0.665	3.245
50-Year, 12-Hour SCS	Ultimate, Restr. Outfall	94.695	16.745	15.273	0.745	3.417
100-Year, 12-Hour SCS	Ultimate, Restr. Outfall	94.739	19.907	17.322	0.813	3.550
July 1st, 1979 Event	Ultimate, Free Outfall	94.783	23.540	19.630	0.888	3.685
August 4th, 1988 Event	Ultimate, Restr. Outfall	94.773	22.614	19.005	0.868	3.650
August 8th, 1996 Event	Ultimate, Restr. Outfall	94.673	15.286	14.300	0.712	3.348

The 30 m long spillway has a maximum longitudinal slope of 8.3% to the Carp River. As above, the velocities over the weir were calculated based on the highest flow of the interim and ultimate conditions, and of free outfall and restrictive downstream conditions. Free flow velocities on the spillway were calculated using Manning's equation with a Manning's roughness coefficient of 0.03, as presented in ***Table 10***.

Table 10
Pond Outfall Spillway Flows, Depths and Velocities

Event	Critical Condition	Pond Level (m)	Pond Outflow (m^3/s)	Pipe Outflow (m^3/s)	Pipe Flow Depth (m)	Pipe Velocity (m/s)
25-Year, 12-Hour SCS	Ultimate, Restr. Outfall	94.643	13.390	0.442	0.021	0.719
50-Year, 12-Hour SCS	Ultimate, Restr. Outfall	94.695	16.745	1.472	0.042	1.162
100-Year, 12-Hour SCS	Ultimate, Restr. Outfall	94.739	19.907	2.585	0.059	1.455
July 1st, 1979 Event	Ultimate, Restr. Outfall	94.786	23.620	4.006	0.077	1.733
August 4th, 1988 Event	Ultimate, Restr. Outfall	94.773	22.614	3.608	0.072	1.662
August 8th, 1996 Event	Ultimate, Restr. Outfall	94.673	15.286	0.986	0.033	0.990

9.0 SITE RESTORATION AND POND PLANTINGS

The stormwater management pond will be planted in accordance with the final landscape drawings and the ***SWMP Design Manual***. Refer to the landscape drawings for details of the proposed plantings for the pond and storm outfall.

10.0 SWM FACILITY MAINTENANCE

10.1 Inspections

As recommended in the ***SWMP Design Manual***, inspections should be made after every significant storm (say, >10 mm) during the first two years of operation to ensure that the facility is functioning properly. It is anticipated that four inspections will be required per year. After the initial period, and after proper operation has been confirmed, an inspection schedule can be established based on the observed operation of the SWM facility. As a minimum requirement, the SWM facility should be inspected annually, although four inspections per year are recommended.

10.2 Regular Operation and Maintenance Activities

Grass Cutting

Grass cutting is not recommended for the SWM facility. Allowing grass to grow enhances the water quality and provides other benefits. However, it is understood that grass cutting enhances the aesthetics of the facility for nearby residents. Therefore, grass cutting should be done as infrequently as possible.

Grass should be cut only in one swatch on each side of the service road, with the rest of the land around the pond remaining in a naturalized state.

Plantings

A vegetative community is required in three different locations – upland / flood, shoreline, and aquatic fringes. Planting methods and any replanting should be carried out in accordance with the approved Landscape Design and the recommendations of the ***SWMP Design Manual***, or as modified by the operating authority.

Trash Removal

Trash and debris should be removed by hand, performed as required based on inspections.

Sediment Removal

In accordance with the ***SWMP Design Manual***, it is recommended that the frequency of sediment removal be determined based on a 5% reduction in the total suspended solids (TSS) removal efficiency. Based on ***Figure 6.1***, ***6.2*** and ***Table 3.2*** of the ***SWMP Design Manual***, we have estimated the SWM facility maintenance frequency to be once approximately every 33 years (92.00 m^3/ha, 37% imperviousness) based on interim conditions or every 16 years (119.33 m^3/ha, 62% imperviousness) based on ultimate conditions. It should be noted that routine cleaning of the sediment forebays should allow for less frequent cleaning of the main cell than indicated in the ***SWMP Design Manual***; however, the extension of service life prior to cleaning cannot be quantified.

Safety

The SWM facility should be provided with appropriate signage that warns the public of the presence of deep water and slopes.

Landscape drawings will be prepared with strategic plantings around the perimeter of the SWM facility in order to discourage direct access to the facility.

All inlets, outlets, structures, and headwalls will be provided with the appropriate grates, covers, and safety features in order to prevent public entry or tampering.

11.0 GEOTECHNICAL CONSIDERATIONS

Soil investigations for the stormwater management pond have been completed by Paterson Group. Field investigations indicate that the pond is located in an area of topsoil and/or organic deposits (peat) overlying alluvial deposits of sandy silt and silty sands, and a deeper deposit of silty clay. Groundwater elevations in the vicinity of the pond were measured between elevations of 92 m and 93 m.

Additional details and analysis of the soil, groundwater, and bedrock conditions are described in ***Geotechnical Review, Proposed Stormwater Management Facility (SWMF) Design – Pond***

4, Proposed Fairwinds Residential Development – Maple Grove Road - Ottawa (Paterson Group, August 2014) provided in ***Appendix J***. Summary sheets for borehole and test pit investigations are also included in ***Appendix J***.

The conclusion of the geotechnical review is that the proposed stormwater management pond is acceptable from a geotechnical perspective. Specifically:

- The natural undisturbed clay deposit will serve as a clay liner for the pond. Where silty sand or sandy silt is encountered along the pond side walls or bottom, consideration should be given to subexcavating the pervious soil and replacing it with suitable clay from the pond excavation.

- The proposed excavation side slopes, varying between 5H:1V and 3H:1V, are considered to be stable in the long term.

- The proposed concrete structures (e.g. headwalls, outlet structures, etc.) can be founded within the firm silty clay, but geotechnical field confirmation must be completed before pouring concrete footings or placing granular materials for these structures.

- The interim conditions east forebay inlet will be removed during construction of the ultimate pond, and should be backfilled with a workable, brown silty clay fill placed in maximum 300 mm loose lifts and compacted using several passes of a sheepsfoot roller. The pond sidewall should be reinstated in the same manner. It is further recommended that the granular thickness below the proposed access pathway be thickened to 500 mm of a Granular A or Granular B Type II, compacted to at least 98% of its SPMDD.

- Portions of the proposed storm sewer will require 50 mm to 100 mm thick rigid insulation in order to provide sufficient frost protections.

12.0 THERMAL MITIGATION

Thermal mitigation is not a concern for the proposed SWM facility, given that the allowable outflow temperature for the Carp River is 30^{o} Celsius. Nonetheless, thermal mitigation measures will be provided at the SWM facility by the application of effective shading with landscape material and increased riparian vegetation along the permanent pool.

13.0 WATER BALANCE

In accordance with the ***KWMSS***, post-development infiltration on the pond block is not intended to compensate for decreases in infiltration on upstream areas; each development is required to provide its own pre- versus post-development water balance.

14.0 EROSION AND SEDIMENT CONTROL

An erosion and sediment control strategy will be implemented at the detailed design stage. The erosion and sediment control strategy will include the following:

- methods for constructing SWM and environmental features in the dry
- methods to stabilize disturbed areas to minimize transfer of sediment
- special measures for works in or adjacent to stream corridors, such as culvert crossings, wetland construction, etc.
- environment fencing
- stone mud mat at all construction entrances
- use of the permanent pond as a temporary silt basins during site construction activities
- regular inspection of the erosion and sediment control devices
- removal and disposal of the erosion and sediment control devices after the site has been stabilized

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Prepared by,
David Schaeffer Engineering Ltd.

Per: Jennifer Ailey, P.Eng.

Per: Zhenyong Li, P.Eng.

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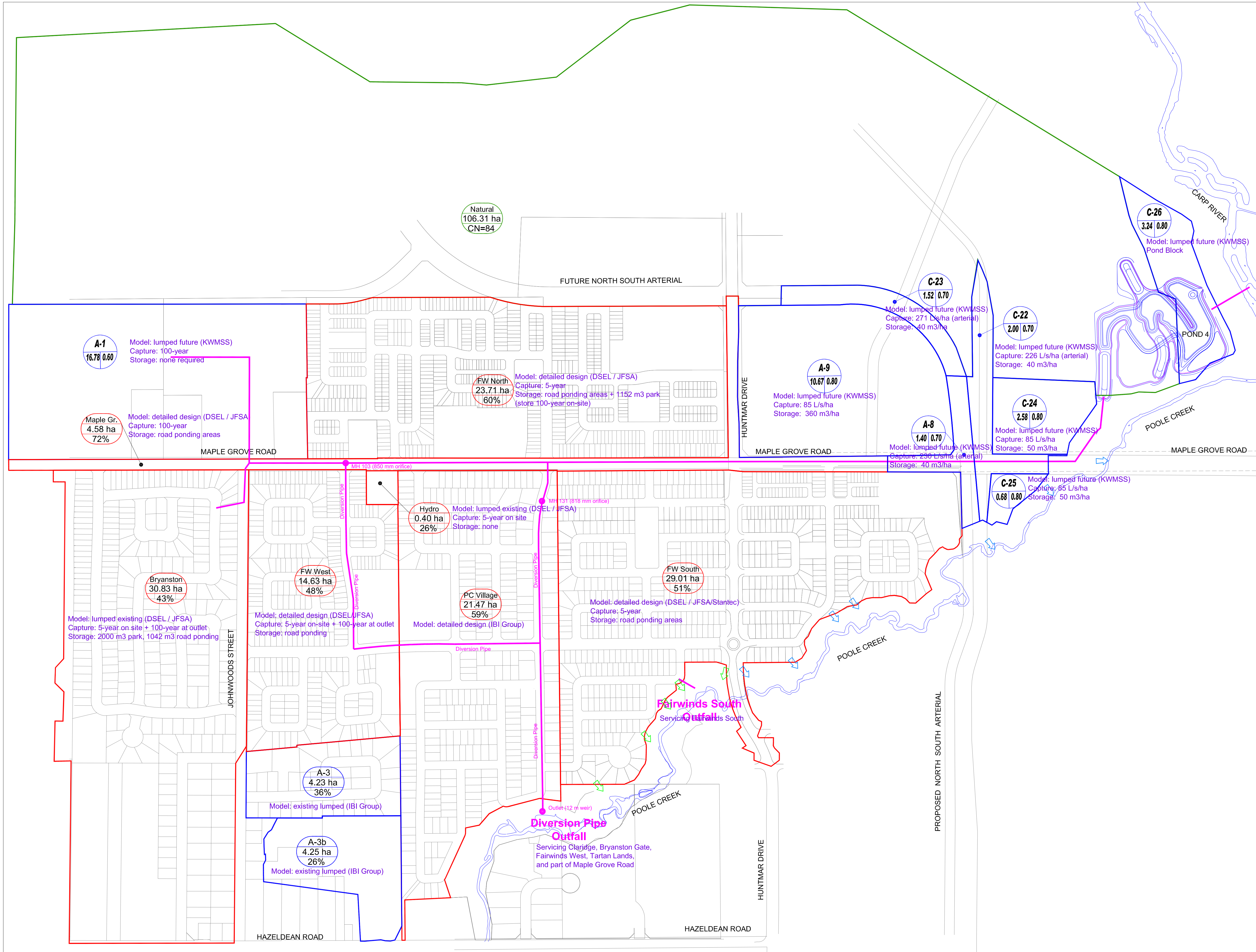
Figure 1: General Location of Subject Site



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PROJECT :
Kanata West Community
Pond 4

No.	BY	DATE	DESCRIPTION	BY

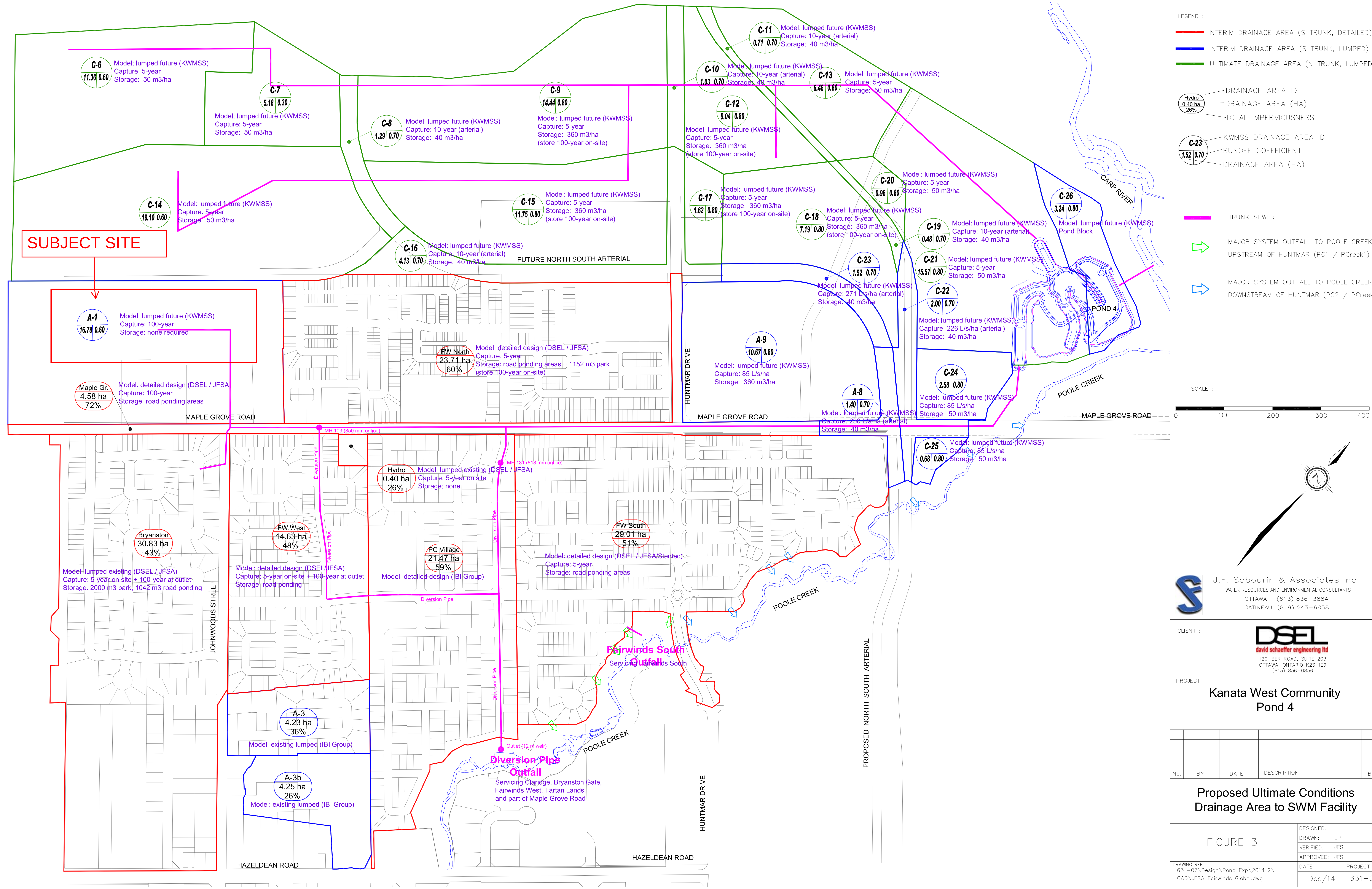
Proposed Interim Conditions
Drainage Area to SWM Facility

FIGURE 2

DESIGNED:	
DRAWN:	LP
VERIFIED:	JFS
APPROVED:	JFS

DRAWING REF.
631-07\Design\Pond Exp\201412\CAD\JFSA Fairwinds Global.dwg

DATE	PROJECT No.
Dec/14	631-07



LEGEND :
INTERIM DRAINAGE AREA (S TRUNK, DETAILED)
INTERIM DRAINAGE AREA (S TRUNK, LUMPED)
ULTIMATE DRAINAGE AREA (N TRUNK, LUMPED)
DRAINAGE AREA ID
DRAINAGE AREA (HA)
TOTAL IMPERVIOUSNESS
KWMSS DRAINAGE AREA ID
RUNOFF COEFFICIENT
DRAINAGE AREA (HA)
TRUNK SEWER
MAJOR SYSTEM OUTFALL TO POOLE CREEK UPSTREAM OF HUNTMAR (PC1 / PCreek1)
MAJOR SYSTEM OUTFALL TO POOLE CREEK DOWNSTREAM OF HUNTMAR (PC2 / PCreek2)
SCALE :
0 100 200 300 400 m
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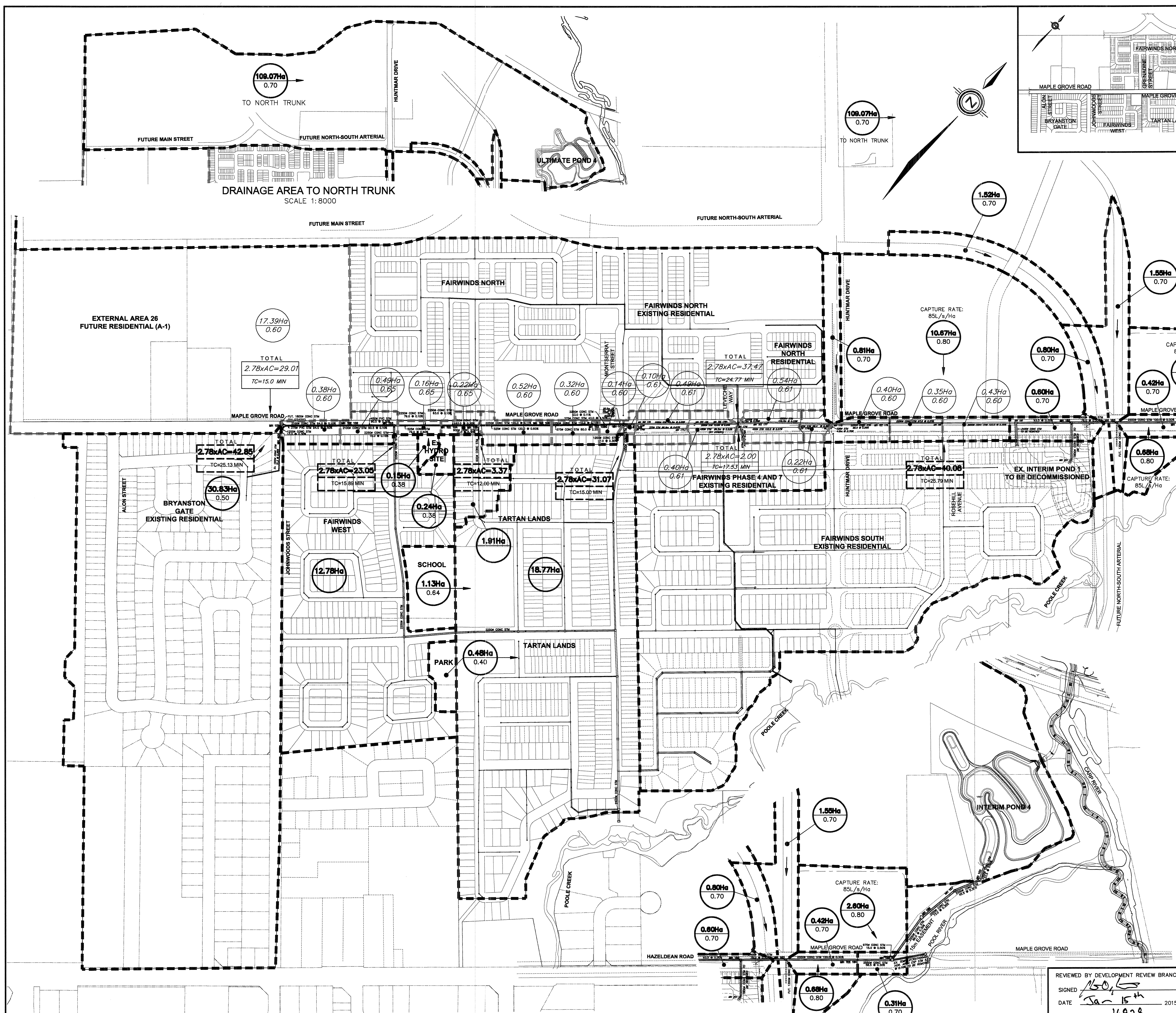
PROJECT :
Kanata West Community
Pond 4
No. BY DATE DESCRIPTION BY
Proposed Ultimate Conditions
Drainage Area to SWM Facility
FIGURE 3
DESIGNED:
DRAWN: LP
VERIFIED: JFS
APPROVED: JFS
DATE Dec/14
PROJECT No. 631-07
DRAWING REF. 631-07\Design\Pond Exp\201412\CAD\JFSA Fairwinds Global.dwg
SUBJECT SITE
A-1 16.78 0.60 Model: lumped future (KWMSS) Capture: 100-year Storage: none required
Maple Gr. 4.58 ha 72% Model: detailed design (DSEL / JFSA) Capture: 100-year Storage: road ponding areas
C-6 11.36 0.60 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-7 5.18 0.30 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-8 1.29 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-9 14.44 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-10 1.03 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-11 0.71 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-12 5.04 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-13 6.46 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-14 19.10 0.60 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-15 11.75 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-16 4.13 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-17 1.62 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-18 7.19 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-19 0.48 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-20 0.96 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-21 15.57 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-22 2.00 0.70 Model: lumped future (KWMSS) Capture: 226 L/s/ha (arterial) Storage: 40 m3/ha
C-23 1.52 0.70 Model: lumped future (KWMSS) Capture: 271 L/s/ha (arterial) Storage: 40 m3/ha
C-24 2.58 0.80 Model: lumped future (KWMSS) Capture: 85 L/s/ha Storage: 50 m3/ha
C-25 0.68 0.80 Model: lumped future (KWMSS) Capture: 85 L/s/ha Storage: 50 m3/ha
C-26 3.24 0.80 Model: lumped future (KWMSS) Pond Block
A-8 1.40 0.70 Model: lumped future (KWMSS) Capture: 250 L/s/ha (arterial) Storage: 40 m3/ha
A-9 10.67 0.80 Model: lumped future (KWMSS) Capture: 85 L/s/ha Storage: 360 m3/ha
FW North 23.71 ha 60% Model: detailed design (DSEL / JFSA) Capture: 5-year Storage: road ponding areas + 1152 m3 park (store 100-year on-site)
Hydro 0.40 ha 26% Model: lumped existing (DSEL / JFSA) Capture: 5-year on site Storage: none
Bryanston 30.83 ha 43% Model: lumped existing (DSEL / JFSA) Capture: 5-year on site + 100-year at outlet Storage: 2000 m3 park, 1042 m3 road ponding
FW West 14.63 ha 48% Model: detailed design (DSEL/JFSA) Capture: 5-year on-site + 100-year at outlet Storage: road ponding
PC Village 21.47 ha 59% Model: detailed design (IBI Group)
FW South 29.01 ha 51% Model: detailed design (DSEL / JFSA/Stantec) Capture: 5-year Storage: road ponding areas
A-3 4.23 ha 36% Model: existing lumped (IBI Group)
A-3b 4.25 ha 26% Model: existing lumped (IBI Group)
Fairwinds South Outfall
Diversion Pipe Outfall
Servicing Claridge, Bryanston Gate, Fairwinds West, Tartan Lands, and part of Maple Grove Road
MH 103 (850 mm orifice)
MH 131 (818 mm orifice)
Outlet (12 m weir)
Diversion Pipe
FUTURE NORTH SOUTH ARTERIAL
MAPLE GROVE ROAD
HUNTMAR DRIVE
JOHNWOODS STREET
HAZELDEAN ROAD
PROPOSED NORTH SOUTH ARTERIAL
POOLE CREEK
CARP RIVER
POND 4



LEGEND

0.25Ha / 0.75	DRAINAGE AREA IN HECTARES RUN-OFF COEFFICIENT (UPDATED AND NEW)
0.25Ha / 0.75	DRAINAGE AREA IN HECTARES (APPROVED FROM RECONSTRUCTION OF MAPLE GROVE ROAD, PROJECT# 10-451) RUN-OFF COEFFICIENT
	STORM SEWER TRIBUTARY BOUNDARY
	STORM SEWER TRIBUTARY BOUNDARY (APPROVED FROM RECONSTRUCTION OF MAPLE GROVE ROAD, PROJECT# 10-451)

TOPOGRAPHIC INFORMATION

TOPOGRAPHIC INFORMATION PROVIDED BY AECOM, RECEIVED ON JANUARY 29, 2013.

LEGAL INFORMATION

CALCULATED DRAFT PLAN PROVIDED BY J.D. BARNES LIMITED, PROJECT No. 05-10-439-05, DATED FEBRUARY 19, 2013.

BENCH MARK No. 0011986U502

TOWNSHIP: STITTSVILLE
CONCRETE CULVERT ALONG HAZELDEAN ROAD, 1.3 KM NE OF ROAD INTERSECTION WITH MAIN ST NORTH. BRASS CAP SET ON TOP OF CONC CULVERT, SOUTH SIDE OF THE ROAD, 30 CM WEST OF EASTERLY EXTREMITY, 30 CM NORTH OF THE SOUTH FACE, SLIGHTLY BELOW ROAD LEVEL.
ELEVATION = 106.039 m

No.	BY	DATE	DESCRIPTION	BY
7	Z.L.	14-12-15	REVISED AS PER FLUVIAL DESIGN	
6	Z.L.	14-11-14	REVISED AS PER CITY'S COMMENTS	
5	Z.L.	14-09-24	REVISED AS PER CITY'S COMMENTS	
4	Z.L.	14-08-20	3rd SUBMISSION	
3	Z.L.	14-06-04	2nd SUBMISSION	
2	Z.L.	14-03-05	1st RE-SUBMISSION	
1	Z.L.	13-08-09	1st SUBMISSION	

Ottawa CITY OF OTTAWA

PROJECT No. 12-644

LICENSED PROFESSIONAL ENGINEER W. LIU 100167932 14-12-15 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER Z. LI 14-12-15 PROVINCE OF ONTARIO

STORM DRAINAGE PLAN

© DSEL

mattamyHOMES

POND 4
KANATA WEST

REVIEWED BY DEVELOPMENT REVIEW BRANCH
SIGNED
DATE Jan 15th 2015
PLAN NUMBER 16828

DSEL david schaeffer engineering ltd
120 Iber Road, Unit 203
Stittsville, ON K2S 1E9
Tel. (613) 836-0856
Fax. (613) 836-7183
www.DSEL.ca

DRAWN BY: W.L./I.V.	CHECKED BY: K.M.	DRAWING NO.	SHEET NO.
DESIGNED BY: K.M.	CHECKED BY: Z.L.		2C
SCALE: 1:3000	DATE: MAY 2013		

M:\12644\Subm_3\Drawings\02C_644_Stm_Drainage.dwg, 02C, 15/01/2015 11:59:01 AM, VY

Appendix F

Stormwater Management Calculations

Guy Forget

From: Jonathon Burnett <jburnett@jfsa.com>
Sent: Wednesday, April 6, 2022 1:49 PM
To: Karla Ferrey; Terry Brule; Santhosh Kuruvilla; Guy Forget; Bobby Pettigrew; Warnock, Charles; Peter Deir; JF Sabourin
Cc: Raad Akrawi; vincent.denomme@claridgehomes.com; Nick Sutherland
Subject: RE: 1927 Maple Grove - Design Criteria - Maple Grove Trunk Storm Sewer
Attachments: Figure 3.pdf

Hi All,

After coordinating with Peter Deir, I can confirm that the hydrology for the subject area (A-1) has not changed since the Dec 2014 JFSA Pond 4 PDB.
Based on the modelling this area has a total drainage area of 16.78 ha at 57% impervious (Rc=0.6) – Per the attached figure from 2014 PDB.

- The 100 year SCS 24Hr peak flow is 3,539 L/s (210.9L/s/ha)
- The 100 Year CHI 3Hr peak flow is 4,046 L/s (241.1 L/s/ha)

Please feel free to reach out if you require the modelling files (from Dec 2014) to check that your detailed modelling work does not have any adverse impacts on the previous analysis.

Regards,

Jono Burnett, B.Eng., P.Eng **(he/him)**
Water Resources Engineer

JFSA
52 Springbrook Drive, Ottawa ON, K2S 1B9
Tel.: 613-322-1253 | Email: jburnett@jfsa.com | Website: www.jfsa.com
Ottawa-Paris(ON)-Gatineau-Montréal-Québec

-----Original Appointment-----
From: Karla Ferrey <kferrey@jlrichards.ca>
Sent: Tuesday, 29 March 2022 12:37 PM
To: Terry Brule; Santhosh Kuruvilla; Guy Forget; Bobby Pettigrew; Warnock, Charles; Peter Deir; Jonathon Burnett; JF Sabourin
Cc: Raad Akrawi; vincent.denomme@claridgehomes.com; Nick Sutherland
Subject: 1927 Maple Grove - Design Criteria - Maple Grove Trunk Storm Sewer
When: Wednesday, 30 March 2022 2:00 PM-3:00 PM (UTC-05:00) Eastern Time (US & Canada).
Where: Microsoft Teams Meeting

Thanks everyone for confirming their availability to meet to discuss the Design Criteria - Maple Grove Trunk Storm Sewer.

See you all tomorrow.

Regards,

Karla

Microsoft Teams meeting

Join on your computer or mobile app
[Click here to join the meeting]

Or call in (audio only)
+1 437-703-4646,,631913575#   Canada, Toronto
Phone Conference ID: 631 913 575#
Find a local number | Reset PIN

Learn More | Meeting options

____

Karla Ferrey, P.Eng.
Associate
Senior Civil Engineer

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700 - 1565 Carling Avenue, Ottawa, ON K1Z 8R1
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J.L. Richards & Associates Limited
ENGINEERS • ARCHITECTS • PLANNERS

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____
From: Terry Brule <tbrule@IBIGroup.com>
Sent: Thursday, March 24, 2022 9:21 AM
To: Karla Ferrey <kferrey@jlrichards.ca>
Cc: Guy Forget <gforget@jlrichards.ca>; Santhosh Kuruvilla <santhosh.kuruvilla@ottawa.ca>; Peter Deir <PDeir@IBIGroup.com>; Bobby Pettigrew <bpettigrew@jlrichards.ca>; Raad Akrawi <rakrawi@groupeheafey.com>; vincent.denomme@claridgehomes.com; Nick Sutherland <sutherland@fotenn.com>
Subject: RE: 1927 Maple Grove - Design Criteria - Maple Grove Trunk Storm Sewer

Good morning Karla, we hope all is well..

Peter and I are free next Wednesday March 30th at 2:00. Please forward a Teams meeting invite if that works for you guys.

A few comments on the emails below:

- Our meeting with the City concluded that the City was supposed to reach out to JFSA to confirm the allocated flow for the lands in questions in the current model for the storm infrastructure in Maple Grove and Pond 4. Rational approach may not apply. Agreement with the City on how to allocate the modelled flow may be required;
- It should be noted that who needs the "pipes" first is responsible for the design and extension as needed;
- We recommend both Santosh and Charles W. be in attendance. IBI is not the model keeper. We were involved in the modelling of the diversion pipe with JFSA but that was several years ago. Hence, ultimately approval is required by the City and we have similar questions.

Cheers!

Terry Brule P.ENG., ING.

Associate Director - Practice Lead, Land Engineering
mob +1 819 664 7322

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400-333 Preston Street
Ottawa ON K1S 5N4 Canada
tel +1 613 225 1311 ext 64068 fax +1 613 225 9868

From: Karla Ferrey <kferrey@jlrichards.ca>
Sent: Thursday, March 24, 2022 9:02 AM
To: Peter Deir <PDeir@IBIGroup.com>; Terry Brule <tbrule@IBIGroup.com>
Cc: Guy Forget <gforget@jlrichards.ca>; Santhosh Kuruvilla <santhosh.kuruvilla@ottawa.ca>; Bobby Pettigrew <bpettigrew@jlrichards.ca>; Raad Akrawi <rakrawi@groupeheafey.com>; vincent.denomme@claridgehomes.com; Nick Sutherland <sutherland@fotenn.com>
Subject: 1927 Maple Grove - Design Criteria - Maple Grove Trunk Storm Sewer

Hi Peter and Terry,

Hope all is well on your fronts.

As you are aware, we have been retained by Maple Grove Towns Inc. to prepare an Adequacy of Public Services Report for the 1927 Maple Grove property, which is adjacent to the 1981 Maple Grove property owned by Claridge. As part of your scope with Claridge, you will be preparing the detailed design of the linear infrastructure (sanitary and storm) along Maple Grove, spanning from your property to a connection with existing infrastructure at ± Johnwoods Street. As part of the pre-consult notes prepared in 2021, we were asked to coordinate with IBI as the proposed infrastructure will serve as the dedicated outlet to our project property.

I know that Alexandre, formerly with JLR, had reached out to IBI to confirm the design basis mainly of the trunk storm sewer which will be designed by IBI on behalf of Claridge. We had also extracted from the City of Ottawa Application platform IBI's Assessment of Adequacy Report (February 2021), which was available in the summer/fall of 2021. Since then, we have obtained the final version of the IBI Report (October 2021). We have prepared our Assessment of Adequacy Report based on the design basis described in IBI's Reports (Section 4.2.1) where the areas serviced by the future trunk storm sewer (west of Santolina Street) were to be restricted to the allowable flow calculated based on a 1:100 year capture, estimated based on a C-Factor of 0.60 and a Tc of 15 min. Thus, for our 1927 Maple Grove property, we still had to provide some surface storage given that our C-Factor will exceed 0.60 and that a Tc of 10 minutes must be used rather than 15 minutes.

Based on the comments received in February 2022, the City did not concur with the above-noted allowable peak flow. We have had a meeting wit the City shortly thereafter, and recently were told that the City (Charles & Santhosh) had a meeting with IBI to discuss the design basis of the upcoming trunk storm sewer as the diversion sewer to Poole Creek (at Santolina street) provides a relief up to the 1:100 year design storm. Given that modelling was carried out by IBI and JFSA, Santhosh has requested that we meet with IBI to better understand the design basis of the Maple Grove Trunk storm sewer as we will need to update our Assessment of Adequacy Report prior to submission to the City.

Based on the above, we ask to meet with IBI to discuss the design basis of the Maple Grove trunk storm sewer system as it is the dedicated storm outlet for 1927 Maple Grove Road. Could you, therefore, send us your availabilities in the near future so we can advance our Report and subsequent submission.

Please note I have also copied Santhosh on this email for information and project tracking purposes only.

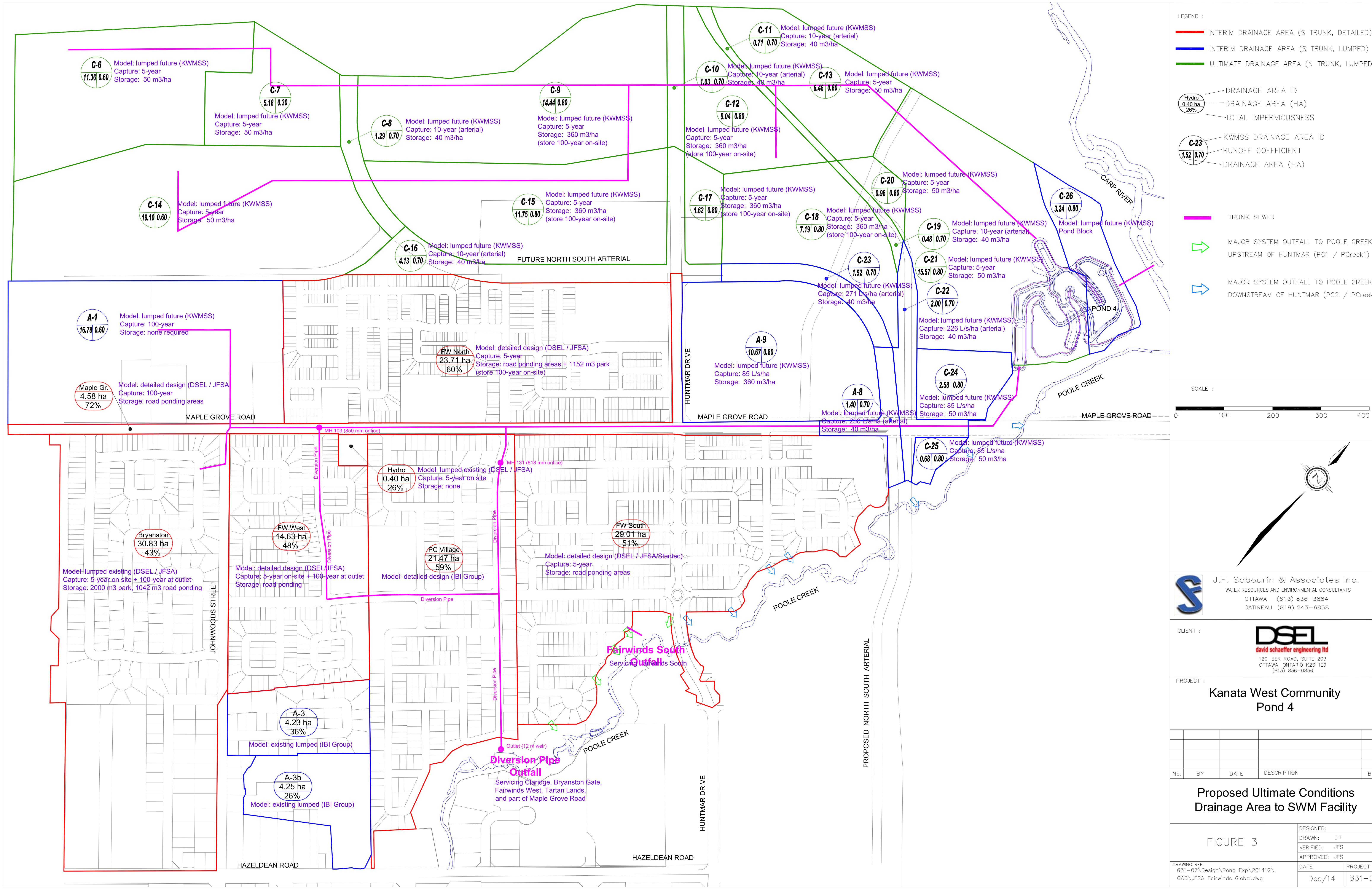
Thank you both in advance for your time and availabilities,

Karla

Karla Ferrey, P.Eng.
Associate
Senior Civil Engineer

J.L. Richards & Associates Limited
700 - 1565 Carling Avenue, Ottawa, ON K1Z 8R1
Direct: 343-306-0062

**** Caution : External Email /// Attention : Courriel externe ****



LEGEND :
INTERIM DRAINAGE AREA (S TRUNK, DETAILED)
INTERIM DRAINAGE AREA (S TRUNK, LUMPED)
ULTIMATE DRAINAGE AREA (N TRUNK, LUMPED)
DRAINAGE AREA ID
DRAINAGE AREA (HA)
TOTAL IMPERVIOUSNESS
KWMSS DRAINAGE AREA ID
RUNOFF COEFFICIENT
DRAINAGE AREA (HA)
TRUNK SEWER
MAJOR SYSTEM OUTFALL TO POOLE CREEK UPSTREAM OF HUNTMAR (PC1 / PCreek1)
MAJOR SYSTEM OUTFALL TO POOLE CREEK DOWNSTREAM OF HUNTMAR (PC2 / PCreek2)
SCALE :
0 100 200 300 400 m
C-6 11.36 0.60 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-7 5.18 0.30 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-8 1.29 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-9 14.44 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-10 1.03 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-11 0.71 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-12 5.04 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-13 6.46 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-14 19.10 0.60 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-15 11.75 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-16 4.13 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-17 1.62 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-18 7.19 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 360 m3/ha (store 100-year on-site)
C-19 0.48 0.70 Model: lumped future (KWMSS) Capture: 10-year (arterial) Storage: 40 m3/ha
C-20 0.96 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-21 15.57 0.80 Model: lumped future (KWMSS) Capture: 5-year Storage: 50 m3/ha
C-22 2.00 0.70 Model: lumped future (KWMSS) Capture: 226 L/s/ha (arterial) Storage: 40 m3/ha
C-23 1.52 0.70 Model: lumped future (KWMSS) Capture: 271 L/s/ha (arterial) Storage: 40 m3/ha
C-24 2.58 0.80 Model: lumped future (KWMSS) Capture: 85 L/s/ha Storage: 50 m3/ha
C-25 0.68 0.80 Model: lumped future (KWMSS) Capture: 85 L/s/ha Storage: 50 m3/ha
C-26 3.24 0.80 Model: lumped future (KWMSS) Pond Block
A-1 16.78 0.60 Model: lumped future (KWMSS) Capture: 100-year Storage: none required
A-8 1.40 0.70 Model: lumped future (KWMSS) Capture: 250 L/s/ha (arterial) Storage: 40 m3/ha
A-9 10.67 0.80 Model: lumped future (KWMSS) Capture: 85 L/s/ha Storage: 360 m3/ha
Maple Gr. 4.58 ha 72% Model: detailed design (DSEL / JFSA) Capture: 100-year Storage: road ponding areas
FW North 23.71 ha 60% Model: detailed design (DSEL / JFSA) Capture: 5-year Storage: road ponding areas + 1152 m3 park (store 100-year on-site)
Hydro 0.40 ha 26% Model: lumped existing (DSEL / JFSA) Capture: 5-year on site Storage: none
Bryanston 30.83 ha 43% Model: lumped existing (DSEL / JFSA) Capture: 5-year on site + 100-year at outlet Storage: 2000 m3 park, 1042 m3 road ponding
FW West 14.63 ha 48% Model: detailed design (DSEL/JFSA) Capture: 5-year on-site + 100-year at outlet Storage: road ponding
PC Village 21.47 ha 59% Model: detailed design (IBI Group)
FW South 29.01 ha 51% Model: detailed design (DSEL / JFSA/Stantec) Capture: 5-year Storage: road ponding areas
A-3 4.23 ha 36% Model: existing lumped (IBI Group)
A-3b 4.25 ha 26% Model: existing lumped (IBI Group)
MH 103 (850 mm orifice)
MH 131 (818 mm orifice)
Diversion Pipe
Outlet (12 m weir)
Diversion Pipe Outfall
Servicing Claridge, Bryanston Gate, Fairwinds West, Tartan Lands, and part of Maple Grove Road
Fairwinds South Outfall
Servicing Fairwinds South
FUTURE NORTH SOUTH ARTERIAL
MAPLE GROVE ROAD
HUNTMAR DRIVE
JOHNWOODS STREET
HAZELDEAN ROAD
PROPOSED NORTH SOUTH ARTERIAL
POOLE CREEK
CARP RIVER
POND 4
J.F. Sabourin & Associates Inc.
WATER RESOURCES AND ENVIRONMENTAL CONSULTANTS
OTTAWA (613) 836-3884
GATINEAU (819) 243-6858
CLIENT :
DSEL
david schaeffer engineering ltd
120 IBER ROAD, SUITE 203
OTTAWA, ONTARIO K2S 1E9
(613) 836-0856
PROJECT :
Kanata West Community
Pond 4
No. BY DATE DESCRIPTION BY
Proposed Ultimate Conditions
Drainage Area to SWM Facility
FIGURE 3
DESIGNED:
DRAWN: LP
VERIFIED: JFS
APPROVED: JFS
DRAWING REF. 631-07\Design\Pond Exp\201412\CAD\JFSA Fairwinds Global.dwg
DATE Dec/14
PROJECT No. 631-07

J.L.Richards
ENGINEERS · ARCHITECTS · PLANNERS

1927 Maple Grove
Maple Grove Towns Inc.

JLR No. 29893-000

STORMWATER MANAGEMENT CALCULATIONS

Allowable release rate based on JFSA model:

Prepared by: GF
Reviewed by: KF

Per JFSA's model:

Unit Flow Rate (1) =	210.9	L/s (100-yr 24 hr SCS)
Unit Flow Rate (2) =	241.1	L/s (100 yr - 3 hr Chicago)

Allowable Peak Flow Rate Calculation:

Unit Flow Rate =	210.9	L/s - most conservative
Area	0.8916	ha
Qp allowable =	188.04	L/s

Proposed Development:

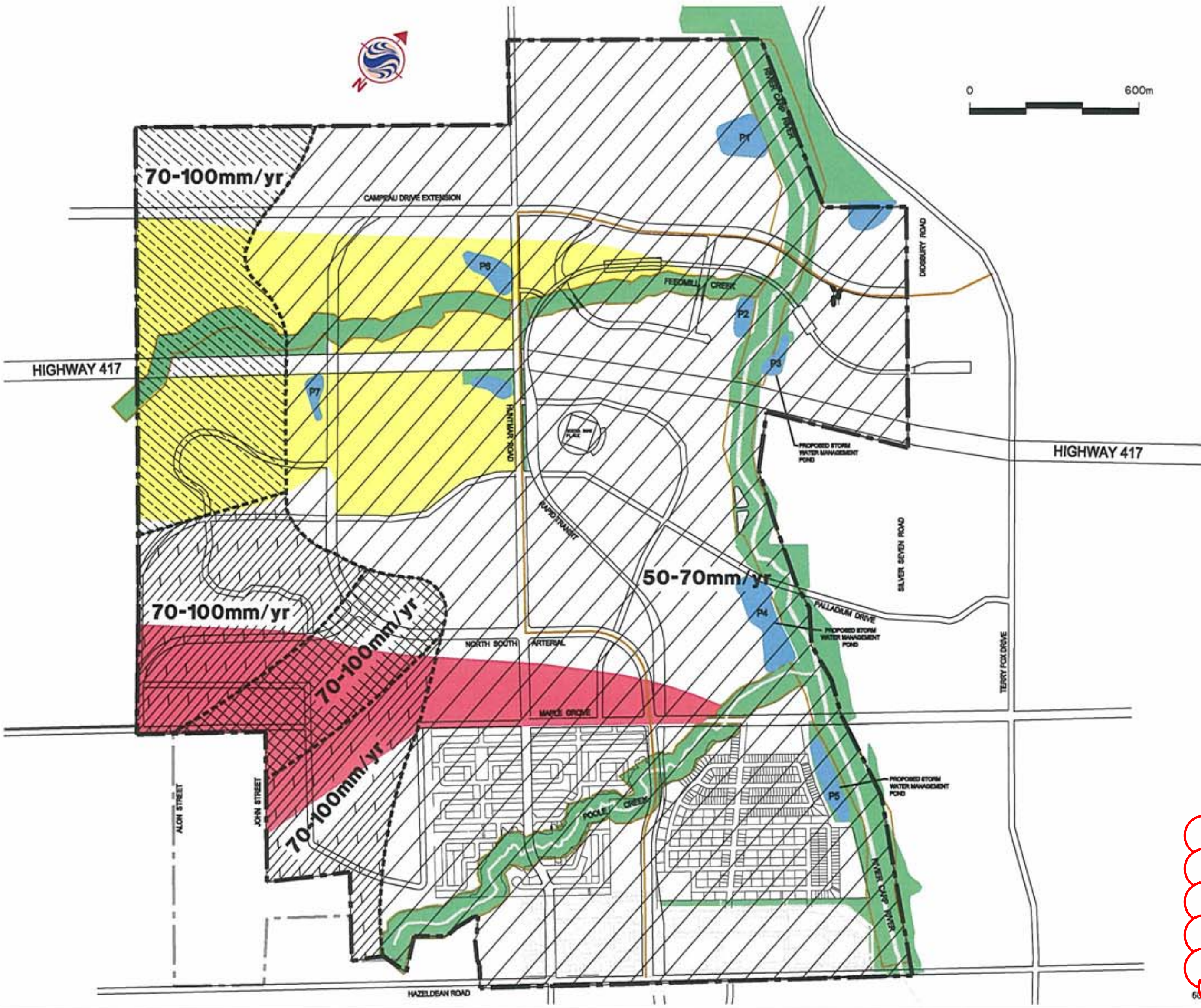
Area	0.8916	ha
Runoff Coefficient =	0.65	
Runoff Coefficient (100-yr) =	0.8125	

Storage Volume Calculations

Time (mins)	1:100-year Intensity (mm/hr)	Peak Flow (L/s)	Allowable (L/s)	Stored (L/s)	Volume (m3)
10	178.56	359.60	188.04	171.56	102.9
15	142.89	287.77	188.04	99.74	89.8
20	119.95	241.57	188.04	53.53	64.2
25	103.85	209.14	188.04	21.10	31.6
30	91.87	185.01	188.04	-3.02	N/A
35	82.58	166.31	188.04	-21.73	N/A
40	75.15	151.34	188.04	-36.70	N/A
45	69.05	139.06	188.04	-48.98	N/A

Appendix G

Infiltration Requirements



INFILTRATION TARGETS

	SOIL TYPE	RECHARGE
	FINE SAND	MODERATE
	PALEOZOIC BEDROCK	MODERATE
	TILL	MODERATE
	CLAY	LOW

Kanata-West Concept Plan Boundary

Area Tributary To Feedmill Creek (Existing Conditions)

Area Tributary To Maple Grove Ditch System and Poole Creek (Existing Conditions)

OPEN SPACE

NOTE:
SOIL TYPES AND RECHARGE POTENTIAL FROM CARP RIVER WATERSHED/SUBWATERSHED STUDY BY ROBINSON CONSULTANTS INC. 2004.
TARGET INFILTRATION RATES OBTAINED FROM ENVIRONMENTAL FACT SHEETS FROM 2004 REPORT.

Stantec

CCL/IBI

MAY 2006

FIG. 5.4

Natural Environment (NE) 20%

All three alternatives will have essentially the same impact on the natural environment. Alternative I has a minor increased impact due to the number of ponds (8) and there location within the KWCP.

5.5.2 Selection of Stormwater Management Alternatives

Based on the above evaluation, Alternative III is selected as the preferred stormwater management alternative. This option offers the greatest amount of flexibility for phasing opportunities while providing an economical servicing solution that meets the objectives of the Carp River Watershed/Subwatershed Study.

5.6 Best Management Practices

The Carp River Watershed/Subwatershed Study (Robinson Consultants, November 2004) proposes target infiltration rates of 104 mm/yr and 73 mm/yr for areas of moderate and low recharge, respectively, within the KWCP. To meet the identified infiltration targets suggested the following best management practices (BMP's) were recommended and are shown on Figures 7.3.3 through 7.3.7 in Appendix 3.4.

- Subsurface Infiltration;
- Biofilters;
- Wet ponds; and
- Dry ponds.

A water balance and subsurface hydrogeological investigation at the detailed design stage will dictate which of the proposed BMPs will be selected for specific developments.

Given the establishment of the dominant soil associations that exist in the Study area **(see Figure 5.4)**, and considering the extent of the poorly draining soils within the nearly flat topography, it is apparent that drainage in the Study area is primarily governed by the characteristics of the poorly draining silty clay to clay soils underlying all but a small percentage of the Study area. As a result, the establishment of the infiltration rates of the soils can be simplified to reflect the silty clay to clay soils and the till material over bedrock. Table 5.6 below summarizes the anticipated infiltration rates of these two principal soil groups, based on soil characteristics and borehole data regarding degree of compaction.

Table 5.6 -Summary of Infiltration Rates of Principal Soil Groups

Soil Groups	**Estimated Infiltration Rates [1] (mm/yr)**	**Percent of Annual Rainfall Infiltrated**
Castor, Dalhousie, North Gower (silty clay to clay)	50-70 mm/yr	5-7
Anstruther, Farmington, Nepean (sandy loams to till)	70-100 mm/yr	7-11

1. Infiltration rates presented in this table are consistent with the average hydraulic conductivities of the individual soils comprising the principal soil group.

As the infiltration rates provided in Table 5.6 reflect estimated hydraulic conductivities only, further in-situ analysis of soils under saturated loading conditions is necessary at each site in order to provide site-specific values. The above rates are based on borehole logs completed to date appended to this report in Appendix 3.5.

Post development infiltration rates are to be increased by 25 percent above the pre-development rate. This rate of infiltration has been established to compensate for those areas (ie. Roadway corridors) that can not provide infiltration.

5.7 Stormwater Management Design

Preliminary site plans of each of the proposed ponds have been prepared and are provided in Appendix 3.1. These ponds have been sized to meet the requirements established in Section 5.2. It is noted that the pond site plans are included to demonstrate the land area required to accommodate an appropriate SWM facility and are not intended for construction purposes. A detailed design of the specific facilities will be required at the subsequent design stage. Stage-storage curves for the proposed ponds are presented in Appendix 3.3.1.

At the detailed design stage for Ponds 6 and 7, consideration shall be made for erosion control volumes in order to comply with any erosion control criteria established for Feedmill Creek.

Low flow velocities for existing and future conditions were modeled for the 2, 5 and 10 year events to assess erosion potential. Pond banks are clay and loam and the calculated velocities do not approach levels that would create erosion for these banks.

The post development analysis addresses the potential changes in the Regulatory 1:100 year flood plain and the potential impact on erosion throughout the reach. The hydrologic and hydraulic analysis, which has been reviewed and supported by the Mississippi Valley Conservation Authority, indicates that there will be no significant impact. A further assessment of the potential for erosion has been conducted in the Flow Characterization and Flood Level Analysis, prepared by CH2MHill and dated June 2006. Pond sizing is provided in **Tables 5.7.1 and 5.7.2** below.

Table 5.7.1 – Stormwater Management Pond Elevations Constraining the Minor System

Pond	***Carp/Poole/Feedmill 100 year Water Level (m)****	***Carp/Poole/Feedmill Normal Water Level (m)***	***100 year Pond Level* (m)***
1	93.65	92.00	93.96
2	93.80	92.25	94.23
3	93.85	92.25	94.20
4	94.20	92.50	94.74
5	94.60	92.70	94.94
6	97.20	96.50	98.94
7	101.80	100.50	102.92

- 100 yr water levels from Mississippi Valley Conservation Authority Regulatory Floodplain Mapping

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J.L.Richards
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