Shea Road Lands Development Conceptual Site Servicing and Stormwater Management Report

Job #160400900

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Sign-off Sheet

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1.0 INTRODUCTION AND BACKGROUND

Stantec Consulting Ltd. has been retained by 1384341 Ontario Inc. to provide a conceptual servicing plan to support the proposed Shea Road Lands Development. The subject property is located on the north-western quadrant of the intersection of Shea Road and Fernbank Road in the City of Ottawa as shown in **Figure 1**

The proposed development comprises approximately 34.0 ha of land, and comprises a school block, a designated park area, a future low density residential block, and a mix of townhomes, semi-detached homes, and single family units. The intent of this report is to provide a servicing scenario for the proposed site that is free of conflicts, includes future development, and utilizes the existing local infrastructure in accordance with the background studies.

Figure 1: Approximate Location of Shea Road Lands Development

1.1 BACKGROUND

In 2008, the City of Ottawa completed the Fernbank Community Design Plan (FCDP). The FCDP covers approximately 675 ha of land between the established communities of Stittsville, Kanata West and Kanata South. The community extends from Hazeldean Road to the north, the Carp River and Terry Fox Drive to the east, Fernbank Road to the south and the existing Urban Area of Stittsville to the west.

In conjunction with preparation of the Community Design Plan, several Class Environmental Assessment Studies/Master Plans were also prepared. Two of those were the Master Servicing Study (MSS) for water and sanitary and an Environmental Management Plan (EMP) for the natural environment and stormwater management (SWM). Those reports identified planning level solutions for on-site storm drainage, wastewater collection and water supply and distribution to the community. The approved EMP and MSS recommended the construction of one stormwater management facility (referred to as Pond 4) and associated storm sewer systems to provide stormwater management for the Fernbank Community tributary to the Faulkner Drain Tributary.

IBI Group prepared a Conceptual Servicing and SWM Report in 2013 to address servicing requirements for the subject lands as well as the lands owned by Tartan Homes as shown in **Figure 1**. IBI's report identified that the urban boundary was extended southerly to include an area called Area 6, which resulted in an increase in the size of the trunk sanitary sewer routed through the site to service the additional Area 6 lands. Since then, the proposed draft plans have been revised, the City of Ottawa SWM guidelines have been updated, an additional section of sanitary trunk sewer has been installed on Robert Grant Avenue, and a conceptual design brief has been prepared for Area 6, south of the site which will be serviced through the proposed site sanitary trunk sewer and connected to the existing Fernbank Road watermain which will also be used to service the proposed site. **Figure 2** shows the extent of the existing watermains and sanitary sewers in the area.

As required by the City of Ottawa, Stantec has prepared the Fernbank SWM Pond 4 Design Brief in support of draft plan approval for the Shea Road Lands Development. The SWM pond design brief will be submitted under separate cover. A summary of the results from the detailed pond design as it pertains to the conceptual site storm servicing plan will be provided in the SWM section of this report.

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Legend

— · — · —	EXISTING WATERMAIN
———	EXISTING SANITARY SEWER

Notes

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SHEA ROAD LANDS
DESIGN BRIEF

Figure No.
2

Title
EXISTING CONDITIONS PLAN

1.2 OBJECTIVE

This servicing report is being prepared in support of draft plan approval for the Shea Road Lands Development. This report will provide a recommended servicing plan for the major municipal infrastructure needed to support development of the subject property. The review will be a macro level detail study with further details to be confirmed and provided during the detailed design process. This report will demonstrate how proposed municipal servicing is in conformance with the MSS and EMP recommendations. Any deviation from the MSS documents will also be identified with rationalization for the change.

1.3 BACKGROUND RESOURCES

The following documents were referenced in the preparation of this report:

- *Fernbank Community Design Plan Existing Conditions Report – Storm Drainage and Hydrology,* Novatech Engineering Consultants Ltd., January 2007
- *Fernbank Community Design Plan Environmental Management Plan,* Novatech Engineering Consultants Ltd., June 24, 2009
- *Fernbank Community Design Plan Master Servicing Study,* Novatech Engineering Consultants Ltd., June 24, 2009
- *Geotechnical Investigation Proposed Residential Development Shea Road, Ottawa, Ontario,* Golder Associates, October 2011
- *Conceptual Site Servicing Plan, Stormwater Management Plan and Erosion and Sediment Control Plan Shea Road Lands Fernbank Community*, IBI Group, March 2013
- *Conceptual Site Servicing Study Davidson Lands – OPA 76 Area 6a, Stittsville South*, IBI Group, November 2015
- *Fernbank Pond 4 Stormwater Management Facility Design Brief*, Stantec Consulting Ltd., December 8, 2017*CRT Lands Phase 1 Fernbank Community Design Brief, IBI Group, July 2017*

Additional documents referenced in designing the conceptual servicing plan for the Shea Road Lands Development include:

- *Stormwater Management Planning and Design Manual*, Ministry of the Environment (Ontario), March 2003
- *Ottawa Design Guidelines – Water Distribution*, City of Ottawa, July 2010
- *City of Ottawa Sewer Design Guidelines*, 2nd Ed., City of Ottawa, October 2012
- *Technical Bulletin PIEDTB-2016-01 Revisions to Ottawa Design Guidelines – Sewer,* City of Ottawa, September 2016

2.0 POTABLE WATER

The Fernbank Community is located within the City's 3W Pressure Zone which includes most of Kanata and Stittsville. Potable water to this area is pressurized at the Glen Cairn Pump Station where a major water storage reservoir (Glen Cairn Reservoir) is located. Two of the major watermains into this pressure zone from the pump station are located along Hazeldean Road and Terry Fox Drive. Another main adjacent to the subject site is located in Abbott Street and the Trans Canada Trail.

The Fernbank Community Design Plan MSS completed a review of the existing water plan adjacent to the area and made recommendations for improvements and expansion to the City's water transmission and distribution system to support the proposed Fernbank Community. **Figure 2** indicates the limits of existing watermains in the vicinity of the subject property while excerpts from IBI's servicing report for the subject lands regarding the watermain plan are included in **Appendix A.4.**

The site will be serviced through three watermain connections: a 200 mm diameter connection to Samuel Mann Avenue, a 400 mm diameter watermain on Fernbank Road, and a 300 mm diameter watermain on Shea Road south of the intersection with Abbott Street East. Proposed ground elevations for the site vary from approximately 110.5 m to 115.0 m. Under normal operating conditions, hydraulic gradelines vary from approximately 148.4 m to 160.8 m as confirmed through boundary conditions provided by the City of Ottawa (See **Appendix A.1**).

2.1 WATER DEMANDS

Water demands for the development were estimated using the City of Ottawa Water Distribution Design Guidelines. A daily rate of 15,000 L/ha/d was used for the proposed school. The population for the low density residential blocks was estimated based on a density of 20 units/ha. See **Appendix A.2** for detailed domestic water demand estimates.

The average day demand (AVDY) for the entire site was determined to be 12.2 L/s. The maximum daily demand (MXDY) was determined to be 30.1 L/s and was calculated as 1.5 times the AVDY (school block) and 2.5 times the AVDY for all other areas (residential). The peak hour demand (PKHR) totaled 66.0 L/s and was calculated as 1.8 times the MXDY (school block) and 2.2 times the MXDY for all other areas (residential).

The fire flow requirement was capped at 10,000 L/min (167 L/s) as per the City of Ottawa Technical Bulletin ISDTB-2014-02 (May 2014), provided that firewalls with a minimum two-hour fire-resistance rating that comply with OBC Div. B, Subsection 3.1.10, are constructed to separate townhouse blocks to the lesser of seven dwelling units and 600 m^2 of building area, and that a minimum 10 m separation exists between rear yards. A 15,000L/min fire flow requirement was used for the school block.

2.2 HYDRAULIC MODEL RESULTS

A hydraulic model was used to simulate the proposed development conditions based on boundary conditions provided by the City of Ottawa. The hydraulic analysis was completed with H2OMAP Water Software and assessed the internal network and connections to the surrounding infrastructure. The model was tested under average day, peak hour, and maximum day plus fire flow conditions.

The proposed watermain layout allows serviceable pressures to be maintained under average day, peak hour, and maximum day plus fire flow demands. The minimum pressure is approximately 41.3 psi (284 kPa) and the maximum pressure modeled is approximately 60.8 psi (419 kPa). These pressures are within the serviceable limit of 40 to 80 psi (276 to 552 kPa) as per City of Ottawa guidelines.

A fire flow analysis was carried out using the hydraulic model to determine the anticipated amount of flow that can be provided for the proposed development under maximum day plus fire flow demands while maintaining a minimum pressure of 20 psi. A fire flow demand of 167 L/s was used for all residential nodes, while a fire flow demand of 250 L/s was used for node 1 which is adjacent to the proposed school block. Results of the modeling analysis indicate that flows in excess of 10,680 L/min (178 L/s) and 16,140 L/min (269 L/s) at all residential nodes and node 1 respectively, can be delivered while still maintaining a residual pressure of 140 kPa (20 psi). Results of the hydraulic modeling are included for reference in **Appendix A.3**.

2.2.1 SUMMARY OF FINDINGS

Based on the findings of the report, the proposed water network is capable of servicing the proposed development and meets all servicing requirements as per City of Ottawa standards under typical demand conditions (peak hour and average day conditions) as well as under emergency fire demand conditions (maximum day + fire flow).

3.0 WASTEWATER SERVICING

3.1 BACKGROUND

The Hazeldean Pump Station (HPS) is the recommended wastewater outlet for all lands in the Fernbank Community, including the subject site. Among other areas in Kanata, including Bridlewood, Kanata South Business Park and the Glen Cairn Community, the HPS also serves most developed lands in Stittsvile west of Terry Fox Drive and south of Hazeldean Road. Sewage peak flows from the Fernbank Community are conveyed to the HPS through an existing 600 mm diameter trunk sewer that runs along the Trans Canada Trail parallel to the Stittsville Trunk sewer on Abbott Street as shown on **Figure 2**.

The 2009 MSS Report completed a sanitary hydraulic gradient (HGL) analysis. The recommended overflow system includes a diversion to the Monahan Constructed Wetlands Stormwater Management Facility. The predicted HGL at the station is 95.0 m. The overflow will protect all development lands in the Fernbank Community and most of the existing sewershed. The sanitary HGL does not impact the subject lands given that the lowest road elevation is approximately 110.5 m.

3.2 DESIGN CRITERIA

As outlined in the City's Design Guidelines for Sewage Works and 2009 Fernbank MSS, the following criteria were used to calculate estimated wastewater flow rates and to size the sanitary sewers:

- Minimum Velocity – 0.6 m/s (City)
- Maximum Velocity – 3.0 m/s (City)
- Manning roughness coefficient for all smooth wall pipes – 0.013 (City)
- Minimum size – 200mm dia. for residential areas, 250mm for commercial areas (City)
- Single Family Persons per unit – 3.3 (MSS)
- Townhouse Persons per unit – 2.5 (MSS)
- Average Apartment Persons per unit – 1.8 (MSS)
- Extraneous Flow Allowance – 0.28 L/s/ha (City/MSS)
- Manhole Spacing – 120 m (City)
- Minimum Cover – 2.5 m (City)
- Average Daily Discharge / Person – 350 L/cap/day (MSS)

3.3 PROPOSED SERVICING

Wastewater from the proposed Shea Road Lands Development will be conveyed through the sanitary trunk sewers within the future CTR lands which ultimately connect to the existing Fernbank trunk sewer on the Trans Canada Trail as shown on the MSS Drawing No. 101108-SAN included in **Appendix B.2**. The conceptual main trunk sewer alignment is shown on **Drawing OSA-**

1. As identified on **Drawing OSA-1,** a higher level sanitary sewer is proposed in all right of ways where the trunk sanitary sewers exceed 5 m in depth to avoid deep residential service connections to the main and facilitate any potential future service repairs. The high level sanitary sewer will be installed along the same alignment as the trunk sanitary sewer and will utilize the same manholes while providing connection to the main sanitary trunk via external drop structures.

The site sanitary trunk sewers will be sized to service the proposed development, the future low density residential block (area R19B), the Tartan development west of the site (draft plan provided in **Appendix B.2**), as well as OPA Area 6 expansion lands which are estimated to generate approximately 84 L/s of sewage peak flows. Additionally, an allowance of 108 L/s has been included from the existing Laird Street Pump Station (see report excerpts in **Appendix B.2**). The conceptual sanitary sewer design sheet can be found in **Appendix B.1**. A breakdown of the estimated sewage peak flows is shown in **Table 1**.

The sewage peak flows to be conveyed through the proposed site to the trunk sewers within the CRT lands are summarized in **Table 1** below.

Table 1: Estimated Wastewater Peak Flows

Property	Population (persons)	Institutional Area (ha)	Residential/Institutional Peak Flow (L/s)	Total Area (ha)	Extraneous Flow (L/s)	Total Peak Flow (L/s)
Shea Road and Tartan Developments	2,369	2.44	38.23	49.12	13.75	51.98
Future Low Density Residential Block (R19B)	238	N/A	3.90	2.57	0.72	4.62
Area 6	N/A	N/A	N/A	N/A	N/A	84.00
Laird Street Pump Station	N/A	N/A	N/A	N/A	N/A	108.00

1. Average daily flows based on 350 L/p/day
2. Population based on 3.3 person/single units and 2.5 person/semi-detached and town homes.
3. 28 units/ha assumed for future low density residential block as per Fernbank MSS
4. Extraneous Flow based on 28 L/s/ha
5. Institutional Peak Flow based on 50,000 L/ha/day
6. Peaking factor for institutional areas of 1.5

4.0 STORM DRAINAGE

The proposed development encompasses approximately 34.0 ha and comprises a school block, designated park land, a stormwater management (SWM) block, a future low density residential block and a mix of single family homes, semi-detached units and town homes. Post development runoff from the development will be directed to a proposed SWM wet pond which will provide quantity and quality control (80% TSS removal) of runoff before discharging to the Faulkner Municipal Drain through an existing 700 mm diameter CSP crossing Fernbank Road. The proposed SWM facility, identified as Pond 4 in the Fernbank Community EMP, will receive runoff from approximately 59.2 ha of land including the proposed development, the adjacent Tartan development to the west and the SWM pond footprint area. **Drawing OSD-1** shows the overall major and minor system flow direction as well as the proposed SWM Pond layout.

The storm drainage objective is to complete a conceptual stormwater management plan for the lands tributary to the Fernbank SWM Pond 4 that meets all relevant design criteria.

4.1 BACKGROUND

IBI Group prepared a Conceptual Servicing and SWM Report in 2013 to address servicing requirements for the subject lands owned by Tartan and Cavanagh (see **Drawing OSD-1**). IBI's report identified that a hydro corridor extends south on the east side of the subject site, crossing Fernbank Road at Shea Road, partially within the proposed Pond 4 location. The report outlined the stage-storage relationship representing Pond 4 in the EMP and MSS was used in their SWMHYMO hydrologic model. IBI's conceptual design provided a SWM pond footprint to meet MOECC quality control volumetric requirements and to restrict post development peak flows from the overall development to pre-development levels as per the EMP. Report excerpts have been provided in **Appendix C.4**. **Table 2** shows the existing condition peak flows to the Faulkner Municipal Drain at Fernbank Road, as presented in the Fernbank EMP.

Table 2: Existing Condition Peak Flows to the Faulkner Tributary at Fernbank Road

Distribution	Peak Flow (m^3/s)					
	2-year	5-year	10-year	25-year	50-year	100-year
12hr AES	0.46	0.74	0.94	1.19	1.37	1.55
12hr SCS	0.48	0.82	1.05	1.39	1.58	1.83
24hr SCS	**0.51**	**0.83**	**1.05**	**1.32**	**1.55**	**1.85**

During the pre-consultation meeting for the proposed Shea Road Lands development in 2016 (see attached correspondence in **Appendix C.4**), the City advised that the allowable release rate from the Fernbank SWM Pond 4 should be coordinated with the work being done to update the Faulkner Municipal Drain Engineer Report by Andy Robinson of Robinson Consultants. Subsequent correspondence with Robinson Consultants confirmed that the existing 700 mm diameter culvert under Fernbank Road has a maximum capacity of 0.9 m^3/s with a 0.5 m head and as such, the 100-year outflow from the proposed SWM Pond 4 should be restricted to 0.9 m^3/s, instead of the 100-year pre-development condition peak flow of 1.85 m^3/s previously identified in the background reports.

Stantec circulated the conceptual SWM Pond 4 layout to HONI stakeholders and obtained approval on October 25, 2017 for the proposed footprint and measures provided to protect the existing Hydro structures and to provide adequate access for maintenance (correspondence included in **Appendix C.4**).

The detailed design of the Fernbank SWM Pond 4 has been prepared by Stantec and will be submitted under separate cover.

4.2 DESIGN CONSTRAINTS AND REGULATORY REQUIREMENTS

The following summarizes the SWM criteria and constraints that will govern the detailed design of the proposed development as per the governing background studies and recent conversations with City staff.

- Design using the dual drainage principle.
- 'Enhanced' level of treatment as per MOECC recommendations which represents an equivalent 80% TSS removal to be provided in the Fernbank SWM Pond 4.
- Quantity control to be provided in the Fernbank SWM Pond 4 to match existing condition peak flows to the Faulkner Municipal Drain at Fernbank Road with a maximum release rate of 0.9 m^3/s.
- Maximum 100-year water depth of 0.35 m in road sags, including overflow spill depth.
- Average sag storage of 40 m^3/ha to be provided in residential areas.
- Proposed school block to provide on-site storage for the 100-year design storm.
- Rear-yard storage is not to be included in calculations.
- Parks and open spaces are to have no surface ponding storage.
- 100-year hydraulic grade line (HGL) to be a minimum 0.30 m below lowest building underside of footing elevation.
- Design inlets along local roadways to capture the 2-year peak flow.
- Design inlets within the school block and along collector roadways to capture the 5-year peak flow.
- Design storm sewers along local and collector roadways to convey the 2-year and 5-year peak flow respectively under free-flow conditions using 2004 City of Ottawa I-D-F parameters and an inlet time of 10 minutes.

- Provide adequate emergency overflow conveyance to SWM Pond 4 as shown on **Drawing OSD-1**.
- The EMP and MSS outline target infiltration rates for the lands tributary to the Faulkner Drain Tributary. Specifically, the MSS identifies a post development infiltration target of 80 mm/year.
- Design and submit a detailed erosion control plan.

4.3 PROPOSED CONDITIONS

Site sewers will outlet to the proposed SWM Pond 4 that will provide quality control and mitigate post development peak flows to the target peak outflows. Inlet control devices at road low points will be sized at the detailed design stage to restrict inflow rates to the sewer to the 2-year runoff for local streets and the 5-year runoff for the school block and for collector roads (Shea Road and Cope Drive) as per the City design criteria. Storm sewer sizes for the proposed development and the adjacent Tartan development are included only for the larger sewers as shown in the storm sewer design sheet included in **Appendix C.1**. All storm sewer sizes will be reviewed and confirmed during the detailed design stage. Major system peak flows from the entire site will be directed towards the proposed SWM pond (see **Drawing OSD-1**).

Detailed design of the Fernbank SWM Pond 4 was done using a lumped hydrologic/hydraulic model of the tributary developments to determine the inflow rates to the pond and assess the pond performance and hydraulic grade line (HGL) across the tributary developments. The results of the analysis will be summarized in this report. However, this report should be read in conjunction with the *Fernbank Pond 4 Stormwater Management Facility Design Brief* which will be submitted by Stantec under separate cover.

4.3.1 End of Pipe Stormwater Management Facility

The conceptual design of Pond 4 was presented in the EMP and MSS to match existing condition peak flows. However, as mentioned in **Section 4.1**, the 100-year outflow from the proposed SWM Pond 4 will be restricted to 0.9 m^3/s, instead of the 100-year pre-development condition peak flow of 1.85 m^3/s previously identified in the background reports. The SWM facility is proposed to be located north of Fernbank Road within the hydro corridor and has been designed to provide water quality and water quantity control of stormwater runoff from the subject site and the adjacent Tartan development. The SWM facility has been designed as a wet pond with one minor system inlet, two major system inlets and an outlet to the Fernabank road side ditch which discharges into the Faulkner Drain Tributary through an existing 700 mm diameter CSP. In accordance with the EMP and MSS, there are no proposed changes to the Faulkner Drain Tributary. The location of Pond 4 is indicated on **Drawing OSD-1**.

The normal water level in the SWM pond has been set at 105.75 m as per the Fernbank Community EMP. The maximum permanent water depth within the forebay and main cell of the facility is 1.5 m. The required level of treatment for the proposed SWM Pond is ‘enhanced’ or 80%

TSS removal as per the Jock River Reach 2 Subwatershed Study. **Table 3** illustrates how the proposed SWM Pond 4 provides this level of treatment.

Table 3: Fernbank Pond 4 MOECC Stormwater Quality Volumetric Requirements

Drainage Area (ha)	Actual % Imp.	Water Quality Unit Volume Requirements			Water Quality Volume Requirements		Water Quality Volumes Provided	
		Total Unit Volume (m^3/ha)	Permanent Pool (m^3/ha)	Extended Detention (m^3/ha)	Permanent Pool (m^3)	Extended Detention (m^3)	Permanent Pool (m^3)	Extended Detention (m^3)
59.2	55	190	150.0	40	8,880	2,368	9,569	5,442

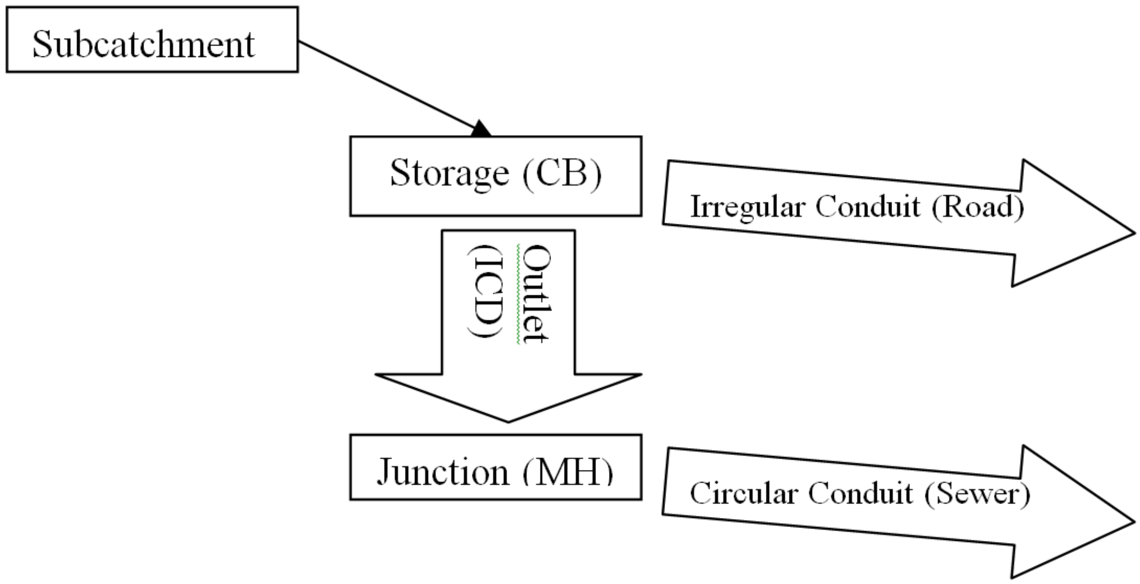
4.4 POST DEVELOPMENT CONCEPTUAL MODELLING RATIONALE

Hydrologic and hydraulic modeling of the SWM pond design was completed using PCSWMM modeling software which uses the EPA-SWMM 5.1.012 computational engine for analysis. The included models can also be opened and reviewed using the free EPA-SWMM GUI. PCSWMM model layout, input parameters, and example input file are provided in Error! Reference source not found.. Electronic model files are provided on the enclosed CD. As previously noted, the site design is currently at a conceptual level and will be further refined at the detailed design stage. The following sections summarize the input parameters used in the conceptual post development model.

4.4.1 SWMM Dual Drainage Methodology

The proposed development is modeled in one modeling program as a dual conduit system (see **Figure 3**), with: 1) circular conduits representing the sewers & junction nodes representing manholes; 2) irregular conduits using street-shaped cross-sections to represent the approximate overland road network and storage nodes representing catchbasins. The dual drainage systems are connected via outlet link objects from storage node (i.e. CB) to junction (i.e. MH), and represent inlet control devices (ICDs). Subcatchments are linked to the storage node on the surface so that generated hydrographs are directed there firstly.

Figure 3 : Schematic Representing Model Object Roles



Storage nodes are used in the model to represent catchbasins. The invert of the storage node represents the invert of the CB and the rim of the storage node represents the top of the CB plus the allowable flow depth on the segment. For the purpose of this conceptual SWM plan, CB inverts have been assumed to be 2.15 m below the top of the CB and a flow depth of 0.60 m has been assumed on grassed swale segments and of 0.35 m on road segments.

Storage nodes on street catchments were assigned a storage curve assuming a maximum storage of 40 m^3/ha. Storage curves in PCSWMM are required to be input as depth-area curves, as such an equivalent area was calculated at a depth of 2.15 m and kept constant at the rim depth. All storage was assumed to occur between the top of the CB (1.8 m head) and a 0.35 m depth (2.15 m head) prior to spilling into the downstream segment. If the available storage volume in a storage node is exceeded, flows spill above the storage node and into the downstream irregular conduit (representing roads) and continue routing through the system until ultimately flows reach the outfall of the major system. No storage has been accounted for within storage nodes at park areas and some street catchments that are expected to have no sags. Capture curves were defined for each catchment to restrict outlet link flows to the 2-year and 5-year rate for local streets and the school and collector roads respectively.

4.4.2 Land Use

The proposed site and adjacent Tartan Development will be developed as a mixture of low and medium density residential areas with two park areas and a school. The existing recreational area located at the northern end of the site has been included in the analysis since it sheet drains towards the site under existing conditions. A portion of Shea Road to the west also contributes stormwater runoff to the proposed system. The tributary drainage area was divided into semi-lumped drainage areas reflective of the conceptual design of the minor system. The post-development drainage scheme is indicated on **Drawing OSD-1**.

Typical impervious ratios for single family and townhouse units applied across the site were based on the *Tartan/Cavanagh Conceptual Site Servicing Plan, Stormwater Management Plan and Erosion and Sediment Control Plan* (IBI Group, March 2013), which were based on typical runoff coefficients. The overall imperviousness for the Fernbank Pond 4 drainage area is 52%, which is higher than the EMP and MSS value of 44%.

4.4.3 Design Storms

The 3 hour Chicago distribution was selected to estimate the 2-year and 5-year capture rates for the proposed subcatchments, and to assess the 100-year HGL across the proposed development. The 24 hour SCS Type II distribution was selected in the EMP as the critical storm and as such, this storm distribution was also used to assess the 100-year HGL across the site and to generate the climate change scenario where by the 100-year intensities are increased by 20%.

4.4.4 Boundary Conditions

A static backwater elevation of 106.53 m was used to assess the worst-case HGL across the site. The static backwater elevation of 106.53 m corresponds to the elevation of the existing 700 mm diameter CSP crossing Fernbank Road with a 0.5 m head (CSP inv=105.33 m).

4.4.5 Modeling Parameters

Table 4 presents the general subcatchment parameters used:

Table 4: General Subcatchment Parameters

Subcatchment Parameter	Value
Infiltration Method	Horton
Max. Infil. Rate (mm/hr)	76.2
Min. Infil. Rate (mm/hr)	13.2
Decay Constant (1/hr)	4.14
N Imperv	0.013
N Perv	0.25
Dstore Imperv (mm)	1.57
Dstore Perv (mm)	4.67
Zero Imperv (%)	0

Table 5 presents the individual parameters that vary for each of the conceptual subcatchments.

Table 5: Conceptual Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient	Subarea Routing	% Routed
C106A	1.01	292	1.0	64.3%	0.65	OUTLET	100
C106B	0.96	241	1.0	0.0%	0.20	PERVIOUS	100
C109A	0.58	362	1.0	64.3%	0.65	OUTLET	100
C116A	1.66	567	1.0	58.6%	0.61	OUTLET	100
C118A	1.04	244	1.0	58.6%	0.61	OUTLET	100
C119A	2.52	656	1.0	58.6%	0.61	OUTLET	100
C121A	1.29	315	1.0	58.6%	0.61	OUTLET	100
F108A	2.44	200	2.0	71.4%	0.70	OUTLET	100
L102A	2.52	556	1.0	58.6%	0.61	OUTLET	100
L107A	4.00	1,864	1.0	58.6%	0.61	OUTLET	100
L109B	1.61	116	2.0	0.0%	0.20	PERVIOUS	100
L109C	0.29	118	0.5	64.3%	0.65	OUTLET	100
L111A	3.52	536	1.0	50.0%	0.55	OUTLET	100
L112A	1.73	353	1.0	58.6%	0.61	OUTLET	100
L113A	3.34	816	1.0	58.6%	0.61	OUTLET	100
L121B	2.39	167	2.0	0.0%	0.20	PERVIOUS	100
L203A	6.66	1,974	1.0	58.6%	0.61	OUTLET	100
L206A	2.68	939	1.0	58.6%	0.61	OUTLET	100
L211A	9.08	2,655	1.0	50.0%	0.55	OUTLET	100
L218A	6.70	2,358	1.0	55.7%	0.59	OUTLET	100
POND	3.13	280	2.0	40.0%	0.48	OUTLET	100

1. The width parameter was measured as twice the road/rear yard swale for two-sided catchments and equal to the length of the road/rear yard swale for one-sided catchments. The width parameter for the commercial block was defined as 225m/ha as per the City of Ottawa Sewer Design Guidelines.

Table 6 summarizes the storage node parameters used in the conceptual model. All roadway catchbasins have been modeled as having an outlet invert of 2.15 m below top of grate so that the required surface storage occurs below the top of grate between a head of 1.8 m and a depth of 0.35 m. Grassed swales were assumed to have a depth of 0.6 m, however no storage was assumed within park/grassed areas. Road areas are modeled assuming catchbasin depths of 2.15 m and a flow depth of 0.35 m.

Table 6: Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Surface Storage (m³)
106A-S(1)	108.56	110.71	2.15	0
106B-S	109.10	111.90	2.80	0
109A-S	110.20	112.35	2.15	0
116A-S	109.23	111.73	2.50	66
118A-S	110.06	112.21	2.15	0
119A-S	109.73	112.23	2.50	101

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Surface Storage (m^3)
121A-S	109.72	112.22	2.50	52
108A-S	108.85	111.35	2.50	198
102A-S	108.12	110.77	2.65	100
107A-S	108.24	110.74	2.50	158
109B-S	112.00	112.60	0.60	0
109C-S	109.35	111.85	2.50	12
111A-S	108.44	110.94	2.50	140
112A-S	108.22	110.87	2.65	69
113A-S	108.53	111.03	2.50	132
121B-S	112.25	112.85	0.60	0
203A-S	109.25	111.75	2.50	265
206A-S	109.73	112.23	2.50	107
211A-S	109.75	112.25	2.50	363
218A-S	110.73	113.23	2.50	266
POND-S	104.25	107.75	3.50	N/A

1. Surface ponding in sag storage was assumed to be 40 m^3/ha.
2. School Block (storage node 108A-S) to provide on-site storage for up to the 100-year storm.

As per the Ottawa Sewer Design Guidelines (OSDG 2012), Manning's roughness values of 0.013 were used for sewer modeling and overland flow corridors representing roadways.

Table 7 summarizes the outlet link maximum flow rates for the 100-year, 3hr Chicago storm event.

Table 7: Conceptual Minor System Capture Rates

Outlet Name	Inlet Node	Outlet Node	Invert Elevation (m)	100-year Flow (L/s)
102A-IC	102A-S	108.12	315.18	315.18
106A-IC	106A-S(1)	108.56	207.00	207.00
106B-IC	106B-S	109.1	18.00	18.00
107A-IC	107A-S	108.24	504.51	504.51
108A-IC	108A-S	108.85	518.00	518.00
109A-IC	109A-S	110.2	128.00	128.00
109C-IC	109C-S	109.35	41.00	41.00
111A-IC	111A-S	108.44	375.00	375.00
112A-IC	112A-S	108.22	216.00	216.00
113A-IC	113A-S	108.53	418.00	418.00
116A-IC	116A-S	109.23	319.58	319.58
118A-IC	118A-S	110.06	193.00	193.00
119A-IC	119A-S	109.73	473.40	473.40
121A-IC	121A-S	109.72	241.40	241.40
203A-IC	203A-S	109.25	837.00	837.00
206A-IC	206A-S	109.73	337.00	337.00

Outlet Name	Inlet Node	Outlet Node	Invert Elevation (m)	100-year Flow (L/s)
211A-IC	211A-S	109.75	976.00	976.00
218A-IC	218A-S	110.73	802.00	802.00

Exit losses at manholes were set for all pipe segments based on the flow angle through the structure. Exit losses were assigned as per City guidelines (Appendix 6b), see **Table 8** below.

Table 8: Exit Loss Coefficients for Bends at Manholes

Degrees	Coefficient
11	0.060
22	0.140
30	0.210
45	0.390
60	0.640
90	1.320
180	0.020

Other parameters applied within the model include the following:

- Orifice Discharge Coefficient = 0.61 (circular)
- Weir Discharge Coefficient = 1.7

4.5 CONCEPTUAL MODEL RESULTS AND DISCUSSION

The following section summarizes the key hydrologic and hydraulic conceptual model results. For detailed model results or inputs please refer to the example input file in **Appendix C.3** and the electronic model files on the enclosed CD.

4.5.1 Proposed Development Conceptual Hydraulic Grade Line Analysis

The worst case 100-year hydraulic grade line (HGL) elevation across the proposed development and the adjacent Tartan development was estimated using the Fernbank Pond 4 PCSWMM model for the 100-year, 3 hour Chicago and the 100-year, 24 hour SCS Type II storms with a static backwater elevation of 106.53 m. **Table 9** below presents the clearance between the trunk sewer worst case HGL and the proposed road grade along the trunk sewer. The storm sewer design sheet is included in **Appendix C.1**. The climate change scenario was also run to stress-test the system.

Table 9: Fernbank Pond 4 HGL Results along Future Trunk Sewers

STM MH	Prop. Grade (m)	100-year Storms				100-year Increased by 20%	
		3 HR Chicago HGL (m)	24 HR SCS HGL (m)	Worst-Case HGL (m)	Prop. Grade-HGL Clearance (m)	24 HR SCS HGL (m)	Prop. Grade-HGL Clearance (m)
100B	108.15	107.27	107.64	107.64	0.51	107.84	0.31
101	108.48	107.27	107.64	107.64	0.84	107.85	0.63
102	108.81	107.27	107.64	107.64	1.17	107.87	0.94
103	110.43	107.70	107.83	107.83	2.60	108.09	2.34
104	110.37	107.86	107.95	107.95	2.42	108.22	2.15
105	110.14	107.87	107.96	107.96	2.18	108.23	1.91
106	110.16	107.98	108.05	108.05	2.11	108.35	1.81
107	111.95	108.17	108.21	108.21	3.74	108.56	3.39
108	111.11	108.05	108.10	108.10	3.01	108.41	2.70
109	111.00	108.37	108.37	108.37	2.63	108.42	2.58
110	111.15	108.21	108.24	108.24	2.91	108.53	2.62
111	111.24	108.87	108.87	108.87	2.37	108.89	2.35
112	110.81	108.17	108.27	108.27	2.54	108.54	2.27
113	110.95	108.43	108.53	108.53	2.42	108.80	2.15
114	111.02	108.27	108.36	108.36	2.66	108.64	2.38
115	112.02	108.37	108.46	108.46	3.56	108.75	3.27
116	111.48	108.56	108.63	108.63	2.85	108.98	2.50
117	112.00	108.64	108.69	108.69	3.31	109.07	2.93
118	110.82	108.70	108.74	108.74	2.08	109.78	1.04
119	112.54	108.77	108.84	108.84	3.70	109.10	3.44
120	113.00	108.85	108.92	108.92	4.08	109.16	3.84
121	112.59	109.21	109.21	109.21	3.38	109.31	3.28
200	113.32	108.69	108.76	108.76	4.56	109.12	4.20
201	115.71	108.82	108.88	108.88	6.83	109.30	6.41
202	115.86	108.86	108.91	108.91	6.95	109.34	6.52
203	116.79	109.01	109.05	109.05	7.74	109.50	7.29
204	113.43	108.73	108.78	108.78	4.65	109.20	4.22
205	116.54	108.85	108.88	108.88	7.66	109.36	7.18
206	116.50	109.04	109.06	109.06	7.44	109.54	6.96
207	112.93	108.42	108.50	108.50	4.43	108.81	4.12

STM MH	Prop. Grade (m)	100-year Storms				100-year Increased by 20%	
		3 HR Chicago HGL (m)	24 HR SCS HGL (m)	Worst-Case HGL (m)	Prop. Grade-HGL Clearance (m)	24 HR SCS HGL (m)	Prop. Grade-HGL Clearance (m)
208	113.70	108.47	108.55	108.55	5.15	108.86	4.84
209	114.17	108.61	108.69	108.69	5.48	109.06	5.11
210	114.12	108.64	108.71	108.71	5.41	109.09	5.03
211	114.11	108.73	108.79	108.79	5.32	109.16	4.95
212	113.31	108.97	109.03	109.03	4.28	109.26	4.05
213	113.94	109.10	109.15	109.15	4.79	109.36	4.58
214	113.75	109.13	109.17	109.17	4.58	109.38	4.37
215	113.69	109.20	109.23	109.23	4.46	109.42	4.27
216	113.16	109.29	109.31	109.31	3.85	109.48	3.68
217	114.15	109.34	109.36	109.36	4.79	109.51	4.64
218	114.20	109.59	109.59	109.59	4.61	109.68	4.52

The model results indicate that there is sufficient clearance between the worst case 100-year HGL and the proposed road grades. Detailed grading of the future developments should be based on the above results to ensure that a minimum clearance of 0.3 m is provided between all under side of footings (USFs) and the 100-year HGL, and that no basement flooding occurs in the climate change scenario.

4.5.2 Major Flow

It is proposed that all major flow from the proposed development and the adjacent Tartan development cascade to the Fernbank SWM Pond 4. There are two major flow outlets proposed to the pond. The maximum overland flow was evaluated at these two downstream locations. The western major flow inlet is proposed to be a walkway block through the proposed development, while the eastern major flow inlet is proposed to be through Shea Road as shown on **Drawing OSD-1**.

The western major flow inlet through the walkway was modeled in PCSWMM as a trapezoidal channel with a 4m-wide bottom, 3:1 side slopes, longitudinal slope of 1.9%, and 0.5 m depth. Similarly, the eastern major flow inlet from Shea Road was modeled as a trapezoidal channel with 2m-wide bottom, 10:1 side slopes, 2.8% longitudinal slope, and 0.5 m depth. The maximum normal flow depth and velocity have been obtained from PCSWMM and the results are presented in **Table 10** below for the 100-year, 3 hour Chicago storm which is commonly used to evaluate the urban component of dual drainage, specifically on-site detention.

Table 10: 100-Year, 3hr Chicago Overland Flow Results

Location	Peak Flow (L/s)	Depth (m)
Western Major System Inlet to Pond- Walkway	8,226	0.49
Western Inlet Most Downstream Street	7,265	0.33
Eastern Major System Inlet to Pond – Shea Road	2,954	0.19
Eastern Inlet Most Downstream Street	2,983	0.19

5.0 GEOTECHNICAL CONSIDERATIONS

A Geotechnical Report was prepared by Golder Associates for the subject lands in October 2011. The geotechnical investigation concluded that the site consists of a discontinuous sand layer underlain by glacial till overlying limestone bedrock which exists at depth from 3 meters to the surface. There are no grade raise restrictions for house construction on this site. A Permit to Take Water (PTTW) was recommended to be obtained for the site servicing work due to the potential for groundwater inflow in areas of rock excavation.

6.0 EROSION CONTROL DURING CONSTRUCTION

Erosion and sediment controls must be in place during construction. The following recommendations to the contractor will be included in contract documents.

1. Until the local storm sewer and SWM pond are constructed, groundwater in trenches will be pumped into a filter mechanism prior to release to the environment. After construction of the SWM facility, any construction dewatering will be routed to the nearest storm sewer.
2. Seepage barriers to be constructed in any temporary drainage ditches.
3. Install a silt fence along the site perimeter.
4. Limit extent of exposed soils at any given time.
5. Re-vegetate exposed areas as soon as possible.
6. Minimize the area to be cleared and grubbed.
7. Protect exposed slopes with plastic or synthetic mulches.
8. Provide sediment traps and basins during dewatering.
9. Install sediment traps (such as SiltSack® by Terrafix) between catchbasins and frames.
10. Plan construction at proper time to avoid flooding.

The contractor will, at every rainfall, complete inspections and guarantee proper performance. The inspection is to include:

1. Verification that water is not flowing under silt barriers.
2. Clean and change silt traps at catchbasins.

7.0 UTILITIES

As the subject site is bound by existing residential development to the west, Hydro, Bell, Gas and Cable servicing for the proposed development should be readily available through existing infrastructure. It is anticipated that existing infrastructure will be sufficient to provide the means of distribution for the proposed site. Exact size, location and routing of utilities, along with determination of any off-site works required for redevelopment, will be finalized after design circulation.

8.0 APPROVALS

The City of Ottawa will review and approve most development applications as they relate to provision of water supply, wastewater collection and disposal, and stormwater conveyance and treatment.

Ontario Ministry of Environment and Climate Change (MOECC) Environmental Compliance Approvals (ECA) will be required for the proposed subdivision works related to stormwater management, inlet control devices, storm sewers and sanitary sewers. The Rideau Valley Conservation Authority (RVCA) will be circulated on this submission.

An MOECC Permit to Take Water (PTTW) may be required for the site. The geotechnical consultant shall confirm at the time of application that a PTTW is required.

The Rideau Valley Conservation Authority (RVCA) will issue all required permits for the stormwater management facility.

9.0 CONCLUSIONS

9.1 WATER SERVICING

Based on the findings of the report, the proposed network is capable of servicing the development area and meets all servicing requirements as per City of Ottawa standards under typical demand conditions (peak hour and average day conditions) as well as under emergency fire demand conditions (maximum day + fire flow). The available fire flow is anticipated to range between 10,683 – 48,622 L/min. The site will be serviced via three watermain connections; a 200 mm diameter connection to Samuel Mann Avenue, a 400 mm diameter watermain on Fernbank Road, and a 300 mm diameter watermain on Shea Road south of the intersection with Abbott Street East.

9.2 SANITARY SERVICING

Wastewater from the proposed Shea Road Lands Development will be conveyed through the sanitary trunk sewers within the future CTR lands which ultimately connect to the existing Fernbank trunk sewer on the Trans Canada Trail.

A higher level sanitary sewer is proposed in all right of ways where the trunk sanitary sewers exceed 5 m in depth to avoid deep residential service connections to the main and facilitate any potential future service repairs. The high level sanitary sewer will be installed along the same alignment as the trunk sanitary sewer and will utilize the same manholes while providing connection to the main sanitary trunk via external drop structures.

The site sanitary trunk sewers will be sized to service the proposed development, the future low density residential block (area R19B), the Tartan development west of the site, as well as OPA Area 6 expansion lands which are estimated to generate approximately 84 L/s of sewage peak flows. Additionally, an allowance of 108 L/s has been included from the existing Laird Street Pump Station.

9.3 STORMWATER SERVICING

The proposed stormwater management plan is in compliance with the requirements outlined in the background documents, the City of Ottawa Sewer Design Guidelines and the Ministry of the Environment and Climate Change Stormwater Management Planning and Design Manual.

Capture curves were defined for each catchment to restrict inflow rates to the sewer to the 2-year runoff for local streets and the 5-year runoff for the school block and for collector roads (Shea Road and Cope Drive) as per the City design criteria. Major system peak flows from the entire site will be directed towards the proposed Fernbank SWM Pond 4.

Quantity and 'Enhanced' quality control will be provided in the proposed Fernbank SWM Pond 4 to restrict peak flows from the site to the target peak outflows and to achieve 80% TSS removal prior to discharging into the Faulkner Municipal Drain.

9.4 UTILITIES

Utility infrastructure exists within the general area of the subject site. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities will be finalized at the detailed design stage.

Appendix A POTABLE WATER SERVICING ANALYSIS

A.1 BOUNDARY CONDITIONS

From:	Surprenant, Eric
To:	Rathnasooriya, Thakshika
Cc:	Paerez, Ana
Subject:	RE: Hydraulic Boundary Conditions - Shea Road Lands Development
Date:	Monday, December 18, 2017 9:06:05 AM
Attachments:	image001.gif Shea Road Lands Development.docx

Hello Thakshika,

Please refer to the attached as it relates to the your request for boundary conditions for the above development.

Thanks

Eric Surprenant, C.E.T. / 613 580-2424 ext.:27794
Project Manager, Infrastructure Approvals
Development Review Suburban Services Branch
Planning, Infrastructure and Economic Development Dept.

Gestionaire de projets, Approbation de l'infrastructure
Examen des demandes d'aménagement (Services Suburbains Ouest)
Services de la planification, de l'infrastructure et du développement économique

City of Ottawa | Ville d'Ottawa
613.580.2424 ext./poste 27794
ottawa.ca/planning / ottawa.ca/urbanisme

From: Rathnasooriya, Thakshika [mailto:Thakshika.Rathnasooriya@stantec.com]
Sent: December 11, 2017 10:30 AM
To: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Cc: Paerez, Ana <Ana.Paerez@stantec.com>
Subject: Hydraulic Boundary Conditions - Shea Road Lands Development

Good morning Eric,

I am looking for watermain hydraulic boundary conditions for the proposed Shea Road Lands Development which is located at the north-west quadrant of the intersection of Shea Road and Fernbank road. We anticipate 3 watermain connections to the proposed Tartan and Cavanagh development as shown in the attached figure.

Connection 1 – existing 200mm
Connection 2 – proposed 400mm (Fernbank Road).
Connection 3 – existing 300mm stub south of the intersection of Shea Road and Abbott Street (Shea Road).

The intended land use is a school block, park land, and mixed residential development consisting of for 343 single family homes and 570 semi-detached or townhomes , as well as 2.58 ha of land designated for low density residential, assumed with 60 units/ha for a total of 155 units.

Please confirm Area 6 (plan attached) demands have been included when generating boundary conditions(not included in demands below).

Estimated domestic demands and fire flow requirements for the site are as follows:
Average Day Demand – 13.5L/s
Max Day Demand - 33.4L/s
Peak Hour Demand - 73.2L/s
Fire Flow Requirement per Technical Bulletin ISDTB-2014-02 section 4.2.11– capped at 167L/s (10,000 L/min) for single detached dwellings, side-by-side town and row houses provided a minimum separation distance between the backs of adjacent units by 10m, and 15,000 L/min for the proposed school block.

Thanks,

Shika Rathnasooriya
Engineering Intern
Stantec
400 - 1331 Clyde Avenue, Ottawa ON K2C 3G4
Phone: (613) 722-4420
Thakshika.Rathnasooriya@stantec.com

Boundary Conditions Shea Road Lands Development

Information Provided

Date provided: 15 December 2017

Scenario	Demand	
	L/min	L/s
Average Daily Demand	810	13.5
Maximum Daily Demand	2004	33.4
Peak Hour	4392	73.2
Fire Flow Demand # 1	10000	166.7
Fire Flow Demand # 2	15000	250.0

Location

Results

Connection 1 - Samuel Mann Ave

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	160.6	63.7
Peak Hour	153.6	53.8
Max Day plus Fire (10,000 l/min)	149.4	47.8
Max Day plus Fire (15,000 l/min)	141.1	36.0

[1] Ground Elevation = 115.7 m

Connection 2 - Fernbank Rd

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	160.5	68.9
Peak Hour	153.2	58.5
Max Day plus Fire (10,000 l/min)	148.4	51.6
Max Day plus Fire (15,000 l/min)	139.4	38.8

[1] Ground Elevation = 112.1 m

Connection 3 - Shea Rd

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	160.8	68.2
Peak Hour	154.7	59.6
Max Day plus Fire (10,000 l/min)	154.8	59.6
Max Day plus Fire (15,000 l/min)	151.5	55.0

[1] Ground Elevation = 112.9 m

Notes:

1) As per the Ontario Building Code in areas that may be occupied, the static pressure at any fixture shall not exceed 552 kPa (80 psi.) Pressure control measures to be considered are as follows, in order of preference:
   a) If possible, systems to be designed to residual pressures of 345 to 552 kPa (50 to 80 psi) in all occupied areas outside of the public right-of-way without special pressure control equipment.
   b) Pressure reducing valves to be installed immediately downstream of the isolation valve in the home/ building, located downstream of the meter so it is owner maintained.

Disclaimer

The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.

A.2 HYDRAULIC MODEL PARAMETERS

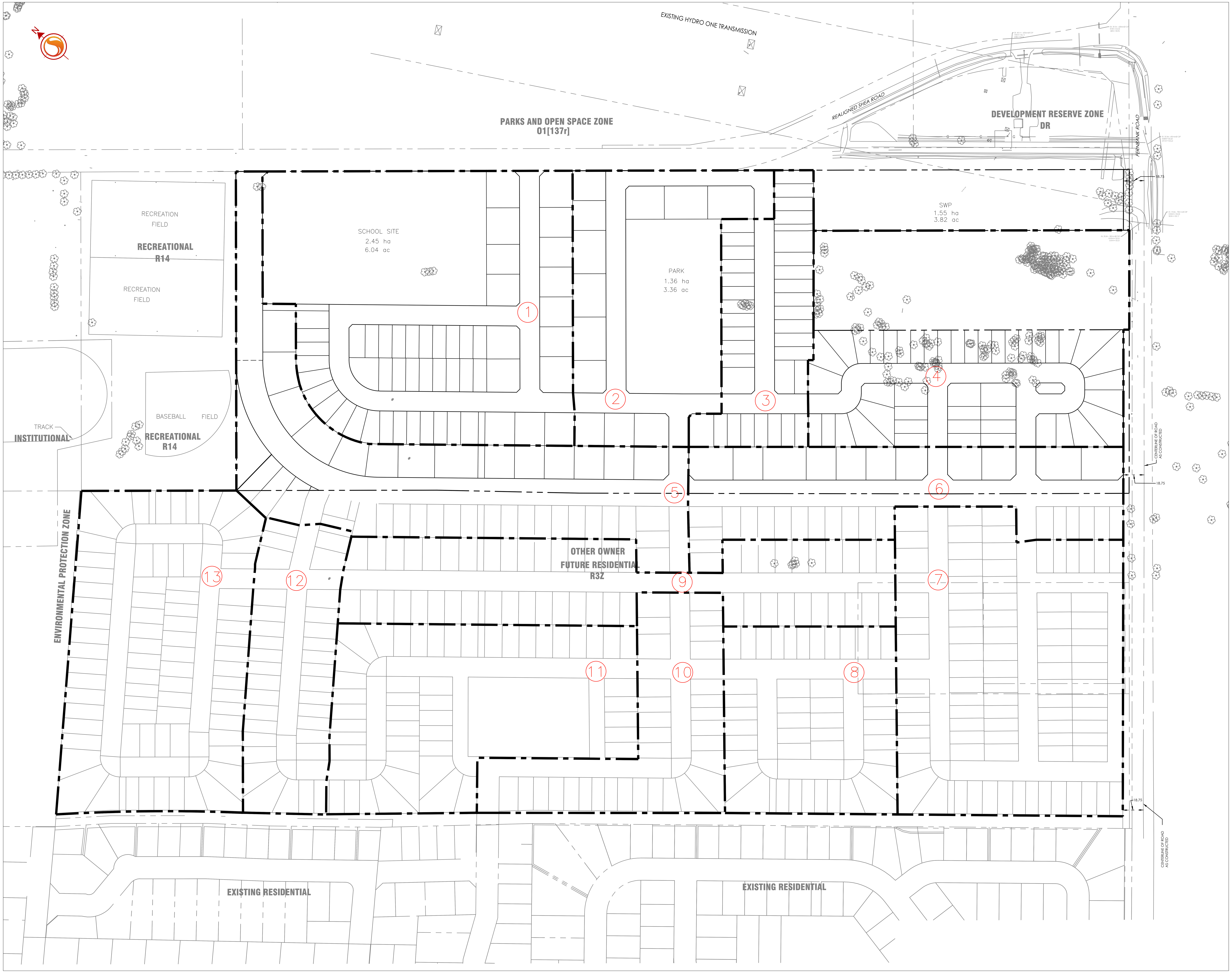
Shea Road - Domestic Water Demand Estimates

Densities as per City Guidelines:

LD	3.4	ppu
Towns and Semis	2.7	ppu
Singles	3.4	ppu

Building ID	Area (ha)	Population	Daily Rate of Demand [1]	Avg Day Demand [2]		Max Day Demand [3]		Peak Hour Demand [3]	
				(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Residential 1		298	350	72.4	1.21	181.0	3.02	398.2	6.64
School	2.45	-	15000	25.5	0.43	38.3	0.64	68.9	1.15
Total				**97.9**	**1.6**	**219.3**	**3.7**	**467.1**	**7.8**
Residential 2		159	350	38.7	0.65	96.8	1.61	213.0	3.55
Residential 3		118	350	28.6	0.48	71.5	1.19	157.2	2.62
Residential 4		153	350	37.2	0.62	93.0	1.55	204.5	3.41
LD	2.58	155	350	37.6	0.63	94.1	1.57	206.9	3.45
Total				**74.8**	**1.2**	**187.0**	**3.1**	**411.5**	**6.9**
Residential 5		317	350	77.1	1.28	192.7	3.21	424.0	7.07
Residential 6		227	350	55.2	0.92	138.1	2.30	303.9	5.06
Residential 7		372	350	90.5	1.51	226.3	3.77	497.8	8.30
Residential 8		150	350	36.4	0.61	90.9	1.52	200.0	3.33
Residential 9		286	350	69.6	1.16	173.9	2.90	382.6	6.38
Residential 10		115	350	27.9	0.47	69.8	1.16	153.5	2.56
Residential 11		190	350	46.1	0.77	115.3	1.92	253.7	4.23
Residential 12		120	350	29.1	0.48	72.7	1.21	159.9	2.66
Residential 13		252	350	61.2	1.02	152.9	2.55	336.3	5.61
Total Site :				**733.1**	**12.2**	**1807.2**	**30.1**	**3960.5**	**66.0**

1 Average day water demand for residential areas equal to 350 L/cap/d and 15,000 L/ha/d for the school block
2 City of Ottawa water demand criteria used to estimate peak demand rates for residential areas are as follows:
maximum day demand rate = 2.5 x average day demand rate for residential, 1.5 for Institutional
maximum hour demand rate = 2.2 x maximum day demand rate for residential, 1.8 for institutional



EXISTING HYDRO ONE TRANSMISSION
PARKS AND OPEN SPACE ZONE
O1[137r]
REALIGNED SHEA ROAD
DEVELOPMENT RESERVE ZONE
DR
FERNBANK ROAD
RECREATION FIELD
RECREATIONAL
R14
RECREATION FIELD
SCHOOL SITE
2.45 ha
6.04 ac
PARK
1.36 ha
3.36 ac
SWP
1.55 ha
3.82 ac
TRACK
INSTITUTIONAL
BASEBALL FIELD
RECREATIONAL
R14
ENVIRONMENTAL PROTECTION ZONE
OTHER OWNER
FUTURE RESIDENTIAL
R3Z
1
2
3
4
5
6
7
8
9
10
11
12
13
18.75
CENTRELINE OF ROAD AS CONSTRUCTED
EXISTING RESIDENTIAL
EXISTING RESIDENTIAL

A.3 HYDRAULIC MODELING RESULTS

Hydraulic Model Results - Average Day Analysis

Junction Results

ID	Demand	Elevation	Head	Pressure	
	(L/s)	(m)	(m)	(psi)	(Kpa)
1	1.60	112.00	150.56	54.81	377.90
10	0.47	113.71	148.35	49.24	339.50
11	0.77	114.00	148.70	49.33	340.12
12	0.48	113.31	149.39	51.29	353.63
13	1.02	113.98	149.39	50.34	347.08
15	0.00	110.50	153.24	60.75	418.86
16	0.00	112.40	149.91	53.32	367.63
17	0.00	111.04	148.21	52.84	364.32
18	0.00	111.46	144.98	47.65	328.54
19	0.00	111.60	144.86	47.29	326.06
2	0.65	111.21	148.69	53.28	367.35
20	0.00	112.00	144.18	45.74	315.37
21	0.00	114.00	148.75	49.40	340.60
22	0.00	114.50	149.07	49.14	338.81
23	0.00	115.50	145.29	42.35	291.99
24	0.00	113.43	144.81	44.62	307.65
25	0.00	115.00	146.47	44.74	308.47
26	0.00	114.50	149.02	49.07	338.33
27	0.00	114.20	147.60	47.48	327.37
28	0.00	114.10	149.39	50.17	345.91
29	0.00	113.00	150.10	52.75	363.70
3	0.48	110.81	147.79	52.56	362.39
30	0.00	111.50	153.82	60.16	414.79
4	1.20	111.20	145.74	49.10	338.53
5	1.28	112.00	148.18	51.44	354.67
6	0.92	111.52	145.72	48.62	335.23
7	1.51	113.30	146.45	47.13	324.95
8	0.61	114.50	147.25	46.55	320.95
9	1.16	112.88	148.24	50.27	346.60

Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)
12	5	6	232.94	297	120	117.65	1.70
13	6	20	84.02	297	120	158.49	2.29
16	15	1	158.26	204	110	51.77	1.58
17	1	16	406.64	204	110	14.48	0.44
18	1	16	76.53	204	110	35.69	1.09
19	16	2	76.28	204	110	50.17	1.53
20	2	3	534.76	204	110	14.90	0.46
21	3	17	76.71	204	110	-28.30	0.87
22	2	17	59.39	204	110	34.62	1.06
23	17	5	78.45	204	110	6.32	0.19
24	20	1002	131.64	297	120	228.92	3.30
25	3	4	172.57	204	110	42.71	1.31
26	4	18	84.28	204	110	36.86	1.13
27	18	19	106.46	204	110	11.70	0.36
28	18	19	25.78	204	110	25.16	0.77
29	19	20	76.25	204	110	36.86	1.13
30	1001	22	83.80	297	120	69.30	1.00
31	10	9	76.64	297	120	39.95	0.58
32	9	5	83.66	297	120	26.57	0.38
33	21	10	158.33	297	120	54.22	0.78
34	22	21	99.06	297	120	61.96	0.89
35	10	27	74.72	204	110	39.07	1.20
36	27	8	77.24	204	110	25.45	0.78
37	8	25	79.52	204	110	38.46	1.18
38	9	7	233.50	204	110	33.72	1.03
39	27	8	245.92	204	110	13.62	0.42
41	23	24	166.24	204	110	19.81	0.61
43	24	20	84.08	204	110	33.57	1.03
45	23	24	326.37	204	110	13.76	0.42
47	23	25	155.57	204	110	-33.57	1.03
49	25	7	86.57	204	110	4.88	0.15
51	7	6	80.24	204	110	37.10	1.13
53	22	26	114.16	204	110	7.34	0.22
55	26	11	126.60	204	110	18.29	0.56
57	21	11	88.50	204	110	7.74	0.24
59	11	10	79.05	204	110	25.26	0.77
61	12	9	346.52	204	110	21.51	0.66
63	26	12	395.25	204	110	-10.96	0.34
65	13	28	397.34	204	110	0.09	0.00
67	28	13	163.60	204	110	0.09	0.00
69	13	12	76.83	204	110	-1.02	0.03
71	29	5	324.23	297	120	86.04	1.24
73	12	29	91.71	204	110	-33.96	1.04
74	4	6	103.34	204	110	4.66	0.14
76	30	29	338.30	297	120	120.00	1.73
77	1004	30	327.65	297	120	171.77	2.48
78	30	15	250.82	297	120	51.77	0.75

Hydraulic Model Results -Peak Hour Analysis

Junction Results

ID	Demand	Elevation	Head	Pressure	
	(L/s)	(m)	(m)	(psi)	(Kpa)
1	7.80	112.00	149.44	53.22	366.94
10	2.56	113.71	147.60	48.18	332.19
11	4.23	114.00	148.10	48.48	334.26
12	2.66	113.31	148.60	50.16	345.84
13	5.61	113.98	148.58	49.18	339.09
15	0.00	110.50	152.56	59.79	412.24
16	0.00	112.40	148.83	51.79	357.08
17	0.00	111.04	147.32	51.57	355.56
18	0.00	111.46	144.23	46.59	321.23
19	0.00	111.60	144.13	46.25	318.88
2	3.55	111.21	147.69	51.86	357.56
20	0.00	112.00	143.54	44.84	309.16
21	0.00	114.00	148.22	48.65	335.43
22	0.00	114.50	148.76	48.70	335.78
23	0.00	115.50	144.52	41.25	284.41
24	0.00	113.43	144.10	43.60	300.61
25	0.00	115.00	145.55	43.43	299.44
26	0.00	114.50	148.52	48.36	333.43
27	0.00	114.20	146.74	46.26	318.95
28	0.00	114.10	148.58	49.01	337.91
29	0.00	113.00	149.34	51.66	356.19
3	2.62	110.81	146.85	51.23	353.22
30	0.00	111.50	153.23	59.33	409.07
4	6.90	111.20	144.88	47.88	330.12
5	7.07	112.00	147.32	50.21	346.19
6	5.06	111.52	144.88	47.43	327.02
7	8.30	113.30	145.51	45.79	315.71
8	3.33	114.50	146.34	45.26	312.06
9	6.38	112.88	147.41	49.09	338.47

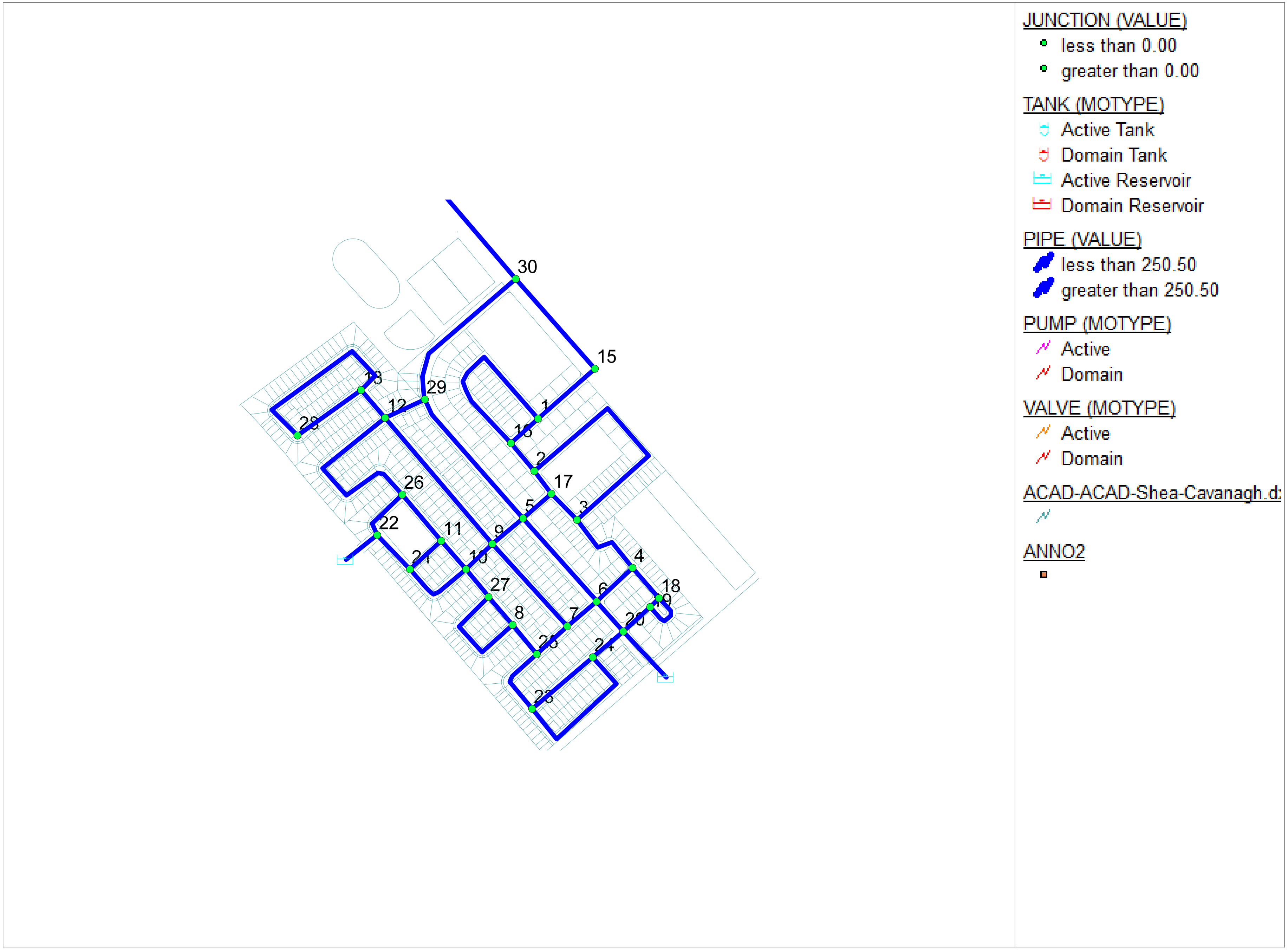
Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)
12	5	6	232.94	297	120	116.87	1.69
13	6	20	84.02	297	120	146.84	2.12
16	15	1	158.26	204	110	56.19	1.72
17	1	16	406.64	204	110	13.97	0.43
18	1	16	76.53	204	110	34.42	1.05
19	16	2	76.28	204	110	48.39	1.48
20	2	3	534.76	204	110	14.41	0.44
21	3	17	76.71	204	110	-29.96	0.92
22	2	17	59.39	204	110	30.43	0.93
23	17	5	78.45	204	110	0.47	0.01
24	20	1002	131.64	297	120	212.00	3.06
25	3	4	172.57	204	110	41.75	1.28
26	4	18	84.28	204	110	33.92	1.04
27	18	19	106.46	204	110	10.77	0.33
28	18	19	25.78	204	110	23.16	0.71
29	19	20	76.25	204	110	33.92	1.04
30	1001	22	83.80	297	120	98.71	1.42
31	10	9	76.64	297	120	54.44	0.79
32	9	5	83.66	297	120	35.03	0.51
33	21	10	158.33	297	120	68.72	0.99
34	22	21	99.06	297	120	81.95	1.18
35	10	27	74.72	204	110	42.10	1.29
36	27	8	77.24	204	110	27.43	0.84
37	8	25	79.52	204	110	38.77	1.19
38	9	7	233.50	204	110	34.86	1.07
39	27	8	245.92	204	110	14.68	0.45
41	23	24	166.24	204	110	18.43	0.56
43	24	20	84.08	204	110	31.24	0.96
45	23	24	326.37	204	110	12.81	0.39
47	23	25	155.57	204	110	-31.24	0.96
49	25	7	86.57	204	110	7.53	0.23
51	7	6	80.24	204	110	34.10	1.04
53	22	26	114.16	204	110	16.76	0.51
55	26	11	126.60	204	110	21.38	0.65
57	21	11	88.50	204	110	13.23	0.40
59	11	10	79.05	204	110	30.38	0.93
61	12	9	346.52	204	110	21.84	0.67
63	26	12	395.25	204	110	-4.62	0.14
65	13	28	397.34	204	110	0.09	0.00
67	28	13	163.60	204	110	0.09	0.00
69	13	12	76.83	204	110	-5.61	0.17
71	29	5	324.23	297	120	88.44	1.28
73	12	29	91.71	204	110	-34.73	1.06
74	4	6	103.34	204	110	0.92	0.03
76	30	29	338.30	297	120	123.17	1.78
77	1004	30	327.65	297	120	179.36	2.59
78	30	15	250.82	297	120	56.19	0.81

Hydraulic Model Results -Fire Flow Analysis (167 L/s)

ID	Static Demand	Static Pressure		Static Head	Fire-Flow Demand	Residual Pressure		Available Flow at Hydrant	Available Flow Pressure	
	(L/s)	(psi)	(Kpa)	(m)	(L/s)	(psi)	(Kpa)	(L/s)	(psi)	(Kpa)
1	3.7	54.29	374.32	150.19	250	23.12	159.41	268.82	20	137.90
10	1.16	48.91	337.22	148.11	167	43.77	301.79	759.41	20	137.90
11	1.92	49.07	338.33	148.52	167	42.30	291.65	448.30	20	137.90
12	1.21	50.92	351.08	149.13	167	40.02	275.93	331.98	20	137.90
13	2.55	49.96	344.46	149.13	167	22.43	154.65	178.05	20	137.90
2	1.61	52.82	364.18	148.36	167	40.85	281.65	323.37	20	137.90
3	1.19	52.13	359.43	147.48	167	40.54	279.52	325.64	20	137.90
4	3.1	48.70	335.78	145.46	167	40.10	276.48	396.72	20	137.90
5	3.21	51.04	351.91	147.91	167	45.19	311.58	810.38	20	137.90
6	2.3	48.23	332.54	145.45	167	42.43	292.55	783.36	20	137.90
7	3.77	46.69	321.92	146.15	167	38.16	263.11	404.60	20	137.90
8	1.52	46.14	318.13	146.95	167	34.69	239.18	290.19	20	137.90
9	2.9	49.89	343.98	147.98	167	44.41	306.20	781.29	20	137.90

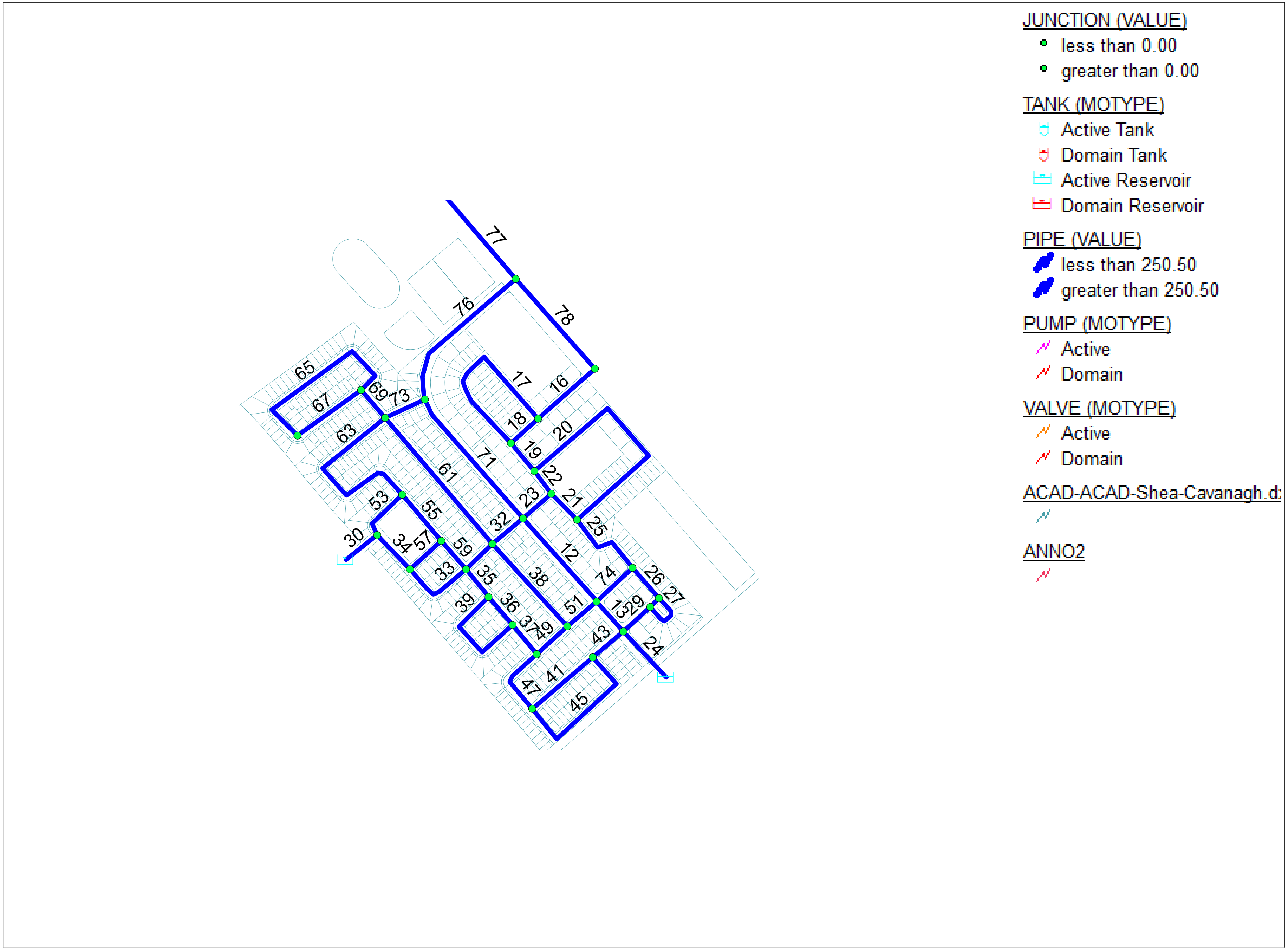
160400900 - Junction ID's



Prepared By:

Date: Wednesday, March 14, 2018

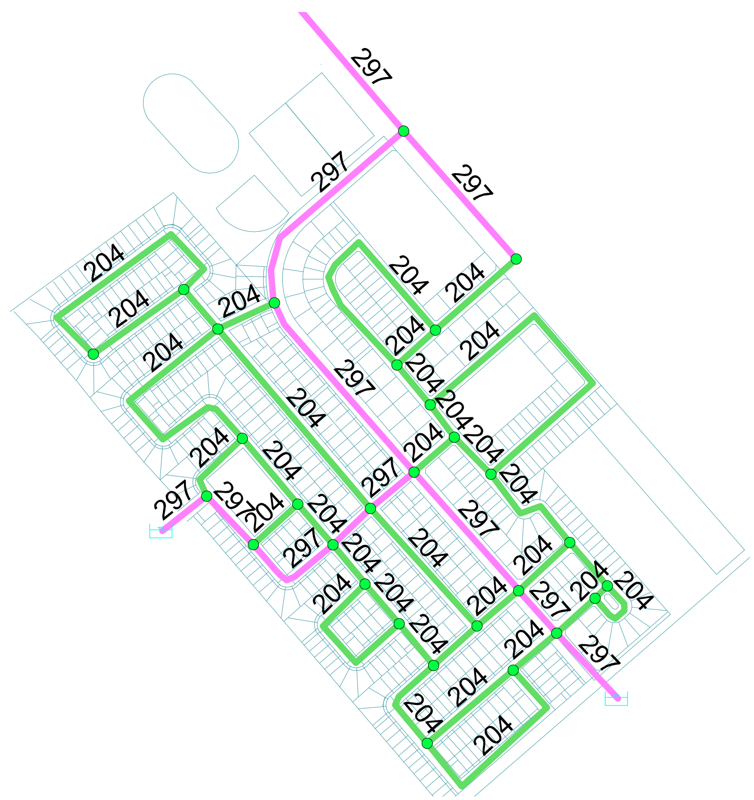
160400900 - Pipe ID's



Prepared By:

Date: Wednesday, March 14, 2018

160400900 - Pipe Diameters



JUNCTION (VALUE)
- less than 0.00
- greater than 0.00

TANK (MOTYPE)
- Active Tank
- Domain Tank
- Active Reservoir
- Domain Reservoir

PIPE (VALUE)
- less than 250.50
- greater than 250.50

PUMP (MOTYPE)
- Active
- Domain

VALVE (MOTYPE)
- Active
- Domain

ACAD-ACAD-Shea-Cavanagh.d:

Prepared By:

Date: Wednesday, March 14, 2018

A.4 WATERMAIN DESIGN BACKGROUND REPORT EXCERPTS

2. WATER SUPPLY

2.1 Existing Conditions

The Fernbank Community is located within the City's 3W Pressure Zone which includes most of Kanata and Stittsville and is one of the most rapidly growing areas in the City. Potable water to this area is pressurized at the Glen Cairn Pump Station where a major water storage reservoir (Glen Cairn Reservoir) is located. Two of the major watermains into this pressure zone from the pump station are located along Hazeldean Road and Terry Fox Drive. Another main adjacent to the subject site is located in Abbott Street and the Trans Canada Trail. In support of the FCDP, the June 24, 2009 MSS completed a review of the existing water plan adjacent to area and made recommendations for improvements and expansion to the City's water transmission and distribution system to support the proposed development. Figure 4 indicates the limits of existing watermains in the vicinity of the subject property.

2.2 Master Servicing Study

The Master Servicing Study recommended a conceptual water plan for the FCDP. A copy of the recommended plan, Watermain Layout Drawing No. 101108-WM, Revision 3, is included in Appendix B. For the subject lands, there are two connections to existing 200 mm diameter mains shown on the MSS. One is on Fernbank Road at the south end of the site and the other at Samuel Mann Avenue at the west side of the site. At the north east corner of the site the MSS identifies a connection to an existing 300 mm diameter watermain north on Shea Road. Along Shea Road in the subject area the MSS shows a 300 mm watermain with connections to the east at both ends.

2.3 Design Criteria

In order to determine the watermain plan needed to adequately service the subject site, a hydraulic model was prepared using H20 MAP software by MWH Soft Inc. The City of Ottawa supplied boundary conditions at Fernbank Road and Samuel Mann Avenue.

The following parameters were also used in the analysis for the subject site:

Residential:

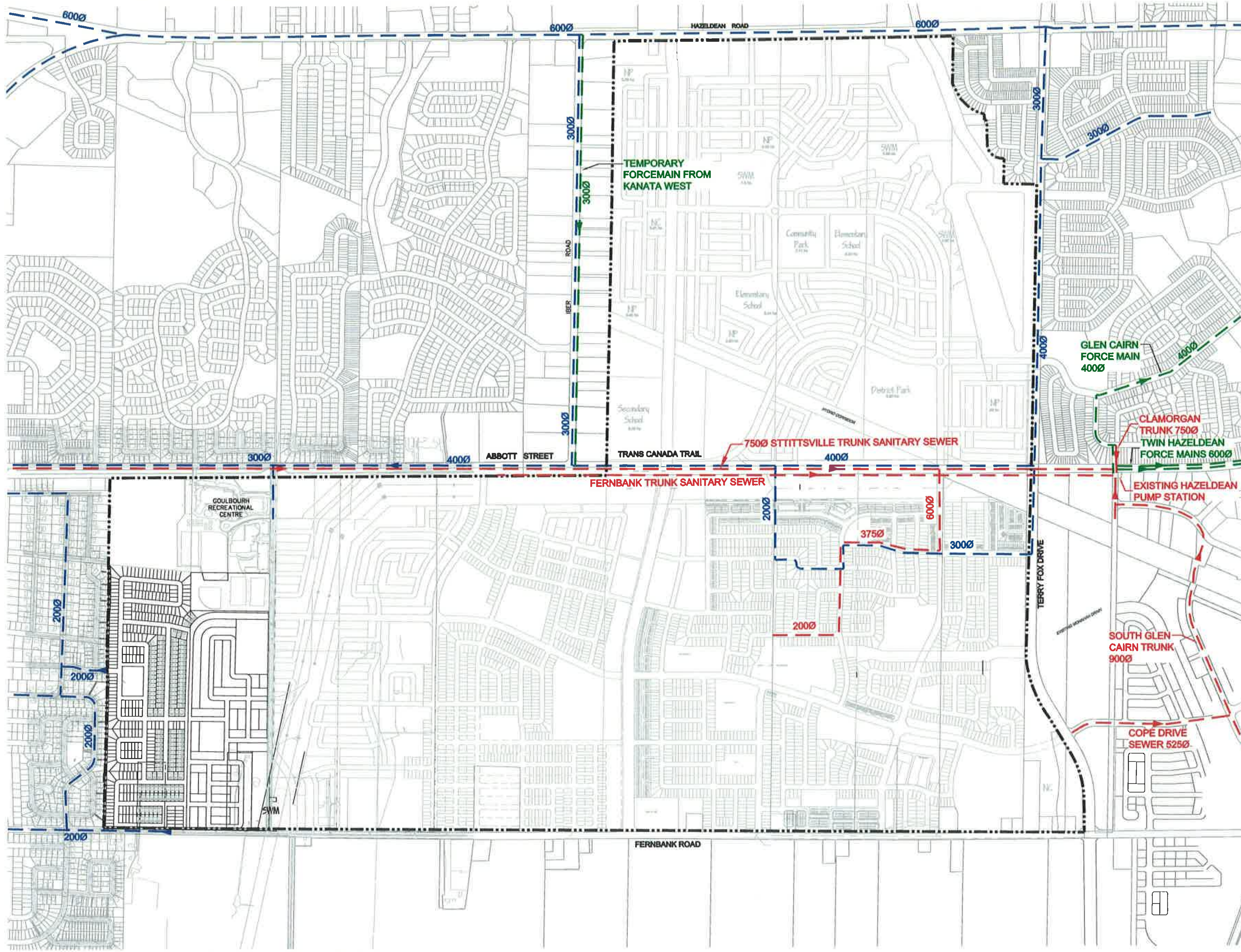
- Average Daily Demand (ADD) — 350 l/cap/day
- Maximum Daily Demand (MDD) – 2 X MDD — 875 l/cap/day
- Peak Hourly Demand – 2.2 X MDD — 1925 l/cap/day
- Fire Demand - singles & townhouse — 133 l/s (8000 l/min)

Institutional

- Average Daily Demand (ADD) — 15,000 l/ha/day
- Maximum Daily Demand (MDD) – 1.5 (ADD) — 22,500 l/ha/day
- Maximum Hourly Demand – 1.8 (MDD) — 40,500 l/ha/day
- Fire Demand — 250 l/s (15,000 l/min)

Hydraulic Gradient

- Minimum – max hour — 275 kPa
- Minimum – max day and fire — 140 kPa



LEGEND

- 300Ø EXISTING WATERMAIN AND DIAMETER
- 300Ø EXISTING SANITARY SEWER AND DIAMETER
- 300Ø EXISTING FORCEMAINS AND DIAMETER
- LIMITS OF FERNBANK COMMUNITY DESIGN PLAN

J:\11218_FernbankLand\5.9 Drawings\59civil\current\ReportFigs_2013-01-29\11218-Fig1-5base.dwg Sheet Set: ####

Plot Style: AIA STANDARD COLOR-HALF.CTB Plot Scale: 1:1 Plotted At: Feb. 13, 13 9:32 AM Printed By: DON SIURNA Last Saved By: DSIURNA Last Saved At: Feb. 13, 13

IBI GROUP

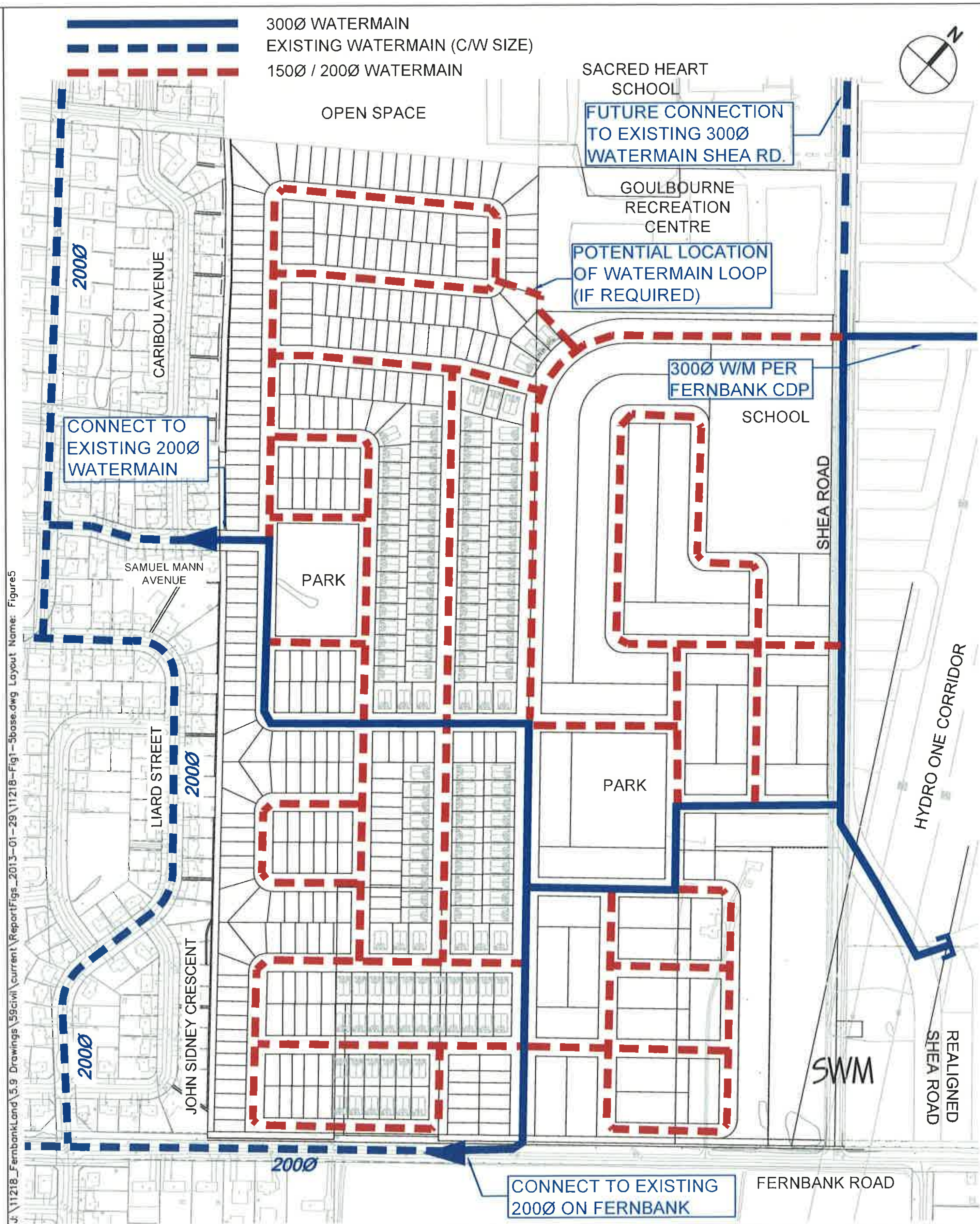
Scale	Project Title	Drawing Title	Sheet No.
1 : 15 000	CONCEPTUAL SITE SERVICING STUDY SHEA ROAD LANDS FERNBANK COMMUNITY	EXISTING WATERMAINS AND SANITARY SEWERS	FIGURE 4

A copy of the hydraulic analysis report and details on boundary conditions are included in Appendix B.

2.4 Proposed Water Plan

The proposed watermain layout for the subject lands is shown on Figure 5. As per the MSS connections to existing mains are shown at Fernbank Road and Samuel Mann Avenue. The two connection locations are joined together with a 300 mm watermain which will be part of the first phase of construction. A 300 mm watermain will be extended from the connection points to Shea Road. As per the MSS, a 300 mm watermain will be constructed on Shea Road with future connections to the north and east of the subject lands. The remaining watermains will be 150 mm or 200 mm diameter determined by hydraulic modelling during detailed design.

Results of the preliminary hydraulic modeling included in Appendix B shows that the peak hour pressures and fire flows exceed the City criteria. A check with the maximum hydraulic grade line under basic day conditions has all areas less than 550 kPa so that pressure reducing valves will not be required on this site.



Plot Style: AIA STANDARD COLOR-HALF.CTB Plot Scale: 1:1 Plotted At: Feb. 13, 13 10:07 AM Printed By: DON SIURNA Last Saved By: DSIURNA Last Saved At: Feb. 13, 13

IBI GROUP

CONCEPTUAL SITE SERVICING STUDY
SHEA ROAD LANDS
FERNBANK COMMUNITY

PROPOSED WATER PLAN

FIGURE 5

Appendix B SANITARY SEWER CALCULATIONS

B.1 CONCEPTUAL SANITARY SEWER DESIGN SHEET

	SUBDIVISION: Shea Road Lands		SANITARY SEWER DESIGN SHEET (City of Ottawa)		DESIGN PARAMETERS					
Stantec	DATE:	April 11, 2018			MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	350 L/p/day	MINIMUM VELOCITY	0.60 m/s
	REVISION:	0			MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	50,000 L/ha/day	MAXIMUM VELOCITY	3.00 m/s
	DESIGNED BY:	WAJ	FILE NUMBER:	160400900	PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 L/ha/day	MANNINGS n	0.013
	CHECKED BY:	AMP			PEAKING FACTOR (COMM., INST.):	1.5	INDUSTRIAL (LIGHT)	35,000 L/ha/day	BEDDING CLASS	B
					PERSONS / SINGLE[4]	3.3	INSTITUTIONAL	50,000 L/ha/day	MINIMUM COVER	2.50 m
					PERSONS / TOWNHOME[4]	2.5	INFILTRATION	0.28 L/s/ha		
					PERSONS / APARTMENT[4]	1.8				

LOCATION			RESIDENTIAL AREA AND POPULATION									COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	UNITS SINGLE	UNITS TOWN	UNITS APT	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)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
R24A, AREA 6*, LAIRD STREET PUMP STATION*	24	23	0.68	6	11	0	47	0.68	47	4.00	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.68	0.68	0.2	193.0	106.8	600	CONCRETE	SDR 35	0.15	250.7	**76.96%**	0.86	0.84
Future R306A	FUT306	FUT305	2.68	21	46	0	184	2.68	184	4.00	3.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	2.68	2.68	0.8	3.7	160.9	200	PVC	SDR 35	0.35	19.8	**18.89%**	0.62	0.40
	FUT305	FUT304	0.00	0	0	0	0	2.68	184	4.00	3.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	2.68	0.8	3.7	78.1	200	PVC	SDR 35	0.35	19.8	**18.89%**	0.62	0.40
	FUT304	23	0.00	0	0	0	0	2.68	184	4.00	3.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	2.68	0.8	3.7	81.0	200	PVC	SDR 35	0.35	19.8	**18.89%**	0.62	0.40
R23A	23	22	0.37	0	8	0	20	3.73	252	4.00	4.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.37	3.73	1.0	197.1	79.9	600	CONCRETE	SDR 35	0.15	250.7	**78.62%**	0.86	0.84
Future R303A	FUT303	FUT302	6.66	59	94	0	430	6.66	430	4.00	7.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	6.66	6.66	1.9	8.8	143.6	200	PVC	SDR 35	0.35	19.8	**44.62%**	0.62	0.51
	FUT302	FUT301	0.00	0	0	0	0	6.66	430	4.00	7.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.66	1.9	8.8	17.9	200	PVC	SDR 35	0.35	19.8	**44.62%**	0.62	0.51
	FUT301	FUT300	0.00	0	0	0	0	6.66	430	4.00	7.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.66	1.9	8.8	158.2	200	PVC	SDR 35	0.35	19.8	**44.62%**	0.62	0.51
	FUT300	22	0.00	0	0	0	0	6.66	430	4.00	7.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.66	1.9	8.8	81.0	200	PVC	SDR 35	0.35	19.8	**44.62%**	0.62	0.51
R22A	22	21	1.67	0	50	0	125	12.05	806	3.86	12.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	1.67	12.05	3.4	208.0	234.0	600	CONCRETE	SDR 35	0.15	250.7	**82.95%**	0.86	0.86
																		1.00																	
R31A	31	30	1.27	4	20	0	63	1.27	63	4.00	1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	1.27	1.27	0.4	1.4	176.2	200	PVC	SDR 35	0.35	19.8	**6.98%**	0.62	0.30
Future R318A	FUT318	FUT317	6.79	105	8	0	367	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	6.79	6.79	1.9	7.8	187.0	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
	FUT317	FUT316	0.00	0	0	0	0	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.79	1.9	7.8	17.4	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
	FUT316	FUT315	0.00	0	0	0	0	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.79	1.9	7.8	53.1	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
	FUT315	FUT314	0.00	0	0	0	0	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.79	1.9	7.8	18.5	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
	FUT314	FUT313	0.00	0	0	0	0	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.79	1.9	7.8	27.0	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
	FUT313	FUT312	0.00	0	0	0	0	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.79	1.9	7.8	79.7	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
	FUT312	30	0.00	0	0	0	0	6.79	367	4.00	5.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	6.79	1.9	7.8	94.4	200	PVC	SDR 35	0.35	19.8	**39.63%**	0.62	0.50
R30A	30	29	0.29	1	4	0	13	8.35	443	4.00	7.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.29	8.35	2.3	9.5	49.5	200	PVC	SDR 35	0.35	19.8	**48.10%**	0.62	0.53
R29A	29	21	2.17	0	69	0	173	10.51	616	3.93	9.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	2.17	10.51	2.9	12.7	271.3	200	PVC	SDR 35	0.35	19.8	**64.37%**	0.62	0.57
Future R311A	FUT311	FUT310	8.30	67	108	0	491	8.30	491	3.98	7.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	8.30	8.30	2.3	10.2	81.1	200	PVC	SDR 35	0.35	19.8	**51.76%**	0.62	0.54
	FUT310	FUT309	0.00	0	0	0	0	8.30	491	3.98	7.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	8.30	2.3	10.2	18.1	200	PVC	SDR 35	0.35	19.8	**51.76%**	0.62	0.54
G309A	FUT309	FUT308	0.00	0	0	0	0	8.30	491	3.98	7.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.79	0.0	0.79	9.09	2.5	10.5	262.2	200	PVC	SDR 35	0.35	19.8	**52.87%**	0.62	0.54
	FUT308	FUT307	0.00	0	0	0	0	8.30	491	3.98	7.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.0	0.00	9.09	2.5	10.5	79.0	200	PVC	SDR 35	0.35	19.8	**52.87%**	0.62	0.54
	FUT307	21	0.00	0	0	0	0	8.30	491	3.98	7.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.0	0.00	9.09	2.5	10.5	81.0	200	PVC	SDR 35	0.35	19.8	**52.87%**	0.62	0.54
R21A	21	20	0.12	0	0	0	0	30.98	1913	3.60	27.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.0	0.12	31.77	8.9	228.8	81.0	600	CONCRETE	SDR 35	0.15	250.7	**91.25%**	0.86	0.88
R20A	20	19	0.35	4	2	0	18	31.33	1931	3.60	28.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.0	0.35	32.12	9.0	229.1	77.6	600	CONCRETE	SDR 35	0.15	250.7	**91.39%**	0.86	0.88
R28A	28	19	3.39	50	0	0	165	3.39	165	4.00	2.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.39	3.39	0.9	3.6	311.0	200	PVC	SDR 35	0.50	23.6	**15.32%**	0.74	0.45
R19A, R19B	19	18	4.01	97	0	0	320	38.73	2416	3.52	34.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.0	4.01	39.52	11.1	237.5	204.2	600	CONCRETE	SDR 35	0.15	250.7	**94.73%**	0.86	0.89
R27A, G27B	27	26	1.38	0	42	0	105	1.38	105	4.00	1.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.36	1.36	0.0	2.74	2.74	0.8	2.5	203.3	200	PVC	SDR 35	0.35	19.8	**12.48%**	0.62	0.35
R26A	26	18	0.54	0	17	0	43	1.92	148	4.00	2.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.36	0.0	0.54	3.28	0.9	3.3	134.2	200	PVC	SDR 35	0.35	19.8	**16.72%**	0.62	0.38
	18	17	0.00	0	0	0	0	40.65	2564	3.50	36.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.15	0.0	0.00	42.80	12.0	240.3	16.8	600	CONCRETE	SDR 35	0.15	250.7	**95.85%**	0.86	0.89
R17A	17	16	0.68	0	0	0	0	41.33	2564	3.50	36.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.15	0.0	0.68	43.48	12.2	240.5	212.2	600	CONCRETE	SDR 35	0.15	250.7	**95.93%**	0.86	0.89
R25A	25	16	4.00	0	115	0	288	4.00	288	4.00	4.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	4.00	4.00	1.1	5.8	383.7	200	PVC	SDR 35	0.80	29.9	**19.32%**	0.94	0.60
G16A, I16A, R16B, R16A	16	15	0.81	0	0	0	0	46.14	2851	3.46	40.0	0.00	0.00	0.00	0.00	0.00	0.00	2.44	2.44	0.96	3.11	2.1	4.22	51.69	14.5	248.6	253.5	600	CONCRETE	SDR 35	0.15	250.7	**99.14%**	0.86	0.90
	15	14	0.00	0	0	0	0	46.14	2851	3.46	40.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.44	0.00	3.11	2.1	0.00	51.69	14.5	248.6	85.0	600	CONCRETE	SDR 35	0.15	250.7	**99.14%**	0.86	0.90
	14	13	0.00	0	0	0	0	46.14	2851	3.46	40.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.44	0.00	3.11	2.1	0.00	51.69	14.5	248.6	120.0	600	CONCRETE	SDR 35	0.15	250.7	**99.14%**	0.86	0.90
	13	12	0.00	0	0	0	0	46.14	2851	3.46	40.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.44	0.00	3.11	2.1	0.00	51.69	14.5	248.6	120.0	600	CONCRETE	SDR 35	0.15	250.7	**99.14%**	0.86	0.90
	12	11	0.00	0	0	0	0	46.14	2851	3.46	40.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.44	0.00	3.11	2.1	0.00	51.69	14.5	248.6	120.0	600	CONCRETE	SDR 35	0.15	250.7	**99.14%**	0.86	0.90
	11	10	0.00	0	0	0	0	46.14	2851	3.46	40.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.44	0.00	3.11	2.1	0.00	51.69	14.5	248.6	120.0	600	CONCRETE	SDR 35	0.15	250.7	**99.14%**	0.86	0.90
																												600							
				342	594	0	2607																												

*Notes:

1. Area 6 outlet to be rerouted from Laird Street pump station SAN26 as per Conceptual Site Servicing Study, Report 37533-5.2.2 dated November 2015. 84 L/s to be conveyed from Area 6 to 600mm dia. trunk sanitary sewer.

2. Laird Street pump station to be rerouted to discharge to SAN26. 108.00 L/s to be conveyed from Laird Street pump station to 600mm dia. trunk sanitary sewer.

3. Future low density residential block (R19B) assumed with 28units/ha as per Fernbank MSS

4. Population densities as per 2009 Fernbank Community MSS

B.2 SANITARY DESIGN BACKGROUND REPORT EXCERPTS AND CORRESPONDENCE

From:	Bougadis, John
To:	Balima, Nadege
Subject:	RE: 27970 - PDFs and Design sheet for City
Attachments:	image002.png Pages from (2013.12.19)Area6_MSR.pdf Pages from 07 - Annex A.2 - Wastewater Project Sheets.pdf

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Hi Nadege,

We can speak more on this on Monday. I have attached the two pieces of information which relate to diverting the area 6 PS (ultimate capacity of 84 l/s) and Liard PS (ultimate capacity of 108 l/s) to the future Fernbank trunk via the CRT Phase 1 Lands.

Thanks

John
x14990

From: Balima, Nadege
Sent: 2017/01/13 3:21 PM
To: Bougadis, John
Subject: RE: 27970 - PDFs and Design sheet for City

Thanks for your prompt reply John.
My next question was therefore going to be: should this be identified somewhere/to someone? Would it be useful for IBI include the information from this analysis in their report to show the exercise was done or was this simply an attempt to see if more future growth could be accommodated?
Thanks,

Nadege.

From: Bougadis, John
Sent: Friday, January 13, 2017 1:22 PM
To: Balima, Nadege
Subject: RE: 27970 - PDFs and Design sheet for City

Hi Nadege,

The future flow allowance must consider at least 192 l/s (Liard PS rated capacity =108 l/s plus Area 6/Stittsville South Pump station rated capacity of 84 l/s). The total flow I provided considered future flows beyond the current urban boundary.

I don't have a problem if the future flow allowance is reduced to 192 l/s to alleviate issues IBI is currently having with their design. The only problem I see is that capacity may not be available in this area to accommodate growth beyond the urban boundary. This will have to be assessed at that time.

Thanks

John

x14990

From: Balima, Nadege
Sent: 2017/01/13 12:29 PM
To: Bougadis, John
Subject: FW: 27970 - PDFs and Design sheet for City

Hi John,
Please see below for your information.
I haven't had a chance to discuss this with you and I have to head out for the rest of the day.
I'll contact you next week to go over this issue.
Thanks,

Nadège Balima, P.Eng., M.P.M., LEED Green Assoc.
Project Manager, Infrastructure Approvals
Development Review Services (West)
613.580.2424 ext. 13477

From: Jim Moffatt [mailto:jmoffatt@IBIGroup.com]
Sent: Wednesday, January 11, 2017 10:17 AM
To: Balima, Nadege
Cc: Jim Burghout; Shawn Malhotra; Karlinda Hinds
Subject: FW: 27970 - PDFs and Design sheet for City

As per our conversation yesterday, the recent request from the City to include an additional 110 l/s in the sub trunk sewer in the CRT property will have a detrimental effect on the design of the subdivision. The sub trunk sewer was already at capacity and cannot accept additional flows without increasing its slope. The increased slope will essentially mean that we will have to raise grades over large portions of the site to avoid sewer conflicts. Attached for your reference are copies of the revised spreadsheets and marked drawings which indicate the impact to the sanitary design. The spreadsheets shown the sewer deficiencies with the new flows added and the marked drawings shown the needed sewer slope to accommodate the added flows.This design assumes we maintain the proposed sewer size of 600mm dia which matches the existing trunk sewer size.
As you may recall the MSS document recognized that there could be a significant grade raise within the subdivision and to counter this recommended that the stormwater outlet, the Flewellyn Drain, be lowered to reduce HGL's and the need for significant grade raises and fill requirements. Adding additional flows to the proposed 600mm dia trunk sewer puts us back in the pre MSS situation and generally goes against the intent of the MSS recommendations in this respect. Additionally there are areas within the subdivision that are approaching recommended grade raise limits and those areas cannot accommodate any upward changes to the proposed grades.
All this to say we need to collectively review the need to handle the extra about 300 l/s (Laird Street PS, Area 6 and the 100l/s for future areas) that were never anticipated in the MSS document. However we need to do this in a timely fashion since we are about to get our approvals on the pond and outlet channel and our clients wants to complete the first phase by fall '17.
Let me know your thoughts on this matter.

Jim Moffatt

Associate | Manager, Land Engineering
email jmoffatt@IBIGroup.com web www.ibigroup.com

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tel +1 613 225 1311 fax +1 613 225 9868

From: Karlinda Hinds
Sent: Wednesday, January 11, 2017 9:00 AM
To: Jim Moffatt <jmoffatt@IBIGroup.com>
Subject: 27970 - PDFs and Design sheet for City

Karlinda Hinds

email Karlinda.Hinds@ibigroup.com web www.ibigroup.com

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NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

No.	REVISION	DATE	BY
3	ISSUED WITH MASTER SERVICING STUDY	MAY 25/09	MAB
2	UPDATED WITH DRAFT MASTER SERVICING STUDY	SEP 12/08	MAB
1	ISSUED WITH DRAFT MASTER SERVICING STUDY	MAY 02/08	MAB

LICENSED PROFESSIONAL ENGINEER M.A. BISSETT PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER J.G. RIDDELL PROVINCE OF ONTARIO

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Email: novainfo@novatech-eng.com

DESIGN	KJM
CHECKED	MAB
DRAWN	KJM
CHECKED	MAB
APPROVED	JGR

SCALE 1 : 5000

CITY OF OTTAWA
FERNBANK CDP
SANITARY DRAINAGE AREA PLAN

PROJECT No. 101108-0
DATE AUGUST 2007
DRAWING No. 101108-SAN

PLANBI.DWG - 1000mmX707mm

Table 3.2 Elements Tributary to MH-FT18

DESIGN	AREA (HA)	POPULATION
2012	13.19	538
CRT Phase 1*	12.21	524

** The areas and populations for the MH-FT24 outlet have been adjusted to account for OP Expansion Area 6.*

As is evident from these tables, the areas and population estimates for each outlet are relatively consistent. There are to be some minor differences expected between final design, when final lotting is known, and the more macro focused master study estimates. Therefore, the sanitary design is in general conformance with the 2012 Trunk Sewer Report.

There are some changes now recommended to the sanitary drainage area boundaries, especially along the west side of Robert Grant Avenue and the drainage divide along the Phase 1A limits. The changes are identified in **Figure 3.1**. The significant change is that the school site, Block 361, adjacent to Robert Grant Avenue is now proposed to be serviced from Cope Drive and be tributary to the proposed 600 mm Ø sub-trunk sewer in Goldhawk Drive. The MSS report recommended that the school site be tributary eastward to the Fernbank Crossing development. The change is recommended because of ownership boundaries.

Upstream of MH-FT24 on the Fernbank Trunk Sewer, the 2009 MSS document recommended construction of a 525 mm diameter sub-trunk sewer along Goldhawk Drive and a 450 mm diameter sewer oversized for external lands west of Shea Road. A copy of the 2009 MSS Sanitary Drainage Area Plan (Drawing 101108-SAN) is included in **Appendix D**. Since the 2009 MSS report was completed, the City of Ottawa has requested that the CRT sanitary sewer be oversized to account for wastewater flows to the existing Laird Street Pump Station and also expected flow from the 2012 OPA Area 6 expansion lands. The latter areas were brought into the urban envelope in 2012 as part of the last Official Plan review by the City.

In accordance with recent instructions from the City of Ottawa, an allowance for external flows of 192 l/s has been provided in the proposed 600 mm Ø sub-trunk sewer in the subject property, 108 l/s for the Liard Street Pump Station and 84 l/s for the OPA 76 Area 6 lands. Refer to an e-mail string last dated January 31, 2017 from the City located in **Appendix B**.

Therefore, the recommended sanitary sewer extension through the CRT Phase 1 site to accommodate the revised design criteria is now a 600 mm diameter pipe as opposed to the 450/525 pipe recommended in the MSS report.

As recently agreed with the City, the proposed 600 mm diameter sanitary sub-trunk sewer through the CRT property has been sized to accommodate the following external flows:

• Liard Street Pump Station	108 l/s
• OPA 76 Area 6 Pump Station	84 l/s
	192 l/s

Those flows are in addition to other upstream flows from future developments within the Fernbank CDP area.

IBI Group
400-333 Preston Street
Ottawa, Ontario
K1S 5N4

SANITARY SEWER DESIGN SHEET

PROJECT: CRT DEVELOPMENT
LOCATION: CITY OF OTTAWA
CLIENT: CRT DEVELOPMENT INC.

LOCATION				RESIDENTIAL									ICI AREAS							INFILTRATION ALLOWANCE			TOTAL	PROPOSED SEWER DESIGN						
				UNIT TYPES				AREA	POPULATION		PEAK	PEAK	AREA (Ha)						PEAK	AREA (Ha)		FLOW	FLOW	CAPACITY	LENGTH	DIA	SLOPE	VELOCITY	AVAILABLE	
													INSTITUTIONAL		COMMERCIAL		INDUSTRIAL		FLOW									(full)	CAPACITY	
STREET	AREA ID	FROM MH	TO MH	SF	SD	TH	APT	(Ha)	IND	CUM	FACTOR	FLOW (L/s)	IND	CUM	IND	CUM	IND	CUM	(L/s)	IND	CUM	(L/s)	(L/s)	(L/s)	(m)	(mm)	(%)	(m/s)	L/s	(%)
																							0.00							
	LSPS	Allowance						0.00	0.0	0.0													108.00							
	STITTSVILLE 6 PS		110A					0.00	0.0	0.0				0.00		0.00		0.00	0.00				84.00							
Future Street	INST.3	BLKHD	110A					0.00	0.0	0.0			2.47	2.47		0.00		0.00	2.14											
	PARK4	BLKHD	110A					0.83	0.0	0.0				0.00		0.00		0.00	0.00											
	PARK5	BLKHD	110A					1.04	0.0	0.0				0.00		0.00		0.00	0.00											
	RES.9	BLKHD	110A					34.81	2610.8	2610.8				0.00		0.00		0.00	0.00											
	RES.7	BLKHD	110A					4.24	318.0	318.0				0.00		0.00		0.00	0.00											
	RES.13	BLKHD	110A					2.22	133.2	133.2				0.00		0.00		0.00	0.00											
	RES.12	BLKHD	110A					43.89	2633.4	2633.4				0.00		0.00		0.00	0.00											
	INST.4	BLKHD	110A					0.00	0.0	0.0			2.44	2.44		0.00		0.00	2.12											
	COMM.	BLKHD	110A					0.00	0.0	0.0				0.00	0.63	0.63		0.00	0.55											
	HYD.4	BLKHD	110A					3.06	0.0	0.0				0.00		0.00		0.00	0.00											
	RES.8	BLKHD	110A					2.30	172.5	172.5				0.00		0.00		0.00	0.00											
	HYD.5	BLKHD	110A					5.20	0.0	0.0				0.00		0.00		0.00	0.00											
Future Street	RES.11	BLKHD	110A					6.91	414.6	414.6				0.00		0.00		0.00	0.00											
	PARK6	BLKHD	110A					1.19	0.0	0.0				0.00		0.00		0.00	0.00											
	RES.10	BLKHD	110A					1.92	115.2	115.2				0.00		0.00		0.00	0.00											
	HYD.3	BLKHD	110A					6.31	0.0	0.0				0.00		0.00		0.00	0.00											
TOTAL		**BLKHD**	**110A**					**113.92**		**6397.7**	**3.14**	**81.49**		**4.91**		**0.63**		**0.00**	**4.81**	**119.46**	**119.46**	**33.45**	**311.74**	**320.28**	**24.02**	**600**	**0.25**	**1.097**	**8.54**	**2.67**
GOLDHAWK DRIVE		**110A**	**109A**					**0.00**	**0.0**	**9779.6**	**2.96**	**117.43**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**186.59**	**52.25**	**374.66**	**378.96**	**61.28**	**600**	**0.35**	**1.298**	**4.30**	**1.14**
GOLDHAWK DRIVE	**110A**	**1101A**	**1092A**	**1**				**0.18**	**3.3**	**3.3**	**4.00**	**0.05**								**0.18**	**0.18**	**0.05**	**0.10**	**28.63**	**61.28**	**200**	**0.70**	**0.883**	**28.52**	**99.64**
GOLDHAWK DRIVE		**109A**	**108A**					**0.00**	**0.0**	**9782.9**	**2.96**	**117.47**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**186.77**	**52.30**	**374.74**	**378.96**	**57.50**	**600**	**0.35**	**1.298**	**4.22**	**1.11**
GOLDHAWK DRIVE	**109A**	**1091A**	**1082A**	**5**				**0.32**	**16.5**	**16.5**	**4.00**	**0.27**								**0.32**	**0.32**	**0.09**	**0.36**	**28.63**	**57.50**	**200**	**0.70**	**0.883**	**28.27**	**98.75**
GOLDHAWK DRIVE		**108A**	**107A**					**0.00**	**0.0**	**9799.4**	**2.96**	**117.64**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**187.09**	**52.39**	**375.00**	**378.96**	**53.32**	**600**	**0.35**	**1.298**	**3.96**	**1.05**
GOLDHAWK DRIVE	**108A**	**1081A**	**1072A**	**4**				**0.30**	**13.2**	**13.2**	**4.00**	**0.21**							**0.00**	**0.30**	**0.30**	**0.08**	**0.30**	**28.63**	**53.32**	**200**	**0.70**	**0.883**	**28.33**	**98.96**
GOLDHAWK DRIVE		**107A**	**106A**					**0.00**	**0.0**	**9812.6**	**2.96**	**117.77**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**187.39**	**52.47**	**375.22**	**378.96**	**62.94**	**600**	**0.35**	**1.298**	**3.74**	**0.99**
GOLDHAWK DRIVE	**107A**	**1071A**	**1062A**	**7**				**0.31**	**23.1**	**23.1**	**4.00**	**0.37**		**0.00**		**0.00**		**0.00**	**0.00**	**0.31**	**0.31**	**0.09**	**0.46**	**28.63**	**62.94**	**200**	**0.70**	**0.883**	**28.17**	**98.39**
GOLDHAWK DRIVE		**106A**	**105A**					**0.00**	**0.0**	**9835.7**	**2.96**	**118.01**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**187.70**	**52.56**	**375.54**	**378.96**	**60.09**	**600**	**0.35**	**1.298**	**3.42**	**0.90**
GOLDHAWK DRIVE	**106A**	**1061A**	**1052A**	**2**				**0.24**	**6.6**	**6.6**	**4.00**	**0.11**		**0.00**		**0.00**		**0.00**	**0.00**	**0.24**	**0.24**	**0.07**	**0.17**	**28.63**	**60.09**	**200**	**0.70**	**0.883**	**28.45**	**99.39**
		105A	**104A**					**0.00**	**0.0**	**10558.3**	**2.93**	**125.37**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**200.47**	**56.13**	**386.48**	**389.64**	**72.85**	**600**	**0.37**	**1.335**	**3.16**	**0.81**
GOLDHAWK DRIVE	**105A**	**1051A**	**1042A**	**7**				**0.45**	**23.1**	**23.1**	**4.00**	**0.37**								**0.45**	**0.45**	**0.13**	**0.50**	**27.59**	**72.85**	**200**	**0.65**	**0.851**	**27.09**	**98.19**
GOLDHAWK DRIVE		**104A**	**103A**					**0.00**	**0.0**	**10581.4**	**2.93**	**125.60**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**200.92**	**56.26**	**386.84**	**389.64**	**48.77**	**600**	**0.37**	**1.335**	**2.80**	**0.72**
GOLDHAWK DRIVE	**104A**	**1041A**	**1032A**	**9**				**0.47**	**29.7**	**29.7**	**4.00**	**0.48**							**0.00**	**0.47**	**0.47**	**0.13**	**0.61**	**27.59**	**48.77**	**200**	**0.65**	**0.851**	**26.97**	**97.78**
GOLDHAWK DRIVE		**103A**	**102A**					**0.00**	**0.0**	**10611.1**	**2.93**	**125.90**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**201.39**	**56.39**	**387.27**	**389.64**	**45.00**	**600**	**0.37**	**1.335**	**2.37**	**0.61**
GOLDHAWK DRIVE	**103A, HYD1**	**1031A**	**1021A**	**6**				**2.01**	**19.8**	**19.8**	**4.00**	**0.32**							**0.00**	**2.01**	**2.01**	**0.56**	**0.88**	**27.59**	**45.00**	**200**	**0.65**	**0.851**	**26.70**	**96.80**
GOLDHAWK DRIVE	**102A**	**102A**	***FT-24 (EX)***					**0.12**	**0.0**	**10630.9**	**2.93**	**126.10**		**14.32**		**0.63**		**0.00**	**12.98**	**0.12**	**203.52**	**56.99**	**388.07**	**389.64**	**102.59**	**600**	**0.37**	**1.335**	**1.57**	**0.40**
HYDRO EASEMENT		***FT-24 (EX)***	***FT-23 (EX)***					**0.00**	**0.0**	**10650.7**	**2.93**	**126.30**		**14.32**		**0.63**		**0.00**	**12.98**	**0.00**	**205.53**	**57.55**	**388.83**	**400.03**	**107.50**	**600**	**0.39**	**1.371**	**11.20**	**2.80**

Design Parameters:

Residential			ICI Areas			Peak Factor
SF	3.3	p/p/u	INST	50,000	L/Ha/day	1.5
TH/SD	2.5	p/p/u	COM	50,000	L/Ha/day	1.5
APT	1.8	p/p/u	IND	35,000	L/Ha/day	MOE Chart
Low	60	p/p/Ha				
Med	75	p/p/Ha				
High	90	p/p/Ha				

Notes:
1. Mannings coefficient (n) = 0.013
2. Demand (per capita): 350 L/day
3. Infiltration allowance: 0.28 L/s/Ha
4. Residential Peaking Factor:
   Harmon Formula = 1+(14/(4+P^0.5))
   where P = population in thousands

Designed: J.I.M.

Checked: P.K.

Dwg. Reference: 27970 - 501, 501A, 501B

No.	Revision	Date
1.	Submission No. 1 to City of Ottawa	2013-08-29
2.	Submission No. 2 to City of Ottawa	2014-01-22
3.	Submission No. 3 to City of Ottawa	2014-08-22
4.	Submission No. 4 to City of Ottawa	2015-06-15
5.	Submission No. 5 to City of Ottawa	2016-11-10
6.	Submission for MOE Approval	2017-02-10
7.	Resubmission for MOE Approval	2017-07-14

File Reference:	Date:	Sheet No:
27970.5.7.1	2017-07-14	4 of 4

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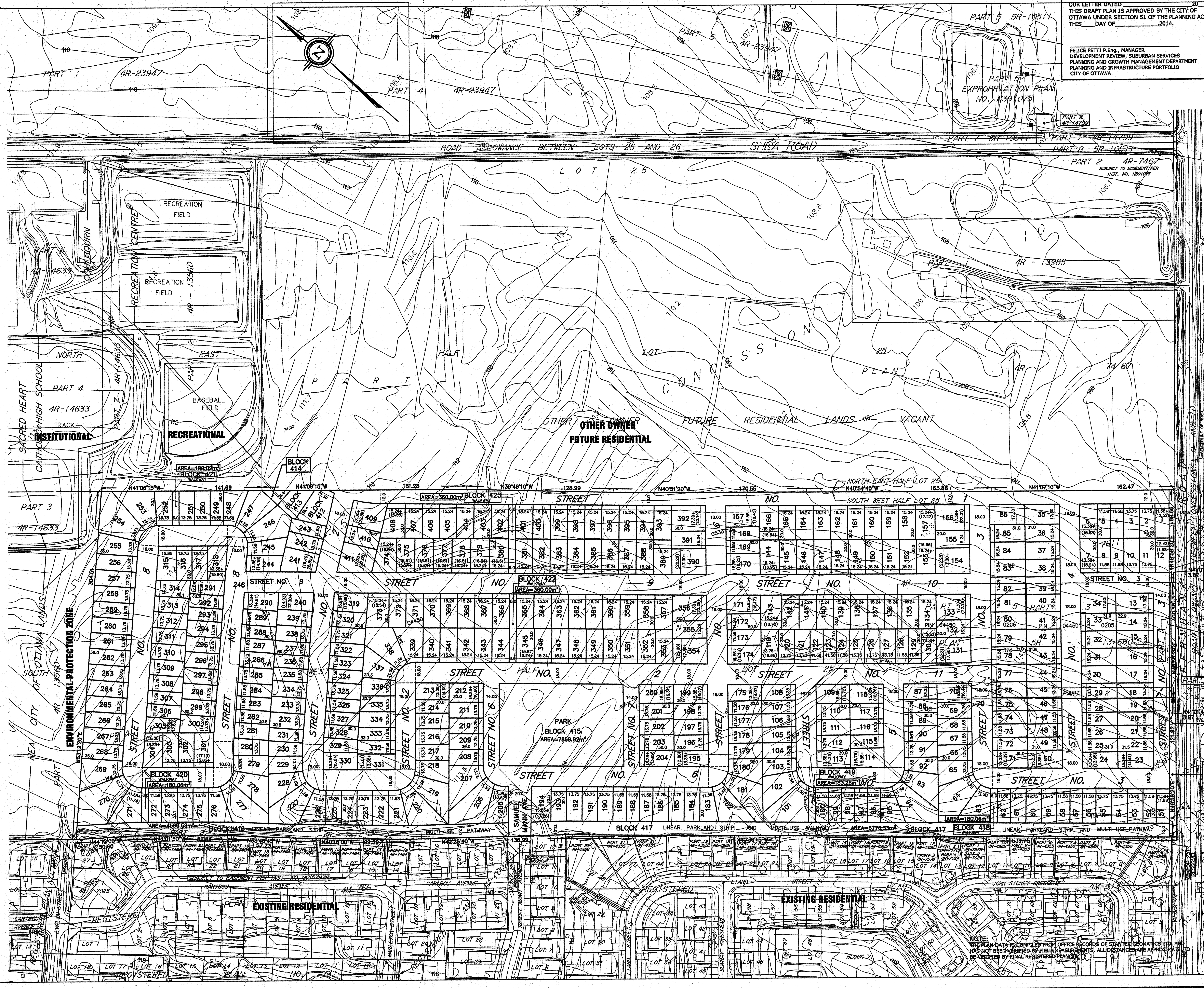
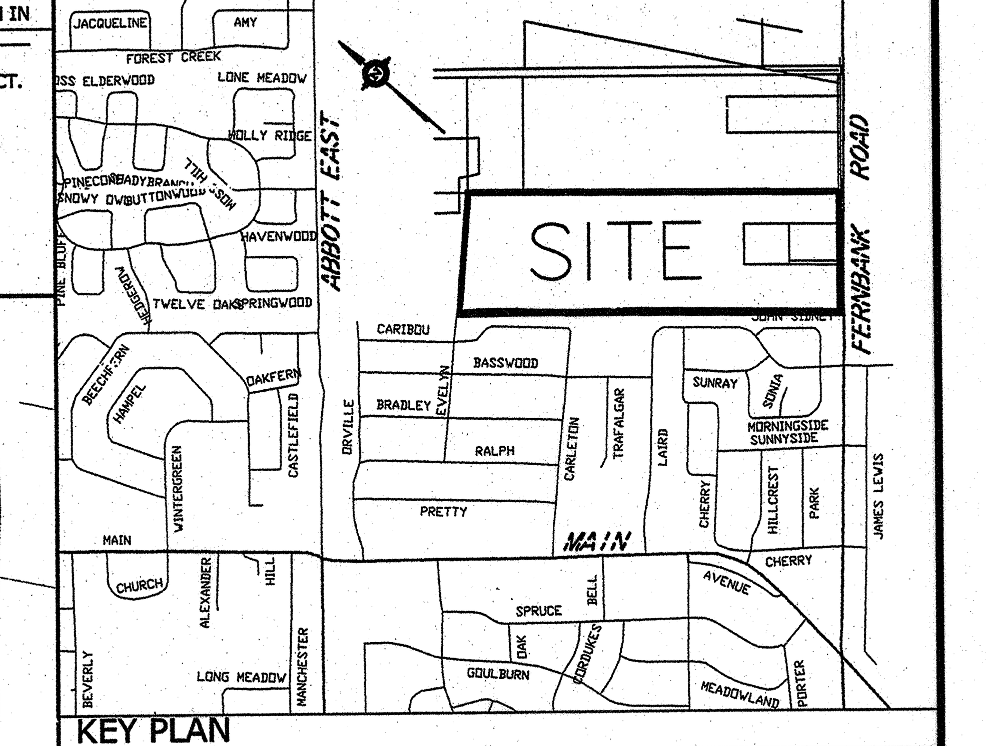


SUBJECT TO THE CONDITIONS, IF ANY, SET FORTH IN OUR LETTER DATED ____________ 20____
THIS DRAFT PLAN IS APPROVED BY THE CITY OF OTTAWA UNDER SECTION 51 OF THE PLANNING ACT.
THIS____DAY OF____________,2014.

FELICE PETTI P.Eng., MANAGER
DEVELOPMENT REVIEW, SUBURBAN SERVICES
PLANNING AND GROWTH MANAGEMENT DEPARTMENT
PLANNING AND INFRASTRUCTURE PORTFOLIO
CITY OF OTTAWA



KEY PLAN

DRAFT PLAN OF SUBDIVISION of

PART OF THE SOUTH HALF OF LOT 25 CONCESSION 10
GEOGRAPHIC TOWNSHIP OF GOULBOURN
CITY OF OTTAWA

Scale 1 : 1500

60 45 30 15 0 30 60 Metres

METRIC CONVERSION

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

SURVEYOR'S CERTIFICATE

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE SUBJECT LANDS AND THEIR RELATIONSHIP TO ADJOINING LANDS HAVE BEEN ACCURATELY SHOWN.

08/05/14
DATE

BRIAN J. WEBSTER
ONTARIO LAND SURVEYOR

OWNER'S CERTIFICATE

I HEREBY AUTHORIZE THIS DRAFT PLAN OF SUBDIVISION TO BE SUBMITTED ON MY BEHALF.

DATED : 5 Aug. 2014

DOUG LAZIER
2087875 ONTARIO LTD.
PRESIDENT
I HAVE THE AUTHORITY
TO BIND THE CORPORATION

SCHEDULE OF LAND USE

LOT/BLOCK	USE	UNITS	AREA (Ha/ac)
1 TO 12, 19 TO 28, 46 TO 75, 87 TO 130, 167 TO 338	SINGLES	264	12.19/30.13
13 TO 18, 29 TO 45, 76 TO 86, 131 TO 166, 339 TO 414	SEMI-DETACHED	282	6.93/17.12
415 TO 417	PARK AND OPEN SPACE		1.92/4.75
418 TO 423	WALKWAY		0.14/0.36
424 TO 426	STREET WIDENING		0.14/0.36
STREETS			7.58/18.73
TOTAL		546	28.90/71.45

INFORMATION: REQUIRED UNDER SECTION 51 (17) OF THE PLANNING ACT R.S.O. 1990

a. SEE PLAN
b. SEE PLAN
c. SEE PLAN
d. SEE PROPOSED LAND USE SCHEDULE (ABOVE)
e. SEE PLAN
f. SEE PLAN
g. SEE PLAN
h. CITY WATER AVAILABLE \ WELL
i. SEE SOIL REPORT
j. SEE TOPOGRAPHICAL INFORMATION
k. ALL CITY SERVICES AVAILABLE
l. NO EASEMENTS REGISTERED ON TITLE

Stantec Geomatics Ltd.
Ontario Land Surveyors
Canada Lands Surveyors
Stantec
1331 CLYDE AVENUE, SUITE 400, OTTAWA, ON. K2C 3G4
PHONE (613)722-4420 FAX (613)722-2799
brian.webster@stantec.com
stantec.com

DRAWN BY:CEC | CHECKED BY: | PM: FP | FIELD: N/A | PROJECT No.: 161612417-131

Appendix C STORMWATER MANAGEMENT CALCULATIONS

C.1 CONCEPTUAL STORM SEWER DESIGN SHEET

Stantec

JOB NAME	
DATE:	2017-12-07
REVISION:	1
DESIGNED BY:	WAJ
CHECKED BY:	AMP

STORM SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160400900

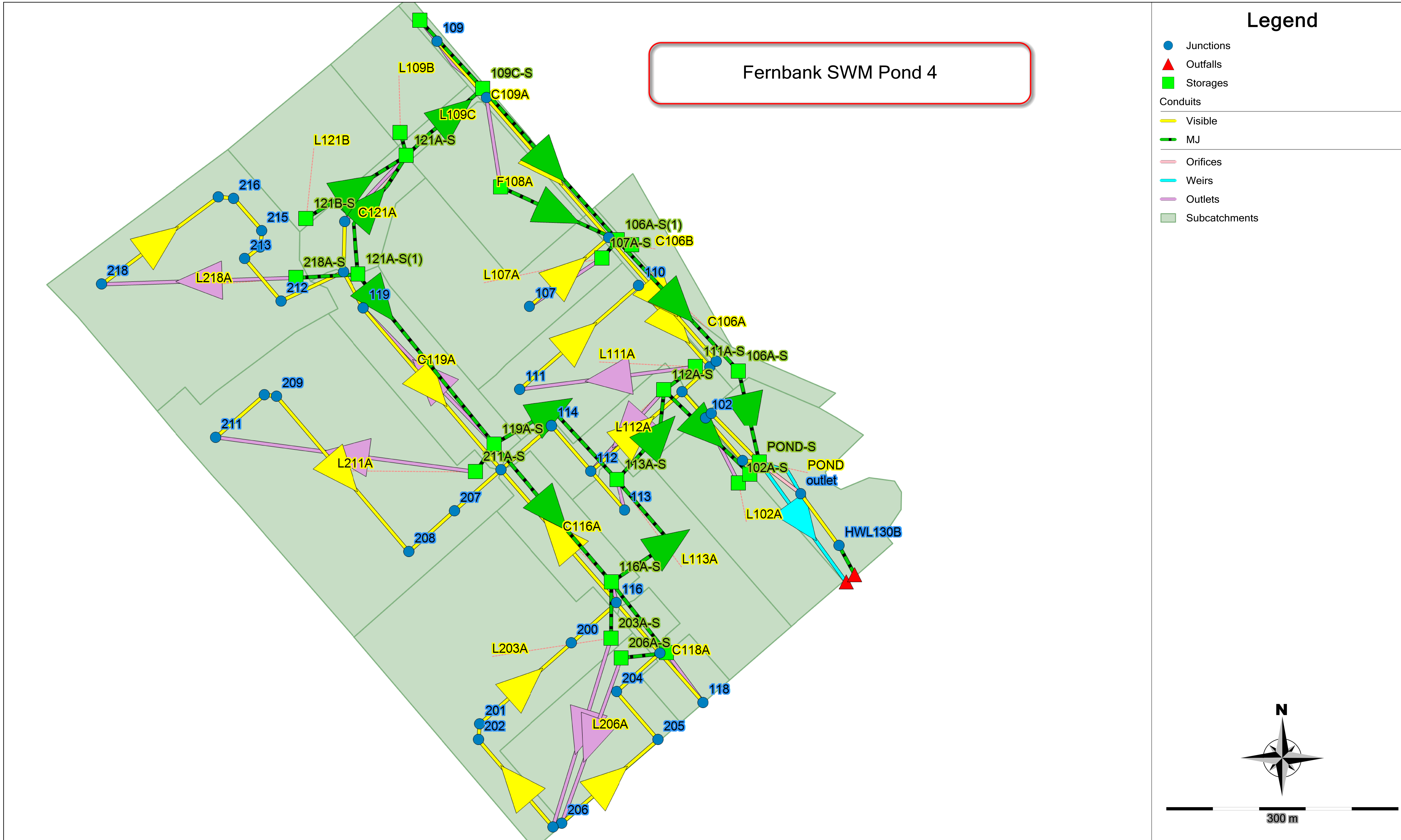
DESIGN PARAMETERS

I = a / (t+b)^c (As per City of Ottawa Guidelines, 2012)

	1:2 yr	1:5 yr	1:10 yr	1:100 yr				
a =	732.951	998.071	1174.184	1735.688	MANNING'S n =	0.013	BEDDING CLASS =	B
b =	6.199	6.053	6.014	6.014	MINIMUM COVER:	2.00 m		
c =	0.810	0.814	0.816	0.820	TIME OF ENTRY	10 min		

LOCATION			DRAINAGE AREA																									PIPE SELECTION											
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (5-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (100-YEAR) (ha)	AREA (ROOF) (ha)	C (2-YEAR) (-)	C (5-YEAR) (-)	C (10-YEAR) (-)	C (100-YEAR) (-)	A x C (2-YEAR) (ha)	ACCUM. AxC (2YR) (ha)	A x C (5-YEAR) (ha)	ACCUM. AxC (5YR) (ha)	A x C (10-YEAR) (ha)	ACCUM. AxC (10YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)	T of C (min)	I_{2-YEAR} (mm/h)	I_{5-YEAR} (mm/h)	$I_{10-YEAR}$ (mm/h)	$I_{100-YEAR}$ (mm/h)	$Q_{CONTROL}$ (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)	LENGTH (m)	PIPE WIDTH OR DIAMETER (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE %	Q_{CAP} (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	VEL. (ACT) (m/s)	TIME OF FLOW (min)
L113A	113	112	3.34	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.00	2.035	2.035	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	434.1	66.4	675	675	CIRCULAR	CONCRETE	-	0.35	518.8	83.67%	1.40	1.40	0.79
																				10.79																			
L121B, C121A	121	120	2.39	1.29	0.00	0.00	0.00	0.20	0.61	0.00	0.00	0.477	0.477	0.789	0.789	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	330.3	65.2	525	525	CIRCULAR	CONCRETE	-	0.80	401.3	82.30%	1.80	1.78	0.61
																				10.61																			
L218A	218	217	6.70	0.00	0.00	0.00	0.00	0.59	0.00	0.00	0.00	3.954	3.954	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	843.7	188.6	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	76.46%	1.23	1.20	2.61
	217	216	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	3.954	0.000	0.000	0.000	0.000	0.000	0.000	12.61	68.04	92.15	107.96	157.73	0.0	0.0	747.4	19.8	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	67.74%	1.23	1.16	0.29
	216	215	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	3.954	0.000	0.000	0.000	0.000	0.000	0.000	12.90	67.21	91.02	106.63	155.78	0.0	0.0	738.3	55.6	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	66.92%	1.23	1.16	0.80
	215	214	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	3.954	0.000	0.000	0.000	0.000	0.000	0.000	13.70	65.01	88.00	103.08	150.57	0.0	0.0	714.1	21.2	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	64.73%	1.23	1.14	0.31
	214	213	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	3.954	0.000	0.000	0.000	0.000	0.000	0.000	14.01	64.21	86.90	101.78	148.66	0.0	0.0	705.3	25.1	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	63.92%	1.23	1.14	0.37
	213	212	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	3.954	0.000	0.000	0.000	0.000	0.000	0.000	14.38	63.28	85.62	100.28	146.46	0.0	0.0	695.0	72.8	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	62.99%	1.23	1.13	1.07
	212	120	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	3.954	0.000	0.000	0.000	0.000	0.000	0.000	15.45	60.72	82.12	96.17	140.42	0.0	0.0	667.0	89.3	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	60.45%	1.23	1.12	1.33
																				16.78																			
	120	119	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.432	0.000	0.789	0.000	0.000	0.000	0.000	16.78	57.86	78.22	91.57	133.68	0.0	0.0	883.8	53.8	1050	1050	CIRCULAR	CONCRETE	-	0.30	1560.3	56.64%	1.75	1.55	0.58
C119A	119	115	0.00	2.52	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.000	4.432	1.540	2.329	0.000	0.000	0.000	0.000	17.36	56.71	76.64	89.72	130.96	0.0	0.0	1193.9	274.5	1200	1200	CIRCULAR	CONCRETE	-	0.15	1575.3	75.79%	1.35	1.31	3.50
																				20.86																			
C118A	118	117	0.00	1.04	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.000	0.000	0.635	0.635	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	183.6	85.1	600	600	CIRCULAR	PVC	-	0.15	248.1	74.02%	0.85	0.82	1.73
																				11.73																			
L206A	206	205	2.68	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.00	1.636	1.636	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	348.9	164.8	750	750	CIRCULAR	CONCRETE	-	0.15	449.8	77.57%	0.99	0.96	2.86
	205	204	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	1.636	0.000	0.000	0.000	0.000	0.000	0.000	12.86	67.33	91.18	106.81	156.05	0.0	0.0	305.9	82.2	750	750	CIRCULAR	CONCRETE	-	0.15	449.8	68.00%	0.99	0.92	1.48
	204	117	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	1.636	0.000	0.000	0.000	0.000	0.000	0.000	14.34	63.36	85.74	100.42	146.66	0.0	0.0	287.9	75.0	750	750	CIRCULAR	CONCRETE	-	0.15	449.8	64.00%	0.99	0.91	1.37
																				15.72																			
	117	116	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	1.636	0.000	0.635	0.000	0.000	0.000	0.000	15.72	60.13	81.31	95.21	139.02	0.0	0.0	416.5	85.9	825	825	CIRCULAR	CONCRETE	-	0.15	580.0	71.81%	1.05	1.00	1.43
																				17.14																			
L203A	203	202	6.66	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.00	4.061	4.061	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	866.4	148.5	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	78.53%	1.23	1.21	2.05
	202	201	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.061	0.000	0.000	0.000	0.000	0.000	0.000	12.05	69.75	94.50	110.73	161.79	0.0	0.0	786.8	20.3	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.4	71.31%	1.23	1.17	0.29
	201	200	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.061	0.000	0.000	0.000	0.000	0.000	0.000	12.34	68.87	93.29	109.30	159.70	0.0	0.0	776.9	158.6	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	70.41%	1.23	1.17	2.26
	200	116	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.061	0.000	0.000	0.000	0.000	0.000	0.000	14.59	62.75	84.90	99.43	145.21	0.0	0.0	707.8	77.8	1050	1050	CIRCULAR	CONCRETE	-	0.15	1103.3	64.16%	1.23	1.14	1.14
																				15.73																			
C116A	116	115	0.00	1.66	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.000	5.697	1.014	1.649	0.000	0.000	0.000	0.000	17.14	57.13	77.22	90.40	131.96	0.0	0.0	1257.8	227.9	1350	1350	CIRCULAR	CONCRETE	-	0.10	1760.8	71.43%	1.19	1.13	3.36
																				20.50																			
L211A	211	210	9.08	0.00	0.00	0.00	0.00	0.55	0.00	0.00	0.00	4.997	4.997	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	1066.0	83.0	1200	1200	CIRCULAR	CONCRETE	-	0.10	1286.2	82.88%	1.10	1.10	1.26
	210	209	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.997	0.000	0.000	0.000	0.000	0.000	0.000	11.26	72.28	97.97	114.82	167.80	0.0	0.0	1003.2	15.6	1200	1200	CIRCULAR	CONCRETE	-	0.10	1286.2	78.00%	1.10	1.08	0.24
	209	208	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.997	0.000	0.000	0.000	0.000	0.000	0.000	11.50	71.49	96.88	113.53	165.91	0.0	0.0	992.2	264.0	1200	1200	CIRCULAR	CONCRETE	-	0.10	1286.2	77.14%	1.10	1.07	4.10
	208	207	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.997	0.000	0.000	0.000	0.000	0.000	0.000	15.60	60.38	81.66	95.62	139.62	0.0	0.0	838.1	79.1	1200	1200	CIRCULAR	CONCRETE	-	0.10	1286.2	65.16%	1.10	1.02	1.29
	207	115	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.997	0.000	0.000	0.000	0.000	0.000	0.000	16.89	57.64	77.91	91.21	133.15	0.0	0.0	800.0	79.9	1200	1200	CIRCULAR	CONCRETE	-	0.10	1286.2	62.20%	1.10	1.01	1.32
																				18.21																			
	115	114	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	15.125	0.000	3.978	0.000	0.000	0.000	0.000	20.86	50.68	68.41	80.05	116.79	0.0	0.0	2885.4	87.3	1800	1800	CIRCULAR	CONCRETE	-	0.10	3792.1	76.09%	1.44	1.40	1.04
	114	112	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	15.125	0.000	3.978	0.000	0.000	0.000	0.000	21.90	49.16	66.33	77.61	113.20	0.0	0.0	2798.2	77.6	1800	1800	CIRCULAR	CONCRETE	-	0.10	3792.1	73.79%	1.44	1.39	0.93
																				22.83																			
L112A	112	103	1.73	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.00	1.054	18.213	0.000	3.978	0.000	0.000	0.000	0.000	22.83	47.88	64.59	75.56	110.20	0.0	0.0	3135.8	156.7	1800	1800	CIRCULAR	CONCRETE	-	0.10	3792.1	82.69%	1.44	1.44	1.82
																				24.65																			
L111A	111	110	3.52	0.00	0.00	0.00	0.00	0.55	0.00	0.00	0.00	1.937	1.937	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	413.2	203.9	675	675	CIRCULAR	CONCRETE	-	0.30	480.3	86.03%	1.30	1.31	2.60
	110	104	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	1.937	0.000	0.000	0.000	0.000	0.000	0.000	12.60	68.09	92.22	108.05	157.86	0.0	0.0	366.3	140.2	675	675	CIRCULAR	CONCRETE	-	0.30	480.3	76.26%	1.30	1.27	1.85
																				14.44																			
L109B, L109C, C109A	109	108	1.90	0.58	0.00	0.00	0.00	0.27	0.65	0.00	0.00	0.509	0.509	0.379	0.379	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	218.3	97.6	600	600	CIRCULAR	CONCRETE	-	0.15	248.1	87.97%	0.85	0.86	1.89
F108A	108	106	0.00	0.00	0.00	2.44	0.00	0.00	0.00	0.00	0.70	0.000	0.509	0.000	0.379	0.000	0.000	1.711	1.711	11.89	70.23	95.16	111.51	162.94	0.0	0.0	974.0	240.0	1200	1200	CIRCULAR	CONCRETE	-	0.10	1286.2	75.73%	1.10	1.07	3.75
																				15.64																			
L107A	107	106	4.00	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.00	2.437	2.437	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	520.0	135.6	900	900	CIRCULAR	CONCRETE	-	0.10	597.2	87.07%	0.91	0.92	2.46
																				12.46																			
C106A, C106B	106	105	0.00	1.97	0.00	0.00	0.00	0.00	0.43	0.00	0.00	0.000	2.946	0.848	1.227	0.000	0.000	0.000	1.711	15.64	60.29	81.54	95.47	139.41	0.0	0.0	1434.0	212.2	1350	1350	CIRCULAR	CONCRETE	-	0.10	1760.8	81.44%	1.19	1.18	3.00
	105	104	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	2.946	0.000	1.227	0.000	0.000	0.000	1.711	18.64	54.33	73.39	85.90	125.36	0.0	0.0	1290.7	11.1	1350	1350	CIRCULAR	CONCRETE	-	0.10	1760.8	73.30%	1.19	1.15	0.16
																				18.80																			
	104	103	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	4.883	0.000	1.227	0.000	0.000	0.000	1.711	18.80	54.05	73.00	85.44	124.69	0.0	0.0	1574.6	47.5	1350	1350	CIRCULAR	CONCRETE	-	0.10	1760.9	89.42%	1.19	1.21	0.65
																				19.45																			
	102	101	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	0.0	9.9	1500	1500	CIRCULAR	CONCRETE	-	0.26	3760.3	0.00%	2.06	0.00	0.00
	101	Forebay	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	0.0	21.8	1500	1500	CIRCULAR	CONCRETE	-	0.26	3760.3	0.00%	2.06	0.00	0.00
																				10.00									1500										
	103	102	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	23.096	0.000	5.205	0.000	0.000	0.000	1.711	24.65	45.58	61.46	71.88	104.81	0.0	0.0	4311.0	46.0	2100	2100	CIRCULAR	CONCRETE	-	0.12	6266.1	68.80%	1.75	1.65	0.46
L102A	102	100B	2.52	0.00	0.00	0.00	0.00	0.61	0.00	0.00	0.00	1.537	24.633	0.000	5.205	0.000	0.000	0.000	1.711	25.12	45.03	60.71	71.01	103.53	0.0	0.0	4451.3	72.8	2400	1200	RECTANGULAR	CONCRETE	-	0.30	6587.4	67.57%	2.29	2.14	0.57
	100B	Maincell	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	24.633	0.000	5.205	0.000	0.000	0.000	1.711	25.68	44.38	59.83	69.97	102.01	0.0	0.0	4386.9	22.0	2400	1200	RECTANGULAR	CONCRETE	-	0.40	7606.5	57.67%	2.64	2.36	0.16
																				25.84									2400	1200									

C.2 PCSWMM LAYOUT


Fernbank SWM Pond 4
Legend
Junctions
Outfalls
Storages
Conduits
Visible
MJ
Orifices
Weirs
Outlets
Subcatchments
N
300 m
109
109C-S
C109A
L109B
L109C
121A-S
L121B
F108A
216
215
121B-S
C121A
213
218A-S
121A-S(1)
218
L218A
212
119
106A-S(1)
107A-S
C106B
L107A
110
107
C106A
L111A
111A-S
106A-S
111
112A-S
C119A
209
211
114
102
L112A
119A-S
POND-S
L211A
211A-S
112
113A-S
102A-S
POND
outlet
207
113
L102A
208
C116A
HWL130B
L113A
116A-S
116
203A-S
200
206A-S
C118A
L203A
204
118
201
202
L206A
205
206

C.3 POST DEVELOPMENT PCSWMM MODEL INPUT EXAMPLE

```
[TITLE]


[OPTIONS]
;;Options            Value
;;------------------ ------------
FLOW_UNITS           LPS
INFILTRATION         HORTON
FLOW_ROUTING         DYNWAVE
START_DATE           11/17/2017
START_TIME           00:00:00
REPORT_START_DATE    11/17/2017
REPORT_START_TIME    00:00:00
END_DATE             11/19/2017
END_TIME             00:00:00
SWEEP_START          01/01
SWEEP_END            12/31
DRY_DAYS             0
REPORT_STEP          00:01:00
WET_STEP             00:05:00
DRY_STEP             00:05:00
ROUTING_STEP         5
ALLOW_PONDING        NO
INERTIAL_DAMPING     PARTIAL
VARIABLE_STEP        0.75
LENGTHENING_STEP     0
MIN_SURFAREA         0
NORMAL_FLOW_LIMITED  BOTH
SKIP_STEADY_STATE    NO
FORCE_MAIN_EQUATION  H-W
LINK_OFFSETS         ELEVATION
MIN_SLOPE            0
MAX_TRIALS           8
HEAD_TOLERANCE       0.0015
SYS_FLOW_TOL         5
LAT_FLOW_TOL         5
MINIMUM_STEP         0.5
THREADS              4

[EVAPORATION]
;;Type          Parameters
;;------------- ----------
CONSTANT     0.0
DRY_ONLY     NO

[RAINGAGES]
;;               Rain      Time   Snow   Data
;;Name           Type      Intrvl Catch  Source
;;-------------- --------- ------ ------ ----------
RG1              INTENSITY 0:10   1.0    TIMESERIES 100yr_3hr_Chicago_Ottawa

[SUBCATCHMENTS]
;;                                                Total    Pcnt.             Pcnt.
  Curb     Snow
;;Name           Raingage         Outlet           Area     Imperv   Width    Slope
  Length   Pack
;;-------------- ---------------- ---------------- -------- -------- --------
-------- -------- --------
C106A            RG1              106A-S(1)        1.009065 64.286   292      1
  0
C106B            RG1              106B-S           0.960483 0        241      1
  0
C109A            RG1              109A-S           0.58286  64.286   362      1
```

```
  0
C116A           RG1             116A-S          1.663001 58.571   567      1
  0
C118A           RG1             118A-S          1.040192 58.571   244      1
  0
C119A           RG1             119A-S          2.523952 58.571   656      1
  0
C121A           RG1             121A-S          1.293885 58.571   315      1
  0
F108A           RG1             108A-S          2.444837 71.429   200      2
  0
L102A           RG1             102A-S          2.515228 58.571   556      1
  0
L107A           RG1             107A-S          3.995616 58.571   1864     1
  0
L109B           RG1             109B-S          1.609925 0        116      2
  0
L109C           RG1             109C-S          0.28776  64.286   118      0.5
  0
L111A           RG1             111A-S          3.521316 50       536      1
  0
L112A           RG1             112A-S          1.727354 58.571   353      1
  0
L113A           RG1             113A-S          3.335419 58.571   816      1
  0
L121B           RG1             121B-S          2.3862   0        167      2
  0
L203A           RG1             203A-S          6.657405 58.571   1974     1
  0
L206A           RG1             206A-S          2.681205 58.571   939      1
  0
L211A           RG1             211A-S          9.084765 50       2655     1
  0
L218A           RG1             218A-S          6.702376 55.714   2358     1
  0
POND            RG1             POND-S          3.128864 40       280      2
  0

[SUBAREAS]
;;Subcatchment  N-Imperv   N-Perv     S-Imperv   S-Perv     PctZero    RouteTo
PctRouted
;;-------------- ---------- ---------- ---------- ---------- ---------- ----------
----------
C106A           0.013      0.25       1.57       4.67       0          OUTLET
C106B           0.013      0.25       1.57       4.67       0          PERVIOUS
100
C109A           0.013      0.25       1.57       4.67       0          OUTLET
C116A           0.013      0.25       1.57       4.67       0          OUTLET
C118A           0.013      0.25       1.57       4.67       0          OUTLET
C119A           0.013      0.25       1.57       4.67       0          OUTLET
C121A           0.013      0.25       1.57       4.67       0          OUTLET
F108A           0.013      0.25       1.57       4.67       0          OUTLET
L102A           0.013      0.25       1.57       4.67       0          OUTLET
L107A           0.013      0.25       1.57       4.67       0          OUTLET
L109B           0.013      0.25       1.57       4.67       0          PERVIOUS
100
L109C           0.013      0.25       1.57       4.67       0          OUTLET
L111A           0.013      0.25       1.57       4.67       0          OUTLET
L112A           0.013      0.25       1.57       4.67       0          OUTLET
L113A           0.013      0.25       1.57       4.67       0          OUTLET
L121B           0.013      0.25       1.57       4.67       0          PERVIOUS
100
L203A           0.013      0.25       1.57       4.67       0          OUTLET
```

```
L206A            0.013      0.25       1.57       4.67       0          OUTLET
L211A            0.013      0.25       1.57       4.67       0          OUTLET
L218A            0.013      0.25       1.57       4.67       0          OUTLET
POND             0.013      0.25       1.57       4.67       0          OUTLET

[INFILTRATION]
;;Subcatchment   MaxRate    MinRate    Decay      DryTime    MaxInfil
;;-------------- ---------- ---------- ---------- ---------- ----------
C106A            76.2       13.2       4.14       7          0
C106B            76.2       13.2       4.14       7          0
C109A            76.2       13.2       4.14       7          0
C116A            76.2       13.2       4.14       7          0
C118A            76.2       13.2       4.14       7          0
C119A            76.2       13.2       4.14       7          0
C121A            76.2       13.2       4.14       7          0
F108A            76.2       13.2       4.14       7          0
L102A            76.2       13.2       4.14       7          0
L107A            76.2       13.2       4.14       7          0
L109B            76.2       13.2       4.14       7          0
L109C            76.2       13.2       4.14       7          0
L111A            76.2       13.2       4.14       7          0
L112A            76.2       13.2       4.14       7          0
L113A            76.2       13.2       4.14       7          0
L121B            76.2       13.2       4.14       7          0
L203A            76.2       13.2       4.14       7          0
L206A            76.2       13.2       4.14       7          0
L211A            76.2       13.2       4.14       7          0
L218A            76.2       13.2       4.14       7          0
POND             76.2       13.2       4.14       7          0

[JUNCTIONS]
;;               Invert     Max.       Init.      Surcharge  Ponded
;;Name           Elev.      Depth      Depth      Depth      Area
;;-------------- ---------- ---------- ---------- ---------- ----------
100B             105.5      2.65       0          0          0
101              105.477    3.003      0          0          0
102              105.519    3.291      0          0          0
;3000mm
103              105.634    4.792      0          0          0
;2400mm
104              106.282    4.089      0          0          0
;3000mm
105              106.296    3.848      0          0          0
;2400mm
106              106.568    3.592      0          0          0
;1800mm
107              107.154    4.796      0          0          0
;2400mm
108              106.958    4.152      0          0          0
;1200mm
109              107.704    3.296      0          0          0
;1800mm
110              107.377    3.775      0          0          0
;1500mm
111              108.049    3.192      0          0          0
;3000mm
112              105.941    4.867      0          0          0
;1500mm
113              107.298    3.65       0          0          0
114              106.079    4.943      0          0          0
;2400mm
115              106.226    5.79       0          0          0
;2400mm
```

```
116                 106.904    4.58       0          0          0
;1800mm
117                 107.558    4.44       0          0          0
;1200mm
118                 107.91     2.906      0          0          0
;2400mm
119                 107.238    5.3        0          0          0
;2400mm
120                 107.549    5.454      0          0          0
;1200mm
121                 108.596    3.994      0          0          0
;2400mm
200                 107.32     6.004      0          0          0
;2400mm
201                 107.563    8.143      0          0          0
;2400mm
202                 107.623    8.241      0          0          0
;2400mm
203                 107.876    8.91       0          0          0
;1800mm
204                 107.745    5.68       0          0          0
;1800mm
205                 107.928    8.611      0          0          0
;1500mm
206                 108.236    8.264      0          0          0
;2400mm
207                 106.906    6.02       0          0          0
;3000mm
208                 106.988    6.712      0          0          0
;2400mm
209                 107.312    6.855      0          0          0
;2400mm
210                 107.358    6.763      0          0          0
;2400mm
211                 107.471    6.643      0          0          0
;2400mm
212                 107.743    5.566      0          0          0
;3000mm
213                 107.912    6.028      0          0          0
;2400mm
214                 108.01     5.742      0          0          0
;2400mm
215                 108.072    5.622      0          0          0
;2400mm
216                 108.185    4.977      0          0          0
;2400mm
217                 108.245    5.905      0          0          0
;2400mm
218                 108.557    5.643      0          0          0
HWL130B             105.7      2.3        0          0          0
outlet              105.55     2.2        0          0          0

[OUTFALLS]
;;                  Invert     Outfall      Stage/Table      Tide
;;Name              Elev.      Type         Time Series      Gate Route To
;;-------------- ---------- ------------ ---------------- ---- ----------------
OF1                 107.65     FREE                          NO
OF3                 105.58     FREE                          NO

[STORAGE]
;;                  Invert   Max.     Init.    Storage    Curve
Ponded    Evap.
;;Name              Elev.    Depth    Depth    Curve      Params
```

```
Area     Frac.    Infiltration parameters
;;-------------- -------- -------- -------- ---------- -------- -------- --------
-------- -------- -----------------------
102A-S           108.12   2.65     0        TABULAR    102A-S                     0
      0
102A-S(1)        107.5    0.7      0        FUNCTIONAL 0        0        0        0
      0
106A-S           108.27   0.5      0        FUNCTIONAL 0        0        0        0
      0
106A-S(1)        108.56   2.15     0        FUNCTIONAL 0        0        0        0
      0
106B-S           109.1    2.8      0        FUNCTIONAL 0        0        0        0
      0
107A-S           108.24   2.5      0        TABULAR    107A-S                     0
      0
108A-S           108.85   2.5      0        TABULAR    108A-S                     0
      0
109A-S           110.2    2.15     0        FUNCTIONAL 0        0        0        0
      0
109B-S           112      0.6      0        FUNCTIONAL 0        0        0        0
      0
109C-S           109.35   2.5      0        TABULAR    109C-S                     0
      0
111A-S           108.44   2.5      0        TABULAR    111A-S                     0
      0
112A-S           108.22   2.65     0        TABULAR    112A-S                     0
      0
113A-S           108.53   2.5      0        TABULAR    113A-S                     0
      0
116A-S           109.23   2.5      0        TABULAR    116A-S                     0
      0
118A-S           110.06   2.15     0        FUNCTIONAL 0        0        0        0
      0
119A-S           109.73   2.5      0        TABULAR    119A-S                     0
      0
121A-S           109.72   2.5      0        TABULAR    121A-S                     0
      0
121A-S(1)        112.87   0.35     0        FUNCTIONAL 0        0        0        0
      0
121B-S           112.25   0.6      0        FUNCTIONAL 0        0        0        0
      0
203A-S           109.25   2.5      0        TABULAR    203A-S                     0
      0
206A-S           109.73   2.5      0        TABULAR    206A-S                     0
      0
211A-S           109.75   2.5      0        TABULAR    211A-S                     0
      0
218A-S           110.73   2.5      0        TABULAR    218A-S                     0
      0
POND-S           104.25   3.5      1.5      TABULAR    POND                       0
      0

[CONDUITS]
;;               Inlet            Outlet                      Manning    Inlet
Outlet     Init.      Max.
;;Name           Node             Node             Length     N          Offset
Offset     Flow       Flow
;;-------------- ---------------- ---------------- ---------- ---------- ----------
---------- ---------- ----------
C1               121A-S(1)        121A-S           20         0.013      112.87
111.87     0          0
C10              119A-S           113A-S           20         0.013      111.88
110.68     0          0
```

```
C11             119A-S          116A-S          20          0.013       111.88
111.38     0          0
C12             118A-S          116A-S          86          0.013       111.86
111.38     0          0
C13             203A-S          116A-S          4           0.013       111.4
111.38     0          0
C14             206A-S          118A-S          4           0.013       111.88
111.86     0          0
C15             HWL130B         OF3             17.5        0.035       105.7
105.58     0          0
C16             116A-S          113A-S          20          0.013       111.38
110.68     0          0
C17             113A-S          112A-S          20          0.013       110.68
110.37     0          0
C18             111A-S          112A-S          48          0.013       110.59
110.37     0          0
C19             107A-S          106A-S(1)       6           0.013       110.39
110.36     0          0
C2              102             100B            72.87       0.013       106.5
106.28     0          0
C21             106A-S          POND-S          20          0.013       108
107.44     0          0
C23             102A-S          102A-S(1)       5           0.025       110.27
107.5      0          0
C24             100B            POND-S          22          0.013       105.85
105.75     0          0
C25             102A-S(1)       POND-S          10          0.025       107.5
107.35     0          0
C26             112A-S          102A-S(1)       153         0.025       110.37
107.5      0          0
C27             121B-S          121A-S          2           0.025       112.25
112        0          0
C28             109B-S          121A-S          2           0.025       112
111.87     0          0
C29             106B-S          106A-S(1)       2           0.025       110.9
110.36     0          0
C3              121A-S          109C-S          20          0.013       111.87
111.5      0          0
C30             109A-S          109C-S          50          0.013       112
111.5      0          0
C31             outlet          HWL130B         24.7        0.013       105.75
105.7      0          0
C4              109C-S          106A-S(1)       20          0.013       111.5
110.36     0          0
C5              106A-S(1)       106A-S          100         0.013       110.36
108        0          0
C6              108A-S          106A-S(1)       5           0.013       111
110.95     0          0
C7              218A-S          121A-S(1)       2           0.013       112.88
112.87     0          0
C8              121A-S(1)       119A-S          20          0.013       112.87
111.88     0          0
C9              211A-S          119A-S          4           0.013       111.9
111.88     0          0
Pipe_1          102             101             9.87        0.013       105.66
105.64     0          0
Pipe_22         112             103             156.728     0.013       106.241
106.084    0          0
Pipe_23         110             104             140.191     0.013       107.677
107.257    0          0
Pipe_24_(1)     111             110             203.904     0.013       108.349
107.737    0          0
Pipe_27         117             116             85.874      0.013       107.858
```

```
107.729     0           0
Pipe_28           118               117               85.068      0.013       108.21
108.083     0           0
Pipe_29_(1)       116               115               227.891     0.013       107.204
106.976     0           0
Pipe_3_(1)        106               105               212.193     0.013       106.868
106.656     0           0
Pipe_30_(1)       119               115               274.488     0.013       107.538
107.126     0           0
Pipe_31           120               119               53.76       0.013       107.849
107.688     0           0
Pipe_32           121               120               65.24       0.013       108.896
108.374     0           0
Pipe_35           107               106               135.621     0.013       107.454
107.318     0           0
Pipe_43           103               102               45.979      0.013       105.77
105.71      0           0
Pipe_44           113               112               66.392      0.013       107.598
107.366     0           0
Pipe_47           204               117               75.004      0.013       108.045
107.933     0           0
Pipe_48           205               204               82.176      0.013       108.228
108.105     0           0
Pipe_49           206               205               164.822     0.013       108.536
108.288     0           0
Pipe_5            108               106               240         0.013       107.258
107.018     0           0
Pipe_50           200               116               77.768      0.013       107.62
107.504     0           0
Pipe_50_(1)       201               200               158.607     0.013       107.863
107.625     0           0
Pipe_51           202               201               20.319      0.013       107.923
107.893     0           0
Pipe_52           203               202               148.464     0.013       108.176
107.953     0           0
Pipe_53           207               115               79.926      0.013       107.206
107.126     0           0
Pipe_53_(1)       208               207               79.1        0.013       107.288
107.209     0           0
Pipe_54           209               208               264.023     0.013       107.612
107.348     0           0
Pipe_55           210               209               15.557      0.013       107.658
107.642     0           0
Pipe_56           211               210               83.021      0.013       107.771
107.688     0           0
Pipe_58           218               217               188.555     0.013       108.857
108.575     0           0
Pipe_59           217               216               19.773      0.013       108.545
108.515     0           0
Pipe_6            109               108               97.573      0.013       108.004
107.858     0           0
Pipe_61           212               120               89.327      0.013       108.043
107.909     0           0
Pipe_62           213               212               72.822      0.013       108.212
108.103     0           0
Pipe_63           214               213               25.111      0.013       108.31
108.272     0           0
Pipe_63_(1)       215               214               21.157      0.013       108.372
108.34      0           0
Pipe_63_(1)_(1)   216               215               55.562      0.013       108.485
108.402     0           0
Pipe_64           114               112               77.623      0.013       106.379
106.301     0           0
```

```
Pipe_65          115              114              87.275     0.013      106.526
106.439    0          0
Pipe_68          104              103              47.498     0.013      106.582
106.534    0          0
Pipe_69          105              104              11.063     0.013      106.596
106.585    0          0
Pipe_7           101              POND-S           21.827     0.013      105.61
105.55     0          0

[ORIFICES]
;;               Inlet            Outlet           Orifice      Crest      Disch.
 Flap Open/Close
;;Name           Node             Node             Type         Height     Coeff.
 Gate Time
;;-------------- ---------------- ---------------- ------------ ----------
---------- ---- ----------
C22              POND-S           outlet           SIDE         105.75     0.61
 NO   0

[WEIRS]
;;               Inlet            Outlet           Weir         Crest      Disch.
 Flap End      End
;;Name           Node             Node             Type         Height     Coeff.
 Gate Con.     Coeff.     Surcharge  RoadWidth  RoadSurf
;;-------------- ---------------- ---------------- ------------ ----------
---------- ---- -------- ---------- ---------- ---------- ----------
C20              POND-S           OF1              TRANSVERSE   107.65     1.74
 NO   0        0          YES
W1               POND-S           outlet           TRANSVERSE   106.15     1.7
 NO   0        0          YES

[OUTLETS]
;;               Inlet            Outlet           Outflow    Outlet
Qcoeff/                     Flap
;;Name           Node             Node             Height     Type
QTable           Qexpon     Gate
;;-------------- ---------------- ---------------- ---------- ----------------
---------------- ---------- ----
102A-IC          102A-S           102              108.12     TABULAR/HEAD
102A-IC                     NO
106A-IC          106A-S(1)        106              108.56     TABULAR/HEAD
106A-IC                     NO
106B-IC          106B-S           106              109.1      TABULAR/HEAD
106B-IC                     NO
107A-IC          107A-S           107              108.24     TABULAR/HEAD
107A-IC                     NO
108A-IC          108A-S           108              108.85     TABULAR/HEAD
108A-IC                     NO
109A-IC          109A-S           109              110.2      TABULAR/HEAD
109A-IC                     NO
109C-IC          109C-S           109              109.35     TABULAR/HEAD
109C-IC                     NO
111A-IC          111A-S           111              108.44     TABULAR/HEAD
111A-IC                     NO
112A-IC          112A-S           112              108.22     TABULAR/HEAD
112A-IC                     NO
113A-IC          113A-S           113              108.53     TABULAR/HEAD
113A-IC                     NO
116A-IC          116A-S           116              109.23     TABULAR/HEAD
116A-IC                     NO
118A-IC          118A-S           118              110.06     TABULAR/HEAD
118A-IC                     NO
119A-IC          119A-S           119              109.73     TABULAR/HEAD
```

```
119A-IC                          NO
121A-IC          121A-S              121              109.72     TABULAR/HEAD
121A-IC                          NO
203A-IC          203A-S              203              109.25     TABULAR/HEAD
203A-IC                          NO
206A-IC          206A-S              206              109.73     TABULAR/HEAD
206A-IC                          NO
211A-IC          211A-S              211              109.75     TABULAR/HEAD
211A-IC                          NO
218A-IC          218A-S              218              110.73     TABULAR/HEAD
218A-IC                          NO

[XSECTIONS]
;;Link           Shape        Geom1            Geom2      Geom3      Geom4
Barrels
;;-------------- ------------ ---------------- ---------- ---------- ----------
----------
C1               IRREGULAR    24mROW           0          0          0          1

C10              IRREGULAR    18mROW           0          0          0          1

C11              IRREGULAR    24mROW           0          0          0          1

C12              IRREGULAR    24mROW           0          0          0          1

C13              IRREGULAR    18mROW           0          0          0          1

C14              IRREGULAR    18mROW           0          0          0          1

C15              TRAPEZOIDAL  0.65             1          3          3          1

C16              IRREGULAR    18mROW           0          0          0          1

C17              IRREGULAR    18mROW           0          0          0          1

C18              IRREGULAR    18mROW           0          0          0          1

C19              IRREGULAR    18mROW           0          0          0          1

C2               RECT_CLOSED  1.35             2.1        0          0          1

C21              TRAPEZOIDAL  0.5              2          10         10         1

C23              TRAPEZOIDAL  0.5              1          3          3          1

C24              RECT_CLOSED  1.35             2.1        0          0          1

C25              TRAPEZOIDAL  0.5              4          3          3          1

C26              TRAPEZOIDAL  0.5              4          3          3          1

C27              TRIANGULAR   0.6              3.6        0          0          1

C28              TRIANGULAR   0.6              3.6        0          0          1

C29              TRIANGULAR   1                6          0          0          1

C3               IRREGULAR    24mROW           0          0          0          1

C30              IRREGULAR    24mROW           0          0          0          1

C31              CIRCULAR     1.05             0          0          0          1
```

C4	IRREGULAR	24mROW	0	0	0	1
C5	IRREGULAR	24mROW	0	0	0	1
C6	IRREGULAR	16.5mROW	0	0	0	1
C7	IRREGULAR	18mROW	0	0	0	1
C8	IRREGULAR	24mROW	0	0	0	1
C9	IRREGULAR	18mROW	0	0	0	1
Pipe_1	CIRCULAR	1.5	0	0	0	1
Pipe_22	CIRCULAR	1.8	0	0	0	1
Pipe_23	CIRCULAR	0.675	0	0	0	1
Pipe_24_(1)	CIRCULAR	0.675	0	0	0	1
Pipe_27	CIRCULAR	0.825	0	0	0	1
Pipe_28	CIRCULAR	0.6	0	0	0	1
Pipe_29_(1)	CIRCULAR	1.35	0	0	0	1
Pipe_3_(1)	CIRCULAR	1.35	0	0	0	1
Pipe_30_(1)	CIRCULAR	1.2	0	0	0	1
Pipe_31	CIRCULAR	1.05	0	0	0	1
Pipe_32	CIRCULAR	0.525	0	0	0	1
Pipe_35	CIRCULAR	0.9	0	0	0	1
Pipe_43	CIRCULAR	2.1	0	0	0	1
Pipe_44	CIRCULAR	0.675	0	0	0	1
Pipe_47	CIRCULAR	0.75	0	0	0	1
Pipe_48	CIRCULAR	0.75	0	0	0	1
Pipe_49	CIRCULAR	0.75	0	0	0	1
Pipe_5	CIRCULAR	1.2	0	0	0	1
Pipe_50	CIRCULAR	1.05	0	0	0	1
Pipe_50_(1)	CIRCULAR	1.05	0	0	0	1
Pipe_51	CIRCULAR	1.05	0	0	0	1
Pipe_52	CIRCULAR	1.05	0	0	0	1
Pipe_53	CIRCULAR	1.2	0	0	0	1
Pipe_53_(1)	CIRCULAR	1.2	0	0	0	1
Pipe_54	CIRCULAR	1.2	0	0	0	1
Pipe_55	CIRCULAR	1.2	0	0	0	1

```
Pipe_56          CIRCULAR     1.2              0          0          0          1

Pipe_58          CIRCULAR     1.05             0          0          0          1

Pipe_59          CIRCULAR     1.05             0          0          0          1

Pipe_6           CIRCULAR     0.6              0          0          0          1

Pipe_61          CIRCULAR     1.05             0          0          0          1

Pipe_62          CIRCULAR     1.05             0          0          0          1

Pipe_63          CIRCULAR     1.05             0          0          0          1

Pipe_63_(1)      CIRCULAR     1.05             0          0          0          1

Pipe_63_(1)_(1)  CIRCULAR     1.05             0          0          0          1

Pipe_64          CIRCULAR     1.8              0          0          0          1

Pipe_65          CIRCULAR     1.8              0          0          0          1

Pipe_68          CIRCULAR     1.35             0          0          0          1

Pipe_69          CIRCULAR     1.35             0          0          0          1

Pipe_7           CIRCULAR     1.5              0          0          0          1

C22              CIRCULAR     0.25             0          0          0
C20              RECT_OPEN    1                10         3          3
W1               RECT_OPEN    1.15             0.3        0          0

[TRANSECTS]

;Full street, width = 8.5m, curb = 0.15m , cross-slope = 0.02m/m, bank-slope =
0.02m/m, bank-height = 0.23m.
NC 0.02     0.02     0.013
X1 16.5mROW          7        4        12.5     0.0       0.0       0.0      0.0
0.0
GR 0.23     0        0.15     4        0        4         0.13      8.25     0
12.5
GR 0.15     12.5     0.23     16.5

;Full street, width = 8.5m, curb = 0.15m , cross-slope = 0.03m/m, bank-slope =
0.02m/m, bank-height = 0.245m.
NC 0.025    0.025    0.013
X1 18mROW            7        10       18.5     0.0       0.0       0.0      0.0
0.0
GR 0.35     0        0.15     10       0        10        0.13      14.25    0
18.5
GR 0.15     18.5     0.35     28

;Full street, width = 8.5m, curb = 0.15m , cross-slope = 0.03m/m, bank-slope =
0.02m/m, bank-height = 0.27m.
NC 0.02     0.02     0.013
X1 20mROW            7        10       18.5     0.0       0.0       0.0      0.0
0.0
GR 0.35     0        0.15     10       0        10        0.13      14.25    0
18.5
GR 0.15     18.5     0.35     28.5

;Full street, width = 24m, curb = 0.15m , cross-slope = 0.016m/m, bank-slope =
```

```
0.02m/m, bank-height = 0.23m.
NC 0.025    0.025    0.014
X1 24mROW           7         10        21        0.0       0.0       0.0      0.0
0.0
GR 0.35     0        0.15      10        0         10        0.13      15.5     0
21
GR 0.15     21       0.35      31

[LOSSES]
;;Link           Inlet      Outlet     Average    Flap Gate  SeepageRate
;;-------------- ---------- ---------- ---------- ---------- ----------
C2               0          0.39       0          NO         0
C24              0          0.06       0          NO         0
Pipe_1           0          0.39       0          NO         0
Pipe_22          0          1.32       0          NO         0
Pipe_23          0          1.32       0          NO         0
Pipe_24_(1)      0          1.32       0          NO         0
Pipe_27          0          0.06       0          NO         0
Pipe_28          0          0.06       0          NO         0
Pipe_29_(1)      0          0.02       0          NO         0
Pipe_3_(1)       0          0.02       0          NO         0
Pipe_30_(1)      0          0.02       0          NO         0
Pipe_31          0          0.14       0          NO         0
Pipe_32          0          0.21       0          NO         0
Pipe_35          0          1.32       0          NO         0
Pipe_43          0          1.32       0          NO         0
Pipe_44          0          1.32       0          NO         0
Pipe_47          0          1.32       0          NO         0
Pipe_48          0          1.32       0          NO         0
Pipe_49          0          0.02       0          NO         0
Pipe_5           0          0.06       0          NO         0
Pipe_50          0          1.32       0          NO         0
Pipe_50_(1)      0          0.06       0          NO         0
Pipe_51          0          0.39       0          NO         0
Pipe_52          0          0.39       0          NO         0
Pipe_53          0          0.06       0          NO         0
Pipe_53_(1)      0          0.06       0          NO         0
Pipe_54          0          0.02       0          NO         0
Pipe_55          0          0.39       0          NO         0
Pipe_56          0          0.64       0          NO         0
Pipe_58          0          0.39       0          NO         0
Pipe_59          0          0.39       0          NO         0
Pipe_6           0          0.06       0          NO         0
Pipe_61          0          1.32       0          NO         0
Pipe_62          0          1.32       0          NO         0
Pipe_63          0          0.02       0          NO         0
Pipe_63_(1)      0          0.64       0          NO         0
Pipe_63_(1)_(1)  0          0.64       0          NO         0
Pipe_64          0          0.02       0          NO         0
Pipe_65          0          0.02       0          NO         0
Pipe_68          0          1.32       0          NO         0
Pipe_69          0          0.06       0          NO         0
Pipe_7           0          0.64       0          NO         0

[CURVES]
;;Name           Type       X-Value    Y-Value
;;-------------- ---------- ---------- ----------
102A-IC          Rating     0          0
102A-IC                     1.8        315
102A-IC                     2.15       315
102A-IC                     2.65       316

106A-IC          Rating     0          0
```

106A-IC		1.8	207
106A-IC		2.15	207
106B-IC	Rating	0	0
106B-IC		1.8	18
106B-IC		2.8	18
107A-IC	Rating	0	0
107A-IC		1.8	504
107A-IC		2.15	504
107A-IC		2.5	505
108A-IC	Rating	0	0
108A-IC		1.8	518
108A-IC		2.15	518
108A-IC		2.5	519
109A-IC	Rating	0	0
109A-IC		1.8	128
109A-IC		2.15	128
109B-IC	Rating	0	0
109B-IC		1.8	1
109B-IC		2.15	1
109C-IC	Rating	0	0
109C-IC		1.8	40
109C-IC		2.15	41
109C-IC		2.5	41
111A-IC	Rating	0	0
111A-IC		1.8	375
111A-IC		2.15	375
111A-IC		2.5	375
112A-IC	Rating	0	0
112A-IC		1.8	216
112A-IC		2.15	216
112A-IC		2.65	216
113A-IC	Rating	0	0
113A-IC		1.8	418
113A-IC		2.15	418
113A-IC		2.5	418
116A-IC	Rating	0	0
116A-IC		1.8	319
116A-IC		2.15	319
116A-IC		2.5	320
118A-IC	Rating	0	0
118A-IC		1.8	193
118A-IC		2.15	193
119A-IC	Rating	0	0
119A-IC		1.8	473
119A-IC		2.15	473
119A-IC		2.5	474
119A-IC(1)	Rating	0	0
119A-IC(1)		1.8	473
119A-IC(1)		2.15	473

```
121A-IC          Rating     0          0
121A-IC                     1.8        241
121A-IC                     2.15       241
121A-IC                     2.5        242

121B-IC          Rating     0          0
121B-IC                     1.8        1
121B-IC                     2.15       1

203A-IC          Rating     0          0
203A-IC                     1.8        837
203A-IC                     2.15       837
203A-IC                     2.5        837

206A-IC          Rating     0          0
206A-IC                     1.8        337
206A-IC                     2.15       337
206A-IC                     2.5        337

211A-IC          Rating     0          0
211A-IC                     1.8        976
211A-IC                     2.15       976
211A-IC                     2.5        976

218A-IC          Rating     0          0
218A-IC                     1.8        802
218A-IC                     2.15       802
218A-IC                     2.5        802

102A-S           Storage    0          0
102A-S                      1.8        0
102A-S                      2.15       380
102A-S                      2.65       380

106A-S           Storage    0          0
106A-S                      1.8        0
106A-S                      2.15       100

107A-S           Storage    0          0
107A-S                      1.8        0
107A-S                      2.15       450
107A-S                      2.5        450

108A-S           Storage    0          0
108A-S                      1.8        0
108A-S                      2.15       1870
108A-S                      2.5        1870

109A-S           Storage    0          0
109A-S                      1.8        0
109A-S                      2.15       100

109C-S           Storage    0          0
109C-S                      1.8        0
109C-S                      2.15       41
109C-S                      2.5        41

111A-S           Storage    0          0
111A-S                      1.8        0
111A-S                      2.15       305
111A-S                      2.5        305

112A-S           Storage    0          0
```

112A-S		1.8	0
112A-S		2.15	103
112A-S		2.65	103
113A-S	Storage	0	0
113A-S		1.8	0
113A-S		2.15	265
113A-S		2.5	265
116A-S	Storage	0	0
116A-S		1.8	0
116A-S		2.15	175
116A-S		2.5	175
118A-S	Storage	0	0
118A-S		1.8	0
118A-S		2.15	50
119A-S	Storage	0	0
119A-S		1.8	0
119A-S		2.15	322
119A-S		2.5	322
121A-S	Storage	0	0
121A-S		1.8	0
121A-S		2.15	166
121A-S		2.5	166
203A-S	Storage	0	0
203A-S		1.8	0
203A-S		2.15	695
203A-S		2.5	695
206A-S	Storage	0	0
206A-S		1.8	0
206A-S		2.15	328
206A-S		2.5	328
211A-S	Storage	0	0
211A-S		1.8	0
211A-S		2.15	906
211A-S		2.5	906
218A-S	Storage	0	0
218A-S		1.8	0
218A-S		2.15	695
218A-S		2.5	695
POND	Storage	0.00	4995
POND		1.20	6920
POND		1.30	8659
POND		1.40	10399
POND		1.50	12138
POND		1.90	15072
POND		2.00	15665
POND		2.10	16258
POND		2.75	18009
POND		3.09	18925
POND		3.20	19204
POND		3.50	19500

[REPORT]

```
INPUT       YES
CONTROLS    NO
SUBCATCHMENTS ALL
NODES ALL
LINKS ALL

[TAGS]
Link        C1               MJ
Link        C10              MJ
Link        C11              MJ
Link        C12              MJ
Link        C13              MJ
Link        C14              MJ
Link        C15              mj
Link        C16              MJ
Link        C17              MJ
Link        C18              MJ
Link        C19              MJ
Link        C21              MJ
Link        C23              MJ
Link        C25              MJ
Link        C26              MJ
Link        C27              MJ
Link        C28              MJ
Link        C29              MJ
Link        C3               MJ
Link        C30              MJ
Link        C4               MJ
Link        C5               MJ
Link        C6               MJ
Link        C7               MJ
Link        C8               MJ
Link        C9               MJ
Link        C20              MJ
```

C.4 STORM DESIGN BACKGROUND REPORT EXCERPTS AND CORRESPONDENCE

FERNBANK

COMMUNITY DESIGN PLAN

ENVIRONMENTAL MANAGEMENT PLAN
Volume 1 of 2

As Approved by Council
JUNE 24, 2009

Monahan Drain Modeling Results - Summer Event

For the summer event (24-hr SCS distribution), the Fernbank CDP model provides a very close correlation to the RVCA model. The greatest difference in peak flow occurs for the 100-year event: The 100-year peak flow is 41.8 m^3/s for the Fernbank CDP model vs. 40.0 m^3/s for the RVCA model, a difference of approximately 4.5%.

Modeled peak flows from both the RVCA (2004) and Novatech (2007) simulations are both slightly higher than the peak flows modeled by J.L. Richards in 1993. The primary reason for the increase in flows is that J.L. Richards used IDF data from the former City of Kanata in their analysis, which generate slightly smaller runoff volumes than the current City of Ottawa IDF parameters.

Monahan Drain Modeling Results - Spring Event

The model results for the J.L. Richards spring event have been included in **Table 4-1** for comparison purposes, but it should be noted that the 1993 analysis only considered a 24-hour rain-on-snow event and not a 10-day event.

There is a good correlation between the RVCA and Novatech 100-year peak flows for the spring event (10-day Rain+Snow). The 100-year peak flow is 20.1 m3/s for the Fernbank CDP model vs. 21.0 m^3/s for the RVCA model, a difference of approximately 4.5%. The spring peak flows do not correlate as closely for the more frequent return periods. The primary reason for the difference in peak flows is likely due to the influence of the Monahan Drain Constructed Wetlands: The wetlands are modeled as a discrete element in the Fernbank CDP model, while the RVCA model does not specifically account for storage and routing through the wetlands. The wetlands do significantly attenuate peak flows for smaller storm events, but the attenuation effect is reduced for larger storm events.

It should be noted that the Jock River Flood Risk Mapping - Hydrology Report states "…the calibration/validation effort concentrated on the simulation of high flows for the purpose of flood risk mapping, and that the estimates of more frequent Return Period Flows, such as the 2 year and 5 year, should be used with caution."

The Fernbank CDP SWMHYMO model provides a good correlation of peak flows to the RVCA model for the full range of summer events (24-hr SCS distribution), and good correlation to the RVCA model for the 100-year spring event. Therefore, the Fernbank CDP model of the Monahan Drain will provide a good benchmark for the analysis of impacts resulting from development of the Fernbank CDP on the downstream Monahan and Flewellyn Drains.

Faulkner Drain

The Fernbank CDP lands situated northwest of Shea Road are tributary to the Faulkner Drain, which is in turn tributary to Flowing Creek. The lands within the Fernbank Community represent only 48.5 hectares of the 4945 hectare area comprising the Flowing Creek Watershed (approximately 1%), and any meaningful comparison to the Flowing Creek Subwatershed model used in the Jock River Hydrology Study is not possible for this area.

Existing conditions for the Fernbank CDP lands tributary to the Faulkner Drain have instead been modeled based on the physical characteristics of the watershed. Modeling parameters were derived as follows:

- The soil types (and corresponding CN values) have been verified through test pit data;
- The drainage area has been verified based on detailed topographic mapping;
- The time to peak (t_p) has been calculated based on the average slope, length and land use within the catchment.

Table 4-2: Existing Conditions Peak Flows

	Peak Flow (m^3/s)						
	Distribution	**2yr**	**5yr**	**10yr**	**25yr**	**50yr**	**100yr**
Carp Subwatershed							
Carp Headwaters +	12hr AES	2.53	3.92	4.86	6.08	6.93	7.79
Carp River West Tributary	12hr SCS	2.76	**4.46**	**5.62**	**7.27**	**8.20**	**9.39**
HEC-RAS Station 44751	24hr SCS	**2.91**	4.45	5.52	6.84	7.92	9.38
Fernbank Lands north of West	12hr AES	0.65	1.05	1.33	1.74	2.04	2.36
Tributary + Westcreek Meadows	12hr SCS	0.84	**1.46**	**1.90**	**2.53**	**2.90**	**3.37**
HEC-RAS Station 44548	24hr SCS	**0.89**	1.46	1.86	2.37	2.76	3.34
Hazeldean Creek @	12hr AES	0.91	1.38	1.72	2.19	2.53	2.94
Carp River	12hr SCS	**1.82**	**2.65**	**3.28**	**4.42**	**4.74**	**5.45**
HEC-RAS Station 43966	24hr SCS	1.49	2.16	2.68	3.40	3.98	4.83
Jock Subwatershed							
Monahan Drain @	12hr AES	**1.21**	**1.99**	**2.54**	**3.24**	**3.74**	4.24
Terry Fox Drive	12hr SCS	1.13	1.87	2.39	3.13	3.55	4.10
	24hr SCS	1.21	1.92	2.42	3.05	3.57	**4.28**
Flewellyn Drain @	12hr AES	1.12	**1.83**	**2.33**	**2.97**	**3.42**	3.88
Fernbank Road	12hr SCS	1.05	1.76	2.25	2.97	3.37	3.90
	24hr SCS	**1.13**	1.81	2.28	2.88	3.37	**4.05**
Faulkner Tributary @	12hr AES	0.46	0.74	0.94	1.19	1.37	1.55
Fernbank Road	12hr SCS	0.48	0.82	1.05	1.39	1.58	1.83
	24hr SCS	**0.51**	**0.83**	**1.05**	**1.32**	**1.55**	**1.85**

Critical Storm Distributions

The 12-hour SCS distribution appears to be the critical storm distribution for lands in the Carp River subwatershed. This is consistent with the 12-hour SCS distribution used in the Carp River XP-SWMM hydrologic modeling (CH2MHill, MVC).

The 12 hour AES distribution generates higher peak flows for the more frequent return periods on both the Monahan Drain and the Flewellyn Drain. However, the 24hr SCS distribution generates the highest 100-year peak flows for all three catchment areas in the Jock River subwatershed. The 24-hour distribution was used in the Jock River Flood Risk Mapping analysis (PSR Group, RVCA).

6.4 SWM Criteria - Jock River Subwatershed

Stormwater management criteria for the Fernbank Community lands tributary to the Jock River subwatershed have been developed based on the recommendations of the Jock River Reach 2 River Subwatershed Study and input from RVCA:

- The proposed stormwater management strategy will need to adhere to all applicable policies and guidelines of the Rideau Valley Conservation Authority; the City of Ottawa, MOE, and other approvals agencies.

<u>Quality Control / Fish Habitat</u>

- Level 1 - Enhanced protection for lands tributary to the Jock River (80% long term TSS removal);
- End-of-pipe facilities will be designed to provide extended detention storage for both baseflow enhancement and water quality control.
- The proposed development must have no adverse impacts on downstream fish habitat.
- The Monahan Drain, Flewellyn and Faulkner Drains have been classified as intermittent watercourses that provide indirect habitat supporting tolerant warm/cool water fish communities. Temperature mitigation measures are to be incorporated into all proposed SWM facilities tributary to the Jock River, with the goal of ensuring that the temperature of discharged stormwater does not exceed the following target values:
    - Maximum Discharge Temperature = 25ºC
    - Preferred Discharge Temperature = 22ºC

<u>Quantity Control</u>

- Ensure the proposed SWM infrastructure will not result in any adverse impacts on flood elevations or increase the extent of flooding in downstream watercourses.
- Ensure the Monahan Drain ponds are designed to have no adverse impacts the function of the Monahan Drain Constructed Wetlands SWM Facility. No additional analysis of the Constructed Wetlands will be required provided that the proposed development conforms to the following:
    - The main branch of the Monahan Drain is retained upstream of Terry Fox Drive;
    - Fernbank lands tributary to the Monahan Drain to be serviced by 3 SWM facilities:
        - One SWM facility at the headwaters of the Monahan Drain;
        - Two SWM facilities on each side of the Monahan Drain upstream of Terry Fox Drive.
    - The design of the Constructed Wetlands assumed a total drainage area tributary to the Monahan Drain upstream of Terry Fox Drive of approximately 296 hectares with an average imperviousness of 46%.
- Post-development peak flows are not to exceed pre-development levels for all storms up to the 100-year event.
    - **Pre-Development Peak Flow targets are listed in Table 4-2.**

<u>Erosion control / Fluvial Geomorphology</u>

- Continuous hydrologic modeling should be used to demonstrate that the proposed development will not result in an adverse change to the geomorphology of the outlet watercourses. The number of exceedences of the erosion thresholds established by the fluvial geomorphic analysis should not increase under post-development conditions.
    - **Critical flow (Erosion) targets for watercourses are listed in Table 3-7.**

Section 8.0 Post Development Storm Drainage Conditions

8.1 Hydrology

The post-development hydrologic analysis of the Fernbank community has been completed using the SWMHYMO hydrologic model, and includes both event-based modeling (2-100yr), and continuous modeling using long-term rainfall data for the City of Ottawa. The results of the pre-development analysis were used as a benchmark for the evaluation of post-development conditions.

8.1.1 Storm Drainage Areas

The post-development storm drainage areas used in the hydrologic model are based on the storm drainage area plans developed as part of the master servicing study. Minor system capture rates have been approximated at 100 L/s/ha. Major system storage has been approximated at 50 m^3/ha.

Post-development drainage areas have been established based on the proposed macro grading plan for the road network through the Fernbank Community. The grading plan can be found in the Master Servicing Study. The proposed grading plan results in changes to the drainage areas between the Flewellyn, Faulkner, and Monahan Drains. RVCA has confirmed that the proposed post-development drainage areas are acceptable. Correspondence is provided in **Appendix B**.

8.1.2 Modeling Parameters

The impervious values used in the post-development conditions analysis are based on the proposed land use plan from the Fernbank CDP and correspond to the runoff coefficients used in the storm sewer design sheets from the Master Servicing Study.

- The minor system capture rate was established at 100 L/s/ha.
- Major system storage in roadways was estimated at 50 m^3/ha.

Post-development drainage areas are shown on **Figure 8.1**. Modeling parameters are listed in **Table 8-1**.

Table 8-1: Post-Development Storm Drainage Areas to SWM Facilities

SWM Pond ID	Drainage Area[1] (ha)	Imperviousness		Soil CN	Major System Storage (m^3)	Minor System Capture Rate (m^3/s)
		Directly Connected	Total			
Carp River						
P1	77.13	0.45	0.56	80.5	3,857	7.71
P2	23.14	0.47	0.59	80.5	1,157	2.31
P3	91.68	0.34	0.43	80.5	4,584	9.17
Faulkner Drain						
P4	57.94	0.35	0.44	80.5	2,897	5.79
Flewellyn Drain						
P5	138.56	0.32	0.40	80.5	6,928	13.86
Monahan Drain						
P6	98.65	0.39	0.49	80.5	4,933	9.87
P7	43.09	0.29	0.36	80.5	2,155	4.31
P8	62.57	0.42	0.53	80.5	3,129	6.26

1. Drainage area does not include SWMF Block (refer to Figure 8.1)

FERNBANK COMMUNITY DESIGN PLAN

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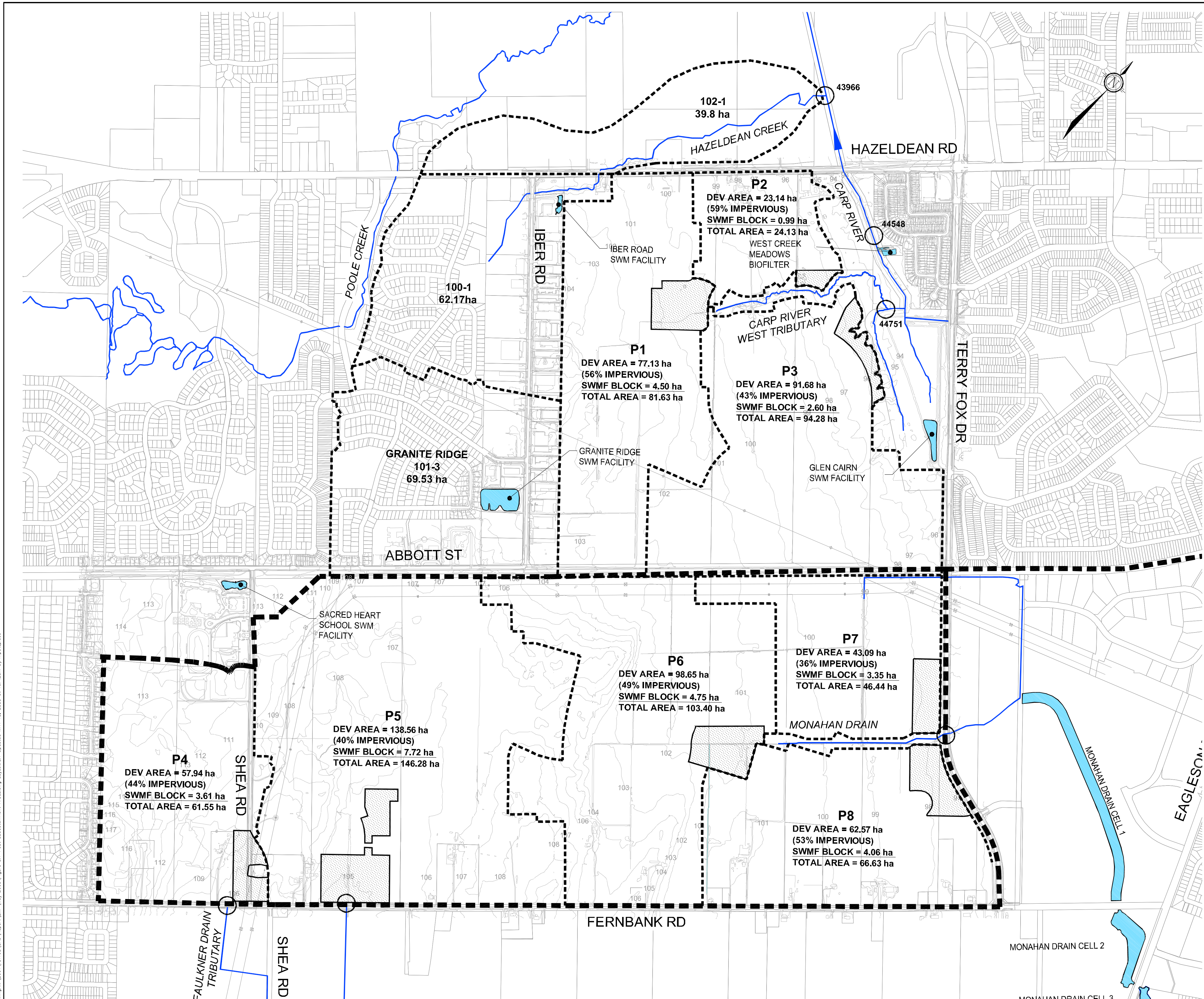
FIGURE 8.1

Post-Development Conditions

Storm Drainage Subcatchments



HYDROGRAPH LOCATIONS

44751 HEC-RAS INFLOW NODE ID (CARP RIVER ONLY)

0 50 100 200 400 600 metres

SCALE : 1:6000 -B1 Sheet
SCALE : Not to Scale -Report

MAY 2009

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9.2 Faulkner Drain SWM Facility

The recommended SWM strategy for the Fernbank Community lands west of Shea Road includes the construction of a SWM facility at the northwest corner of Shea Road and Fernbank Road to provide water quality, erosion, and peak flow control. This facility will outlet to a tributary of the Faulkner Drain that flows southwards from Fernbank Road.

The location of the Faulkner Drain SWM facility is flexible, and two optional locations for this facility were shown in **Figure 7.3** to accommodate current land ownership in this area. This concept shows the facility located partially in the hydro corridor. The placement of the Faulkner Drain can be re-visited as development plans are brought forward in this area.

Conceptual design details for the proposed Faulkner Drain SWM facility (P4) are provided in **Table 9-4**. A conceptual design drawing for this facility is provided as **Figure 9.4**.

Table 9-4: Faulkner Drain SWM Facility (P4)

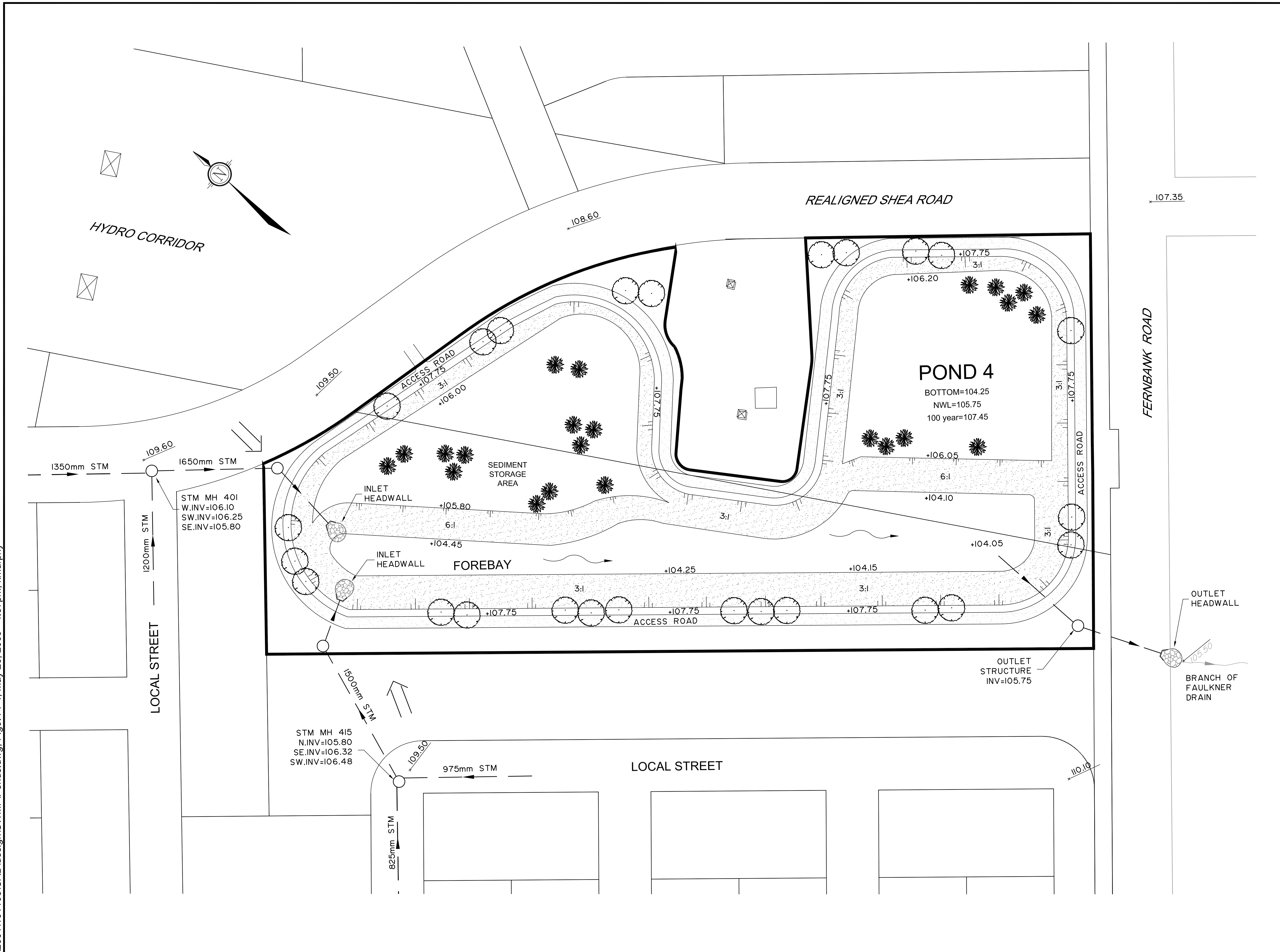
Area of SWM Block	3.61 ha		
Drainage Area to SWMF	57.94 ha	(44% Impervious)	
Quality Control	Enhanced 7,200 m^3 2,400 m^3	(80% TSS Removal) Req. Permanent Pool Volume Req. Extended Detention Volume	
Quantity Control	100yr 1.75 m^3/s	(post-to-pre) Target 100yr Release Rate	
Stage	**Elevation (m)**	**Volume (m3)**	**Release Rate (m^3/s)**
Bottom	104.25	0	0.00
Normal Water Level	105.75	8,700*	0.00
Extended Detention Storage	106.00	2,400	0.04
1:2yr	106.65	13,400	0.29
1:5yr	109.85	18,300	0.45
1:10yr	107.05	21,300	0.67
1:25yr	107.10	24,200	1.05
1:50yr	107.25	26,450	1.35
1:100yr	107.45	29,600	1.75

* Permanent Pool Volume

9.3 Flewellyn Drain SWM Facility

The recommended SWM strategy for the Fernbank Community lands tributary to the Flewellyn Drain includes the construction of a SWM facility to provide water quality, erosion, and peak flow control for the proposed development prior to outletting to the Flewellyn Drain.

Portions of the Flewellyn Drain downstream of the site do not have the capacity to convey the 1:100 year pre-development peak flow, and the increase in runoff associated with development has the potential to increase the extent of flooding in those areas. The facility has been designed to provide reduce post-development peak flows to less than pre-development conditions for larger storm events (>1:10yr event) to reduce the potential for downstream flooding. Storage requirements have been based on providing sufficient storage to control post-development flooding volumes (volume of flow above channel capacity) to pre-development levels.



FERNBANK COMMUNITY DESIGN PLAN

City of Ottawa

ENVIRONMENTAL MANAGEMENT PLAN

FIGURE 9.4

SWMF POND 4

LEGEND

- ACCESS ROAD
- DIRECTION OF POND FLOW
- STORM SEWER AND DIRECTION OF FLOW
- 101.25 / 100.25 — PROPOSED GRADE / EXISTING GRADE
- +96.65 — PROPOSED GRADE
- HEADWALL C/W RIPRAP
- MAJOR OVERLAND FLOW DIRECTION
- NWL — NORMAL WATER LEVEL
- STORM MANHOLE
- 3:1 — TERRACING (MAX SLOPE)
- TREE
- WATER TOLERANT TREE
- EXISTING BRUSH LINE
- EX. NATURAL CHANNEL & DIRECTION OF FLOW

SCALE : 1:1250

JUNE 2009

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TARTAN/CAVANAGH

CONCEPTUAL SITE SERVICING PLAN
STORMWATER MANAGEMENT PLAN AND EROSION AND SEDIMENT CONTROL PLAN
SHEA ROAD LANDS
FERNBANK COMMUNITY

Project: 11218-5.2.2

MARCH 2013

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4. STORMWATER MANAGEMENT

4.1 Background

The stormwater management strategy for the Fernbank Community has been outlined in the Fernbank EMP and MSS. The approved EMP and MSS recommend the construction of one stormwater management facility (referred to as Pond 4) and associated storm sewer systems to provide stormwater management for the Fernbank Community tributary to the Faulkner Drain Tributary. Pond 4 is to provide water quality and quantity control for approximately 60 ha of proposed development northeast of Fernbank Road and Shea Road, and south of Abbott Street East. The proposed SWM facility will outlet to the Faulkner Drain Tributary, which starts on the south side of Fernbank Road. According to the EMP and MSS, the Faulkner Drain Tributary is classified as an intermittent watercourse providing indirect fish habitat.

Under existing conditions, the subject site is comprised of uncultivated grass lands and wooded areas. The site topography is generally between elevations 114 m and 107 m with most of the site draining towards the Faulkner Drain Tributary, itself tributary to the Jock River. The EMP and MSS identify that a northwest portion of the site drains east and south to the Flewellyn Drain, also tributary to the Jock River. The existing topography and general drainage patterns are presented on Figure 7.

In 2012, the City updated its stormwater management criteria from that previously established for the EMP and MSS. The proposed conceptual design is based on the latest stormwater management criteria.

4.2 Objective

The stormwater management objective is to complete the conceptual storm servicing of the lands tributary to Pond 4, as well as to confirm the pond size that was presented in the EMP and MSS.

4.3 Design Constraints and Regulatory Requirements

4.3.1 WATER QUALITY CONTROL

Water quality control targets were established in the "Jock River Reach One Subwatershed Study," (Stantec 2007), and maintained in the EMP and MSS. With respect to suspended solids, it was concluded in the Subwatershed Study that all stormwater facilities discharging to the Jock River must be designed to provide an Enhanced Level of Protection, which corresponds to 80% TSS removal as per the Ontario Ministry of the Environment (MOE) Stormwater Management Planning and Design Manual (March 2003).

According to the EMP and MSS, temperature mitigation measures are to be incorporated into SWM facilities tributary to the Jock River. Specifically, the goal is for the discharged stormwater to not exceed a temperature of 25°C and the preferred temperature identified is 22°C.

4.3.2 WATER QUANTITY CONTROL

Water quantity control criteria for the Faulkner Drain Tributary were outlined in the EMP and MSS. Post-development peak flows are not to exceed pre-development levels for all storm events up to the 100 year event.

potential network for the subject site. Storm sewer sizes are included only for the larger sewers which are also identified in the MSS report. All minor storm sewer sizes will be reviewed and confirmed at the time of detailed design. Major flow will cascade overland from the northern portion of the site downstream to the southern limits of the site, where it will overflow to Pond 4. Conceptual major flow routing is presented on Figure 6.

4.4.2 END-OF-PIPE SWM FACILITY

The conceptual design of Pond 4 was presented in the EMP and MSS. The facility is proposed to be located north of Fernbank Road at the hydro corridor and is designed to provide water quality and water quantity control of stormwater runoff from the subject site. It is designed as a wet pond with two minor system inlets and an outlet to the Faulkner Drain Tributary. In accordance with the EMP and MSS, there are no proposed changes to the Faulkner Drain Tributary. The location of Pond 4 is indicated on Figure 6.

The stage-storage relationship representing Pond 4 in the EMP and MSS has been maintained in the current SWMHYMO evaluation. It is presented in Table 4.1.

Table 4.1 Pond 4 conceptual design data from EMP and MSS

Elevation (m)	Extended Storage (ha-m)
104.25	0
106.00	0.240
106.65	1.340
106.85	1.830
107.05	2.130
107.10	2.420
107.25	2.645
107.45	2.960

As discussed above, the stormwater management facility is designed to provide an Enhanced Level of Protection. According to the MOE Stormwater Management Planning and Design Manual (March 2003), treatment volume is a function of drainage area, the type of pond, the urban imperviousness ratio, and the Level of Protection. The Enhanced Level of Protection corresponds to end-of-pipe storage volumes required for the long-term average removal of 80% of total suspended solids. The storage requirements suggested by the MOE Manual and those provided based on the above conceptual design are summarized in the following table. Supporting calculations are provided in Appendix E.

Table 4.2 Water quality volumes

Enhanced Protection Level 80% Removal of TSS							
Tributary Urban Area (ha)	Imp.% Pond Type Unit Storage	Storage Volume (cu-m)					
		Permanent		Extended Detention		Total	
		Required	Design	Required	Design	Required	Design
56.57	57% Wet Pond 195 cu-m/ha	8749	8700	2263	2400	11012	11100

According to MOE guidelines, the required total storage for the facility is 11012 cu-m. The total water quality storage provided by the proposed facility is 11100 cu-m, which consists of permanent storage of 8700 cu-m and 2400 cu-m of extended detention storage. Due to the increase in imperviousness within the tributary drainage area, the required permanent volume slightly exceeds the permanent volume provided by the conceptual design (by 49 cu-m). This additional volume requirement will be accommodated at the detailed design stage. In practical terms, the bottom of the pond could be lowered by 20 mm to gain the additional volume. The potential to further oversize the permanent volume will be considered at the detailed design stage, as will the use of a bottom-draw outlet, to mitigate temperature increases of the stormwater discharge.

The 25 mm 4 hour Chicago storm simulation indicates that the extended detention drawdown time will be greater than 48 hours. The outflow hydrograph from the facility is presented in Appendix E.

4.5 Hydrological Evaluation

4.5.1 EXISTING CONDITIONS

Existing conditions hydrological analysis was completed in the EMP and MSS. The study area was evaluated using the SWMHYMO computer model. This technique offers single storm event flow generation and routing. The existing conditions model was executed using the 24 hour SCS Type II design storm with 2 through 100 year return periods with a 60 minute time step. Relevant excerpts from the EMP and MSS are enclosed as reference in Appendix D, including a copy of the 100 year existing conditions SWMHYMO input and summary output, as well as Figure 4.1 "Existing Conditions Storm Drainage Subcatchments." A summary of peak flows in the Faulkner Drain Tributary at Fernbank Road are presented in the below table.

Table 4.3 Summary of pre-development flow rates

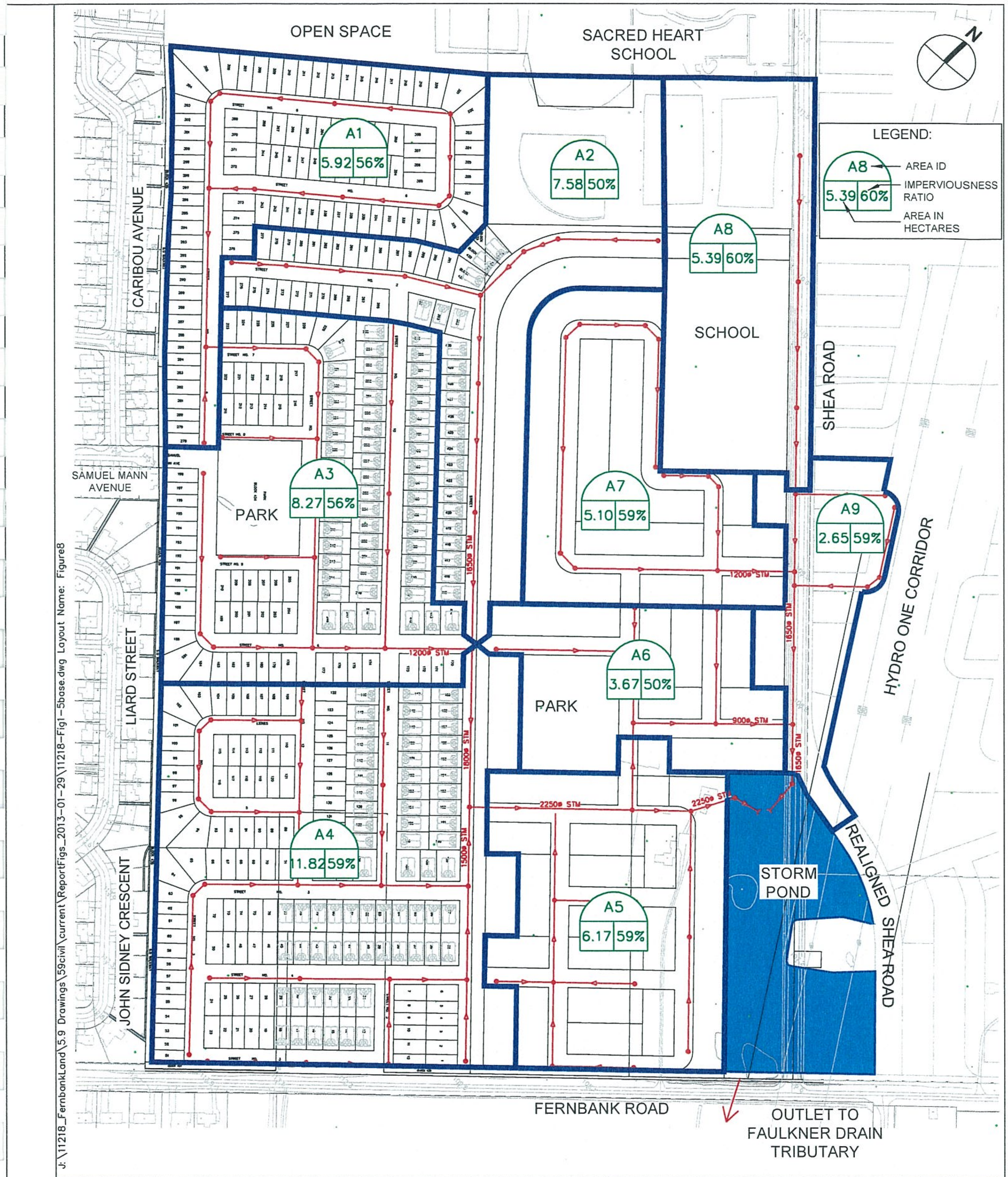
Peak Flow (cms)	24 hour SCS Type II Design Storm		
	2 Year	5 Year	100 Year
Faulkner Drain Tributary at Fernbank Road	0.48	0.76	1.75

4.5.2 POST-DEVELOPMENT CONDITIONS

As mentioned in Section 4.4.1, hydrological analysis of the proposed dual drainage system was conducted using SWMHYMO. This technique offers a single storm event flow generation and routing. The overall post-development SWMHYMO model of the Faulkner Drain Tributary used in the EMP and MSS has been updated to reflect the proposed changes to the tributary drainage area. The model has been revised to isolate the study area tributary to Pond 4. Land use, selected modeling routines, and input parameters are discussed in the following sections. A post-development model schematic and model files are included in Appendix E. As previously noted, the design is currently at a conceptual level and will be further refined at the detailed design stage.

Land Use

The site will be developed as a mixture of low and medium density residential areas with two park areas and a school. The existing recreational area located at the northern end of the site that abuts Shea Road to the west also contributes stormwater runoff to the proposed system. The tributary drainage area was divided into semi-lumped drainage areas reflective of the conceptual design of the minor system. The post-development drainage scheme is indicated in Figure 8.



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CONCEPTUAL SITE SERVICING STUDY
SHEA ROAD LANDS
FERNBANK COMMUNITY

STORM DRAINAGE
AREA PLAN

FIGURE 8

Storms and Drainage Area Parameters

The main hydrology parameters are summarized below and in Table 4.4. Supporting calculations are presented in Appendix E.

- **Design storms:** The site was evaluated using the following storms:
  - 25 mm 4 hour Chicago 12 minute time step (for water quality simulation);
  - 2, 5 and 100 year 24 hour SCS Type II design storm events, 12 minute time step (the EMP and MSS identified the 24 hour SCS Type II as the design storm);
  - 100 year 24 hour SCS Type II design storm events + 20% increase in intensity, 12 minute time step (stress test per City of Ottawa guidelines);
  - 100 year 3 hour Chicago storm event with a 10 minute time step (for dual drainage evaluation, specifically major flow conveyance); and,
  - July 1, 1979, August 4, 1988, and August 8, 1996 storms (historical storms per City of Ottawa guidelines).

- **Area**: Semi-lumped drainage areas are based on the proposed minor system network of storm sewers. The total drainage area contributing minor and major flow to Pond 4 measures 56.57 ha. This is consistent with the drainage area presented in the EMP and MSS of 57.94 ha. The pond block area of 3.61 ha has been maintained as in the EMP and MSS.

- **Imperviousness:** Typical total and directly connected impervious ratios for single family and townhouse units based on typical runoff coefficients have been applied across the site. The calculations to support the runoff coefficients are included in Appendix E. The overall weighted average for the Pond 4 drainage area is 57%, which is higher than the EMP and MSS value of 44%.

- **Minor system capture:** Based on 2012 City guidelines, the minor system capture is based on 5 year rational method flow with a fixed time of concentration of 10 minutes. This results in an overall average unit rate of 170 l/s/ha across the site, which is higher than the EMP and MSS unit flow rate of 100 l/s/ha applied to the Pond 4 drainage area. Supporting calculations are enclosed in Appendix E.

- **Surface storage:** Available surface storage has been approximated based on the macro grading plan. An average unit value of 40 cu-m/ha was applied to the semi-lumped drainage areas in the SWMHYMO model. This value is slightly lower than EMP and MSS value of 50 cu-m/ha.

- **Infiltration:** Infiltration losses were selected to be consistent with the City of Ottawa Sewer Design Guidelines. The Horton values are as follows: f_o = 76.2 mm/h, f_c = 13.2 mm/h, k = 0.00115 s^{-1}.

- **Length:** The impervious length is based on an average of the measured length of the trunk through the catchment and the calculated length based on the SWMHYMO user's manual. The pervious length is based on an average lot depth. This approach is consistent with City of Ottawa Sewer Design Guidelines. Relevant calculations are enclosed in Appendix E.

- **Initial Abstraction (Depression Storage):** Depression storage depths of 1.57 mm and 4.67 mm were used for impervious and pervious areas, respectively. These values are consistent with those in the City of Ottawa Sewer Design Guidelines.

Performance of the Pond

The performance of Pond 4 is summarized in the below table.

Table 4.8 Summary of Pond 4 stage, storage and discharge during various storm events
Post-development conditions SWMHYMO model file: D008.dat

	Storage (ha-m)	Discharge (cms)	Elevation (m)
Permanent Storage	0.87	N/A	105.75
25 mm 4 hour Chicago	0.66*	0.136	106.05
2 year 24 hour SCS Type II	1.19*	0.256	106.31
5 year 24 hour SCS Type II	1.67*	0.397	106.78
100 year 24 hour SCS Type II (Design Storm)	**2.83***	**1.579**	**107.37**
100 year 24 hour SCS Type II + 20%	3.40*	2.314	107.73**
100 year 3 hour Chicago	2.56*	1.230	107.19
July 1 1979	3.39*	2.285	107.72**
August 1988	2.82*	1.574	107.36
August 1996	2.64*	1.341	107.25

** Extended storage only*
*** Extrapolated values*

The maximum water level during the 100 year design storm is 107.37 m, which closely corresponds to the Pond 4 100 year water level published in the EMP and MSS of 107.45 m. It is suggested that at the detailed design stage consideration be given to providing an emergency overflow in the pond at elevation 107.37 m. This provision would allow overflow from the pond during the 100 year 24 hour SCS Type II storm with 20% increase in intensity as well as the July 1, 1979 storm.

4.6 Hydraulic Grade Line Analysis

No Submerged inlets

The maximum water level in the pond is proposed to be 107.37 m during the design storm (100 year 24 hour SCS Type II storm). This is below the obvert elevation of the two inlet pipes to the pond (107.55 m). There is therefore no anticipated hydraulic grade line in the upstream storm system. The required 0.3 m minimum clearance between the underside of footing and storm obvert or hydraulic grade line would therefore be governed by the storm obvert.

The emergency overflow at elevation 107.37 m recommended in Section 4.5.3 would allow for overflow during emergency conditions, preventing the water level in the pond from exceeding the maximum 100 year design water level of 107.37 m.

The hydraulic grade line will be confirmed at the detailed design stage.

From: Paerez, Ana
To: Kilborn, Kris
Subject: RE: Shea Road Lands Cavanagh
Date: Thursday, October 06, 2016 3:51:00 PM
Attachments: image003.png

Hi Kris,
Based on the email below, we will have to restrict post development peak flows from the Tartan/Cavanagh development to approximately 10 year pre-development levels (~0.9 cms). That being said, the SWM pond block will be significantly bigger than what was originally shown in the EMP.
I will be flying to Ottawa on Tuesday and will be at the office on Wednesday so we can touch base then.
Happy Thanksgiving!!
Ana

Ana M. Paerez, P. Eng.
Municipal Engineer
Stantec
Phone: 506-863-0127
Fax: 506-858-8698
ana.paerez@stantec.com

From: Kilborn, Kris
Sent: Thursday, October 06, 2016 2:30 PM
To: Paerez, Ana <Ana.Paerez@stantec.com>
Subject: FW: Shea Road Lands Cavanagh

Ana

Please see email below from Andy Robinson on the flow from pond 4.
I am out of the office on Friday, but maybe we can touch base next Tuesday or wed when you are in Ottawa
Sincerely

Kris Kilborn
Associate, Community Development
Business Center Sector Leader (BCSL)
Stantec
400 - 1331 Clyde Avenue Ottawa ON K2C 3G4
Phone: (613) 724-4337
Cell: (613) 297-0571
Fax: (613) 722-2799
kris.kilborn@stantec.com

From: Andy Robinson [mailto:ajrobinson@rcii.com]
Sent: Thursday, October 06, 2016 11:59 AM
To: 'Surprenant, Eric'; Kilborn, Kris

Cc: Ryan, David W; Gagne, Marc (TUPW)
Subject: RE: Shea Road Lands Cavanagh

Eric & Kris,

In our initial review of the Hydrotechnical Report prepared by Novatech in support of development in Area 6, we pointed out that the culvert under Fernbank Road had a capacity of 0.75 to 0.9 cms. (the 0.9 is with a 0.5 m head). The community development plan used a release rate of 1.75 cms. from Pond 4. Therefore, our position (from the perspective of the flow reaching the Faulkner Municipal Drain) is that the flow from north of Fernbank Road should be limited to a maximum of 0.9 cms.

The update of the Faulkner Municipal Drain is ongoing, but cannot be advance very far until we have the final agreed upon hydrology (pre and post development) and the final plan of the proposed developments (Cavanagh and Tartan) north of Fernbank Road. This includes any modifications to the watershed boundary.

In order to reduce the duplication of effort we generally rely upon the hydrology report completed by the engineer working for the developer, which in turn will have been reviewed and approved by the City. We do review the reports to make sure that we are in agreement. Where there could be a difference for instant is the release rate from Pond 4. From the perspective of the Municipal Drain we will require that the maximum flow rate at Fernbank Road not exceed the present capacity of the controlling culvert under Fernbank Road, whereas the development approvals may be governed by the higher release rate from Pond 4 in the documents supporting the Community Development plan.

Andy

Andy Robinson, P.Eng.
Robinson Consultants Inc.
Ph: (613) 592-6060 ext. 104

From: Surprenant, Eric [mailto:Eric.Surprenant@ottawa.ca]
Sent: October-03-16 12:05 PM
To: 'Kilborn, Kris'
Cc: Ryan, David W; Gagne, Marc (TUPW); Andy Robinson
Subject: RE: Shea Road Lands Cavanagh

Kris,

Sorry for the delay, I started getting back to you but got side tracked. See below my responses in red. Please don't hesitate to contact me if you would like to discuss further.

Thanks

Eric Surprenant, C.E.T. / 613 580-2424 ext.:27794
Project Manager, Infrastructure Approvals

Development Review Suburban Services Branch
Planning, Infrastructure and Economic Development Dept.

Gestionaire de projets, Approbation de l'infrastructure
Examen des demandes d'aménagement (Services Suburbains Ouest)
Services de la planification, de l'infrastructure et du développement économique

City of Ottawa | Ville d'Ottawa
613.580.2424 ext./poste 27794
ottawa.ca/planning / ottawa.ca/urbanisme

From: Kilborn, Kris [mailto:kris.kilborn@stantec.com]
Sent: October 03, 2016 9:53 AM
To: Surprenant, Eric
Subject: RE: Shea Road Lands Cavanagh

Good Monday morning Eric

Just thought I would circle back around on my request for information of sept 21.
Please get back to me at your earliest convenience
Regards

Kris Kilborn
Associate, Community Development
Business Center Sector Leader (BCSL)
Stantec
400 - 1331 Clyde Avenue Ottawa ON K2C 3G4
Phone: (613) 724-4337
Cell: (613) 297-0571
Fax: (613) 722-2799
kris.kilborn@stantec.com

From: Kilborn, Kris
Sent: Wednesday, September 21, 2016 3:19 PM
To: 'Eric.Surprenant@ottawa.ca' (Eric.Surprenant@ottawa.ca) (Eric.Surprenant@ottawa.ca)
Subject: Shea Road Lands Cavanagh

Good afternoon Eric hope all is well.
Further to our preconsultation meeting for the Cavanagh Shea Road Development on July 13 2016 and to the comments received on
July 25, we have reviewed all of the background information for the site, including the previous submission by IBI on Behalf of Tartan and Cavanagh and have a few outstanding items that we would like clarification on from the City.

1. Based on meeting notes from the pre-consultation meeting, the design of Pond 4 needs to include an additional 1,200 m^3. Need confirmation on the design requirements for the pond (i.e. volume requirements, target release rates, imperviousness).

The release rate and Pond 4 design question is somewhat multi-facet. At present the release rate should be coordinated with the work being done by Andy Robinson of Robinson Consultants since Andy is the Drainage Engineer having been appointed through By-Law to look at the Faulkner MD. Pond 4 and this development fall within the drainage area for the Faulkner MD. As discussed at our pre-consult meeting the Fernbank EMP identified a release rate which may be greater than what the Fernbank road crossing culvert may be able to handle at this time, however as stated it will be important to coordinate with the drainage engineer on your design release rate and any additional storage. The Fernbank EMP can be used for the initial pond design, imperviousness etc…however, the revised Sewer Design Guidelines should be applied where your conceptual design is concerned. (Give me a call if you require further clarifications)

2. Based on the pre-consultation meeting notes, there is an on-going review/update of the Faulkner Drain. Need to review the finalized report. Is this report available?

The update to the Drainage Report is currently ongoing. Please see #1. I have copied our Drainage Superintendent(s) and Andy Robinson on this as they may be able to update you further.

3. Please confirm that this development will be utilizing the latest City bulletin with revisions to the SWM guidelines and will l be implemented for this site.

Please see #1.

4. The pre-consultation meeting notes state "reconcile and review the release rate across Fernbank Road". Need clarification. Is there a document where this information can be obtained?

Please see #1.

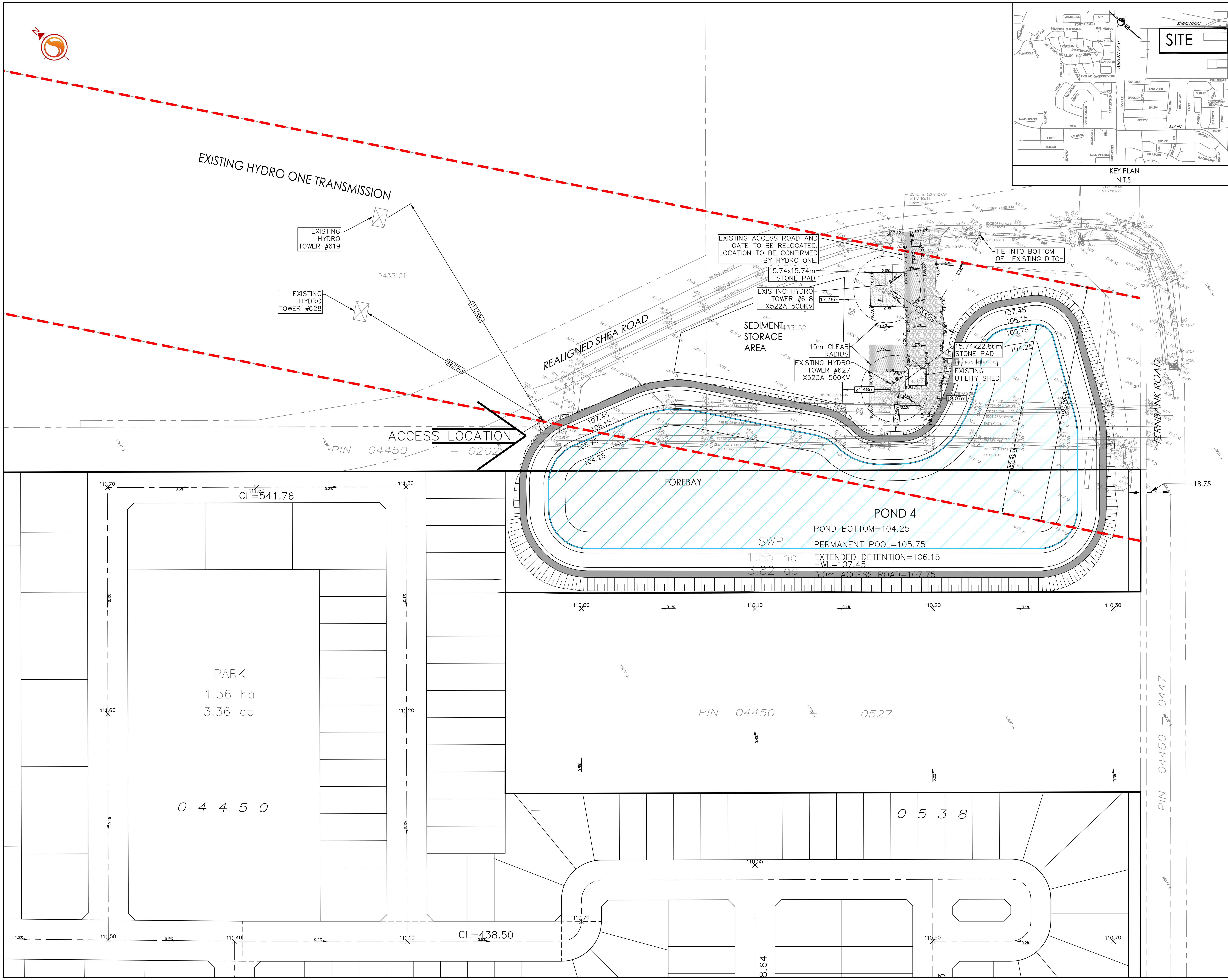
5. IBi's servicing report for Tartan and Cavanagh's lands state that the urban boundary was extended southerly resulting in extended alignment of the sanitary trunk sewer and an increase in peak flows to be routed through our site. Need additional information on the additional drainage area, land use and location. Is there a master servicing report or Drainage Drawings available for the lands south of Fernbank Road.

Area 6 lands will be draining through the Fernbank Lands. Presently a section of the oversized sanitary sewer is being designed through the CRT lands (in Goldhawk Drive). Coordination of the oversized sanitary sewer to accommodate Area 6 flows should be coordinated with CRT's consultants (IBI) and Novatech who are designing the new Area 6 Pump Station and it's forcemains.

Please get back to me at your earliest convenience

Sincerely

Kris Kilborn
Associate, Community Development
Business Center Sector Leader (BCSL)
Stantec
400 - 1331 Clyde Avenue Ottawa ON K2C 3G4
Phone: (613) 724-4337
Cell: (613) 297-0571
Fax: (613) 722-2799
kris.kilborn@stantec.com



Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

Legend

106.15 PROPOSED ELEVATION
106.88 EXISTING ELEVATION
8.07m DIMENSION FROM HYDRO ONE FACILITIES TO PROPOSED GRADE CHANGE
ACCESS LOCATION POND ACCESS
EXISTING GRAVEL ACCESS TO BE REINSTATED
EXISTING GRAVEL ACCESS TO BE REMOVED
PROPOSED GRAVEL ACCESS

PERMANENT POOL (105.75):
A=13860m²
V=17250m³

EXTENDED DETENTION (106.15):
A=15140m²
V=5800m³

Notes

Revision		By	Appd.	YY.MM.DD
3	REVISED ACCESS	WAJ	KJK	17.10.19
2	REVISED ACCESS AND PAD LOCATIONS	WAJ	KJK	17.08.31
1	REVISED AS PER HYDRO ONE COMMENTS	WAJ	KJK	17.07.13

File Name: 160400900-DB	WAJ	KJK	WAJ	16.11.29
	Dwn.	Chkd.	Dsgn.	YY.MM.DD

Permit-Seal

Client/Project
1384341 ONTARIO LTD.

SHEA ROAD LANDS

OTTAWA, ON

Title
PRELIMINARY
SITE AND GRADING PLAN
HYDRO ONE CORRIDOR

Project No. 160400900
Scale 1:750 0 7.5 22.5 37.5m

Drawing No. GP-1
Sheet 1 of 4
Revision 3

W:\active\160400900_cavanagh_stittsville\design\drawing\160400900-HYDRO-CIRC.dwg
2017/10/25 12:08 PM By: Johnson, Warren
ORIGINAL SHEET - ARCH D

Appendix D GEOTECHNICAL INVESTIGATION EXCERPTS

October 2011

REPORT

REPORT ON

Geotechnical Investigation
Proposed Residential Development
Shea Road
Ottawa, Ontario

Submitted to:
1578051 Ontario Inc.
1384341 Ontario Ltd.
C/O Tartan Land Corporation
237 Somerset Street West
Ottawa, Ontario
K2P 0J3

Report Number: 10-1121-0176-3000-4000

Distribution:

2 copies - Tartan Land Corporation

2 copies - Thomas Cavanaugh Construction Ltd.

2 copies - Golder Associates Ltd.

Golder Associates

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APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the site of a proposed residential development to be located just west of Shea Road in the Stittsville community in Ottawa, Ontario.

The geotechnical investigation included an evaluation of the general soil, bedrock, and groundwater conditions across the site by means of 17 test pits. Based on an interpretation of the factual information obtained, along with existing subsurface information for the site, engineering guidelines are provided on the geotechnical design aspects of the project, including construction considerations which could affect design decisions.

The reader is referred to the "Important Information and Limitations of this Report" which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared to develop a residential subdivision on a parcel of land located on the north side of Fernbank Road and on the west side of Shea Road, in the south part of the Stittsville community, in Ottawa, Ontario (see Key Plan, Figure 1).

The site is bounded to the east by Shea Road, to the south by Fernbank Road, to the west by an existing residential development, and to the north by the Goulbourn Recreation Complex and Sacred Heart High School. The property measures approximately 1,000 by 600 metres in size.

An existing residence is located in the southeast part of the site, however the remainder of the site is undeveloped. The majority of the east portion of the site has been cleared of trees, however berms of material traverse the site. The west portion of the site is currently forested with dense and mature tree cover. The site is relatively level. Poorly drained (swampy) ground exists on the north third of the west side of the site.

A preliminary geotechnical investigation for this site was previously carried out by Golder Associates, and the results were provided in a report to Tartan Development Corporation titled "Preliminary Geotechnical Investigation, Proposed Residential Development Site, Fernbank and Shea Roads, Ottawa, Ontario" dated March 2003 (report no. 03-1120-028). That investigation included nine test pits (numbered 03-1 to 03-9, inclusive) and one auger hole (03-10) advanced across the site.

In addition, the Houle Chevrier Engineering Ltd. investigation for the Community Design Plan for the overall development area included several test pits on the eastern portion of the site. The results of that investigation were included in a report to Novatech Engineering Consultants titled "Preliminary Geotechnical Investigation, Fernbank Community Design Plan, Ottawa, Ontario" dated May 2007 (report no. 06-384). Nine test pits (numbered 51 to 59, inclusive) from that investigation are located on the east part of the site.

Based on the results of these previous investigations, the subsurface conditions are expected to consist of sand and glacial till overlying limestone bedrock at shallow depth. Limited thicknesses of peat and marl were encountered on the northwest portion of the property.

Published geologic mapping indicates that the bedrock in the area of this site consists of limestone and dolostone of the Gull River formation.

3.0 PROCEDURES

The field work for this investigation was carried out on May 12, 2011. At that time, seventeen test pits (numbered 11-1 to 11-16, inclusive, and 11-18) were put down at the approximate locations shown on Figure 2.

It had been planned to excavate an additional test pit (to be numbered 11-17) in the southeast corner of the site, however that location was inaccessible.

The test pits were excavated using a track-mounted hydraulic excavator supplied and operated by Thomas Cavanaugh Construction Ltd. of Ottawa, Ontario. The test pits were generally excavated within proposed roadway areas, to avoid disturbing subgrade soils within the future house footprints. The test pits were all advanced to practical refusal to excavating, on the bedrock, at depths ranging from approximately 0.2 to 2.9 metres below the existing ground surface.

The soils exposed on the sides of the test pits were classified by visual and tactile examination. Chunk samples were obtained from the major soil strata encountered in the test pits. The groundwater seepage conditions were observed in the open test pits. The test pits were loosely backfilled upon completion of excavating and sampling.

The field work for this investigation was supervised by a member of our technical staff who located the test pits, directed the excavating operations, logged the test pits and samples, and took custody of the samples retrieved.

Upon completion of the field work, samples of the soils encountered in the test pits were transported to our laboratory for examination by the project engineer and for laboratory testing.

Samples of soil from test pits 11-4 and 11-12 were submitted to Exova Laboratories Ltd. for chemical analysis related to potential corrosion of buried steel elements and potential sulphate attack on buried concrete elements.

The test pit locations were selected by Golder Associates and surveyed and located on-site by Thomas Cavanaugh Construction Ltd. The bedrock surface elevations were also surveyed at each test pit location. The elevations are understood to be referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is provided as follows:

- The test pit records for the present investigation, provided in Table 1.
- The test pit and augerhole records for the previous preliminary investigation by Golder Associates, provided in Appendix A.
- The applicable test pit records from the previous Houle Chevrier investigation, provided in Appendix B.
- The results of the basic chemical analysis from the current investigation, provided in Appendix C.

The subsurface conditions on this site generally consist of a discontinuous sand layer underlain by glacial till, overlying limestone bedrock. The bedrock surface typically exists at depths ranging up to about 3 metres below the existing ground surface, but is locally deeper than about 4 metres. Organic soils (peat and marl) exist within the northwest part of the site.

The following sections present an overview of the subsurface conditions encountered in the test holes from both the previous and current investigations. It should be noted that the ground surface elevations had not been determined at the test pit locations from the previous investigation (test pits 51 to 59 and 03-1 to 03-9, inclusive).

4.2 Fill Material

Fill materials were previously encountered in test pits 03-6 and 03-9 located on the eastern and western edges of the site, respectively. The fill generally consists of silty sand or organic material with variable amounts of gravel, cobbles and boulders. The fill thickness ranges from approximately 0.3 to 0.5 metres.

4.3 Topsoil

Topsoil exists at ground surface or underlies the fill materials at almost all of the test pit locations. The topsoil thickness ranges from approximately 0.1 to 0.3 metres.

4.4 Peat

A layer of peat exists at ground surface within the northwest corner of the site. The peat thickness ranges from approximately 0.4 to 0.8 metres.

4.5 Marl

The peat is commonly underlain by a deposit of marl which typically consists of light grey brown clayey silt with shells. The thickness of the marl deposit ranges from approximately 0.2 to 0.4 metres. The natural water content of the marl was measured at 130% in test pit 11-5 at a depth of 0.9 metres.

4.6 Sand

A deposit of fine sand was encountered in some areas below the topsoil layer (and/or the peat and marl), mostly located at the north and east boundaries of the site and in some areas within the south portion of the site. The sand thickness ranges from approximately 0.1 to 0.6 metres.

4.7 Silty Clay

Test pit 53, excavated by Houle Chevrier in the extreme northeast corner of the site, encountered 0.4 metres of stiff weathered silty clay underlying the topsoil layer.

4.8 Glacial Till

A deposit of glacial till underlies the sand or organic soils at almost all the test pit locations, with the exception of a limited area in the southwest part of the site (test pits 11-2, 11-3 and 11-6) where no glacial till is present. In general, the glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt, silty sand, or clayey silt.

The glacial till was fully penetrated at most of the test pits at depths ranging up to approximately 2.9 metres below the existing ground surface.

Practical refusal to excavating on boulders in the till was also encountered in some test pits (52, 53, 57, 58, and 59, all located in the east part of the site) while test pit 03-6 (southeast part of the site) was terminated without penetrating the glacial till at about 3.3 metres depth.

The natural water content of the glacial till was measured at 19 percent in test pit 11-4 at a depth of 1.5 metres.

4.9 Bedrock

Limestone bedrock was encountered beneath the overburden soils at most of the test pits, except as noted in Section 4.8 where refusal to excavating was encountered within the glacial till and/or the glacial till was not penetrated.

Where encountered, the bedrock surface exists at depths ranging from about 0.2 to 2.9 metres below the existing ground surface. The shallowest bedrock (being essentially at ground surface) exists in the southwest part of the site, around test pits 11-2, 11-3, and 11-6.

4.10 Groundwater

The groundwater conditions were observed in the test pits during the short time that they remained open.

Groundwater seepage was typically observed at depths ranging from about 0.2 to 2.7 metres below the existing ground surface, however many of the test pits were also dry (i.e., no seepage was observed). Groundwater seepage was more commonly observed in the northwest part of the site versus the remaining areas.

In test pit 11-15 (located in the north part of the site) a concentrated and significant inflow of water was observed from the bedrock surface at the bottom of the test pit. It is possible that the inflow is due to a previously drilled borehole/well and that there exists an artesian condition within the bedrock.

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the test pit information and project requirements, and is subject to the limitations in the "Important Information and Limitations of This Report' which follows the text but forms an integral part of this report.

5.2 Site Grading

The subsurface conditions on this site generally consist of a discontinuous sand layer underlain by glacial till, overlying limestone bedrock. The bedrock surface typically exists at depths ranging up to about 3 metres below the existing ground surface, but is locally deeper than about 4 metres. Organic soils (peat and marl) exist within the northwest part of the site. The groundwater level, as observed in several of the open test pits, was encountered at depth ranging from 0.2 to 2.7 metres below ground surface, but in many locations the overburden was 'dry' at the time of investigation (i.e., the groundwater level is in the underlying bedrock).

From a foundation design perspective, no restrictions apply to the thickness of grade raise fill that may be placed within the proposed residential development area (in terms of the compressibility of the subgrade soils).

The presence of shallow bedrock, and minimizing the amount of bedrock excavation (which is costly) should also be a consideration in the design of the site grading.

Due to the potential for significant and permanent flow into the foundation drainage systems, it would be preferred to not design the site grading such that basements would be constructed in bedrock below the groundwater level. If that grading objective is not feasible, further hydrogeologic assessment of the potential inflow may be required.

As a more general guideline, for predictable performance of the structures, roadways, and site services, preparation for filling of the site should include stripping the existing topsoil, localized fill, peat and marl. The topsoil, peat, and marl are not suitable as general fill and should be stockpiled separately for re-use in landscaping applications only. In areas with no structures, roadways, or services, the existing fill, topsoil, peat, and marl may possibly be left in place provided some settlement of the ground surface following filling can be tolerated. The peat may, however, potentially generate methane over time as the peat decomposes and may therefore also best be removed.

5.3 Foundation Design

The peat and marl, as well as the random fill materials on the perimeter of the site, are not suitable for the support of permanent structures and should be removed from structure areas.

It is considered, however, that the proposed residences could be founded on spread footing foundations supported directly on or within the native sand, weathered silty clay (encountered in one test pit only), glacial till or bedrock.

For design purposes, the allowable bearing pressures for spread footings may be taken as 75 kilopascals for the sand, weathered silty clay, and glacial till provided these soils have not been disturbed by groundwater inflow or construction traffic. For footings founded on or within bedrock, an allowable pressure of 250 kilopascals may be used.

The post-construction total and differential settlements of footings supported on soil and sized using the above maximum allowable bearing pressures should be less than 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction. Suitable control of the groundwater inflow is required if such disturbance is to be avoided.

Footings on bedrock should experience negligible settlements.

Based on the above maximum allowable bearing pressures, the house footings may be sized in accordance with Part 9 of the Ontario Building Code.

The glacial till on this site contains cobbles and boulders. If those boulders extend below founding level and are dislodged by the excavator, the soils around the boulders will have become disturbed. In that case, the boulders will need to be fully removed (and not pushed back into place) and the void filled with concrete. Otherwise recompression of the disturbed soils could lead to larger than expected post-construction settlements.

At some locations on the property, and depending on the amount of proposed grade raise (i.e., filling), the inorganic subgrade elevation may be lower than the underside of footing elevation. At these locations, the subgrade may be raised to the footing elevation using engineered fill consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type II, placed in maximum 300 millimetre thick lifts, and compacted to 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The engineered fill material must be placed within the full zone of influence/support of the house foundations, which is considered to extend out and down from the edge of the perimeter footings at a slope of 1 horizontal to 1 vertical.

Where the subgrade at footing level changes from bedrock to overburden, differential settlement could result at this transition due to the different settlement properties of these materials. To limit the magnitude of the differential settlement, transition details (such as placing additional reinforcing steel in the foundation walls, or removing additional bedrock to provide a more gradual transition) may be required. The details will need to be developed on a case-by-case basis, and the structural engineering consultant will need to be involved in the development of those measures. Wherever possible, it is recommended that individual units all be founded on the same medium, i.e., all soil or all bedrock.

Wherever the shallow sandy deposits will be exposed at footing/subgrade level, the surface of these soils should be proof-rolled to provide surficial densification of any loose or disturbed material, prior to construction of footings or the placement of engineered fill. The proof rolling would also densify any loose soils and therefore make them non-liquefiable (i.e., not subject to potential temporary strength loss and post-earthquake settlements).

5.4 Foundation Seismic Design

The seismic design provisions of the 2006 Ontario Building Code (OBC) for structures designed in accordance with Part 4 of the OBC depend, in part, on the shear wave velocity of the upper 30 metres of soil and/or bedrock below founding level. The OBC also permits the Site Class to be specified based solely on the stratigraphy and in situ testing data (e.g., shear strengths and standard penetration test results), rather than from direct measurement of the shear wave velocity. Using that methodology, the applicable Site Class for this site would be 'D' or better, which permits conventional foundation design for this site.

The following additional issues should be noted:

- For the northwest part of the site, this assessment is contingent upon the peat and marl being removed, as specified in Sections 5.2 and 5.3 of this report.
- Since this assessment was carried out using the stratigraphy and in situ test data, it could be conservative. Geophysical measurement of the shear wave velocity for the upper 30 metres of soil and/or bedrock below founding level on this site might allow a more accurate/favourable seismic Site Class to be specified, and to help define parts of the site with different Site Class values. This Site Class would certainly be conservative for structures founded on bedrock.

- The seismic Site Class is not directly applicable to structures designed in accordance with Part 9 of the OBC (i.e., conventional housing), however this assessment is provided to address City of Ottawa requirements that relate to housing on Site Class E sites.
- The soils on this site are not considered to be susceptible to liquefaction during seismic events. Seismic liquefaction is a temporary loss of strength and subsequent compaction/settlement of a soil due to earthquake vibrations. Soils which are more vulnerable to liquefaction are those which are coarse grained (i.e., more probable for sands than for silts), loose, and located below the groundwater level. Although thin surficial sandy soils are present on portions of the site, these soils are to be proof-rolled within foundation areas (as described in Section 5.4) and would therefore be non-liquefiable.

5.5 Frost Protection

The native soils at this site are considered to be frost susceptible. The limestone bedrock may also be frost susceptible if it contains seams within the depth of frost penetration that are filled with frost susceptible soil.

For frost protection purposes, all exterior footings or interior footings in unheated areas should be provided with a minimum of 1.5 metres of earth cover. Isolated, exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

Insulation of the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection. The details for footing insulation could be provided, if and when required.

5.6 Basement Excavations

Excavations for basement areas and the construction of foundation elements will be through topsoil, sands, and glacial till. Depending on the site grading, bedrock excavation may also be required in some areas.

No unusual problems are anticipated in excavating the overburden using conventional hydraulic excavating equipment, recognizing that large boulders may be encountered in the glacial till.

For the shallow anticipated depth of excavation for the basements, side slopes in the overburden materials should be stable in the short term at 1H:1V in accordance with the Occupational Health and Safety Act (OHSA) of Ontario for Type 3 soils.

It has been assumed that the peat and marl would be fully removed from the housing areas prior to the basement excavations being made. If not, however, it should be noted that these soils would be classified as Type 4 soils in accordance with the Occupational Health and Safety Act of Ontario, and excavations side slopes would need to be cut back at 3H:1V. If the thin surficial sand layer is wet at the time of construction, it would also be classified as a Type 4 soil.

Boulders larger than 0.3 metres in size should be removed from the excavation side slopes.

Bedrock removal could be carried out by blasting or hoe ramming. The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. Blast designs should be prepared by a specialist in this field.

A pre-blast survey should be carried out of all of the surrounding structures. Selected existing interior and exterior cracks in the structures should be identified during the pre-blast survey and should be monitored for lateral or shear movements, such as by means of telltales.

The contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This submission would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

The contractor should be limited to only small controlled shots. The following frequency dependent peak vibration limits at the nearest structures and services are suggested.

Frequency Range (Hz)	Vibration Limits (millimetres/second)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting operations be carried out both in the ground adjacent to the closest structures and within the structures themselves.

Near-vertical excavation side slopes in the bedrock should stand unsupported for the construction period.

Some groundwater inflow into the excavations could be expected. However, for the planned basement excavation depths, it should generally be possible to handle the groundwater inflow by pumping from well filtered sumps in the excavations.

For the northwest part of the site, where the organic soils are present and the groundwater level appears to be shallower, there will be more potential for significant groundwater inflow. In this area, the excavations may be subject to disturbance caused by the upward flow of groundwater, resulting in possible disturbance of the excavation subgrade. Some pre-drainage of the site using ditching would be advisable. The construction of the site services may also lower the groundwater level in advance of foundation construction. Where the subgrade is found to be wet and sensitive to disturbance, consideration should be given to placing a working pad consisting of a 150 millimetre thick layer of OPSS Granular A underlain by a non-woven geotextile, to protect the subgrade from construction traffic.

At test pit 11-15 (located in the north part of the site) a concentrated and significant inflow of water was observed from the bedrock surface at the bottom of the test pit. It is possible that the inflow is due to a previously drilled borehole/well and that there exists an artesian condition within the bedrock. Further investigation of this area may be warranted.

5.7 Basement Floor Slabs

In preparation for the construction of basement floor slabs, all loose, wet, and disturbed material should be removed from beneath the floor slab. As previously described, the topsoil, peat, and marl should also be removed.

Provision should be made for at least 200 millimetres of 19 millimetre clear crushed stone to form the base of the floor slab. The underslab fill should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

Wherever the founding level will be below the current/natural ground surface level, there is the potential for the groundwater level to be above the founding level. To prevent hydrostatic pressure build up beneath the floor slab, wherever this is the case, it is suggested that the granular base for the floor slab be drained. This could be achieved by providing a hydraulic link between the underfloor fill and the exterior drainage system. A typical detail for this hydraulic link is provided on Figure 3. As a general guideline, it would be appropriate to provide one link at the front and one link at the rear of each unit (i.e., for each single family house and for each townhouse unit).

Where the footing level is below the natural groundwater level and supported on the sandy soil (rather than the glacial till or bedrock), there would be the potential for the groundwater flow into the underslab drainage system to cause soil particles from the sandy subgrade soils to migrate into the underslab clear stone fill. In the extreme case, loss of fines into the clear stone could cause ground loss beneath the slab (leading to slab/foundation settlement) and plugging of the drainage system. Therefore, where that is the case, the clear stone should be separated from the subgrade soils with a Class II non-woven geotextile, in accordance with OPSS 1860, having a Filtration Opening Size (FOS) not exceeding 150 microns.

5.8 Basement Walls and Foundation Wall Backfill

The soils at this site are considered to be frost susceptible and should not be used as backfill directly against exterior, unheated, or well insulated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should either be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular B Type I or, alternatively, a bond break such as the Platon system sheeting could be placed against the foundation walls.

Drainage of the wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit. Conventional damp proofing of the basement walls is appropriate with the above design approach.

Should the foundations be designed in accordance with Part 4 of the Ontario Building Code, further guidelines on the foundation wall design will need to be provided.

5.9 Site Servicing

Excavations for the installation of site services will be through the overburden soils and, at least on some portions of the site, will likely extend into the limestone bedrock. Based on the observed groundwater conditions in the open test pits, many of these excavations will be below the groundwater level.

The excavation guidelines provided in Section 5.6 are also generally applicable to the service trench excavations, with the following additional notes:

- Due to the sequence of site development, it is possible that the services will be installed before the peat and marl have been fully removed from the northwest part of the site. These soils would be classified as Type 4 soils in accordance with the OHSA, and excavations side slopes would need to be cut back at 3H:1V.
- In general, it may be more practical for the excavations in the overburden to be could be carried out using steeper side slopes (than specified per the OHSA) with all manual labour carried out within a fully braced, steel trench box for worker safety.
- For any significant depth of excavation into the bedrock, hoe-ramming would likely to too slow to be economic. It is expected that drill and blast methods will be required.

- For greater depths of excavation into the bedrock, the rate of groundwater inflow may be significant. Some significant pumping may be required, particularly in view of the concentrated and significant inflow experienced at test pit 11-15. If the servicing design will require significant depths of excavation into the bedrock, further investigation of the hydraulic properties of the bedrock may be required.
- A Permit-To-Take-Water (PTTW) should be obtained from the provincial Ministry of the Environment (MOE) for this work.

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the standard Proctor maximum dry density.

It is should be generally acceptable to re-use the excavated overburden soils as trench backfill. Some of the deeper overburden materials may be too wet to compact. Where that is the case, the wet materials should be wasted (and drier materials imported) or these materials should be placed only in the lower portions of the trench, recognizing that some future settlement of the roadways may occur and some significant padding of the roadways may be required prior to final paving. In that case, it would also be prudent to delay final paving for as long as practical. Topsoil, peat, and marl should not be re-used as trench backfill.

Well fractured or well broken bedrock will be acceptable as backfill for the lower portion of the service trenches in areas where the excavation is in rock. The rock fill, however, should only be placed from at least 300 millimetres above the pipes to minimize damage due to impact or point load. The rock fill should be limited to a maximum of 300 millimetres in size.

In areas where the trench will be covered with hard surfaced materials, the type of material placed within the frost zone (between finished grade and two metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density.

5.10 Pavement Design

In preparation for pavement construction, all topsoil, peat, marl and disturbed/deleterious material should be removed from all pavement areas.

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable compaction equipment.

Transitions from bedrock to earth subgrades (if this condition is encountered) should be carried out in accordance with the OPSD 205 series. The transition depth “t” should be taken as 1.8 metres.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres in four orthogonal directions or longitudinally where parallel to a curb.

The pavement structure for local roads should be:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	375

The pavement structure for collector roadways should be:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

Where the subgrade consists of bedrock, the subbase thickness could be reduced to 300mm, or the equivalent of rock shatter.

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with Table 9 of OPSS 310.

The composition of the asphaltic concrete pavement should be as follows:

Superpave 12.5 mm Surface Course - 40 millimetres

Superpave 19 mm Base Course - 50 millimetres

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B for local roads and Category C for collector roads.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.11 Corrosion and Cement Type

Samples of soil from test pits 11-4 and 11-12 were submitted to Exova Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried ferrous elements and potential sulphate attack on buried concrete elements. The results of this testing are provided in Appendix C.

The results indicate that Type GU cement should be acceptable for substructures and that there is a potential for corrosion of exposed ferrous metal.

6.0 ADDITIONAL CONSIDERATIONS

The soils on this site are sensitive to disturbance from ponded water, construction traffic, and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soils having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill as well as sewer bedding and backfill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

The test pits were loosely backfilled upon completion of excavating and therefore constitute zones of disturbance. The locations were selected to be within roadways such that they are not located beneath future foundations. However, should the development layout change and such that some of those locations would be located within future house areas, then those test pits will need to be repaired by replacing the backfill with compacted engineered fill. Otherwise houses constructed on the test pit backfill could experience unacceptable settlements.

We trust this report contains sufficient information for your present requirements. If you have any questions concerning this report, or if we can be of further service to you on this project, please call us.

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
M. I. CUNNINGHAM
PROVINCE OF ONTARIO

Mike Cunningham, P.Eng
Associate

JMC/MIC/tm/bg
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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, Tartan Land Corporation. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

TABLE 1

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 11-01 (117.10m)	0.00 – 0.20 0.20 – 1.75 1.75	TOPSOIL Brown sandy silt and clayey silt, some gravel with occasional cobbles and boulders (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion.
TP 11-02 (114.76m)	0.00 – 0.20 0.20	TOPSOIL Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion.
TP 11-03 (113.87m)	0.00 – 0.20 0.20 – 0.30 0.30	TOPSOIL Red brown fine SAND, some silt Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion. Sample — Depth (m) 1 — 0.20 – 0.30
TP 11-04 (113.19m)	0.00 – 0.50 0.50 – 2.40 2.40	TOPSOIL/PEAT Grey sandy silt and clayey silt, some gravel with occasional cobbles (GLACIAL TILL). w_n = 19% Refusal on LIMESTONE BEDROCK Note: Groundwater seepage at 0.40 metres depth below ground surface.

TABLE 1 (Continued)

RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 11-05 (113.94m)	0.00 – 0.80 0.80 – 1.00 1.00 – 2.40 2.40	TOPSOIL/PEAT Light grey brown CLAYEY SILT with shells (MARL) w_n = 130% Grey sandy silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Groundwater seepage at 0.80 metres depth below ground surface. <u>Sample</u> <u>Depth (m)</u> 1 0.90
TP 11-06 (116.25m)	0.00 – 0.20 0.20	TOPSOIL Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion.
TP 11-07 (113.37m)	0.00 – 0.20 0.20 – 0.90 0.90	TOPSOIL Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion.
TP 11-08 (112.59m)	0.00 – 0.15 0.15 – 1.20 1.20	TOPSOIL Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Groundwater seepage at 1.20 metres depth below ground surface.

TABLE 1 (Continued)

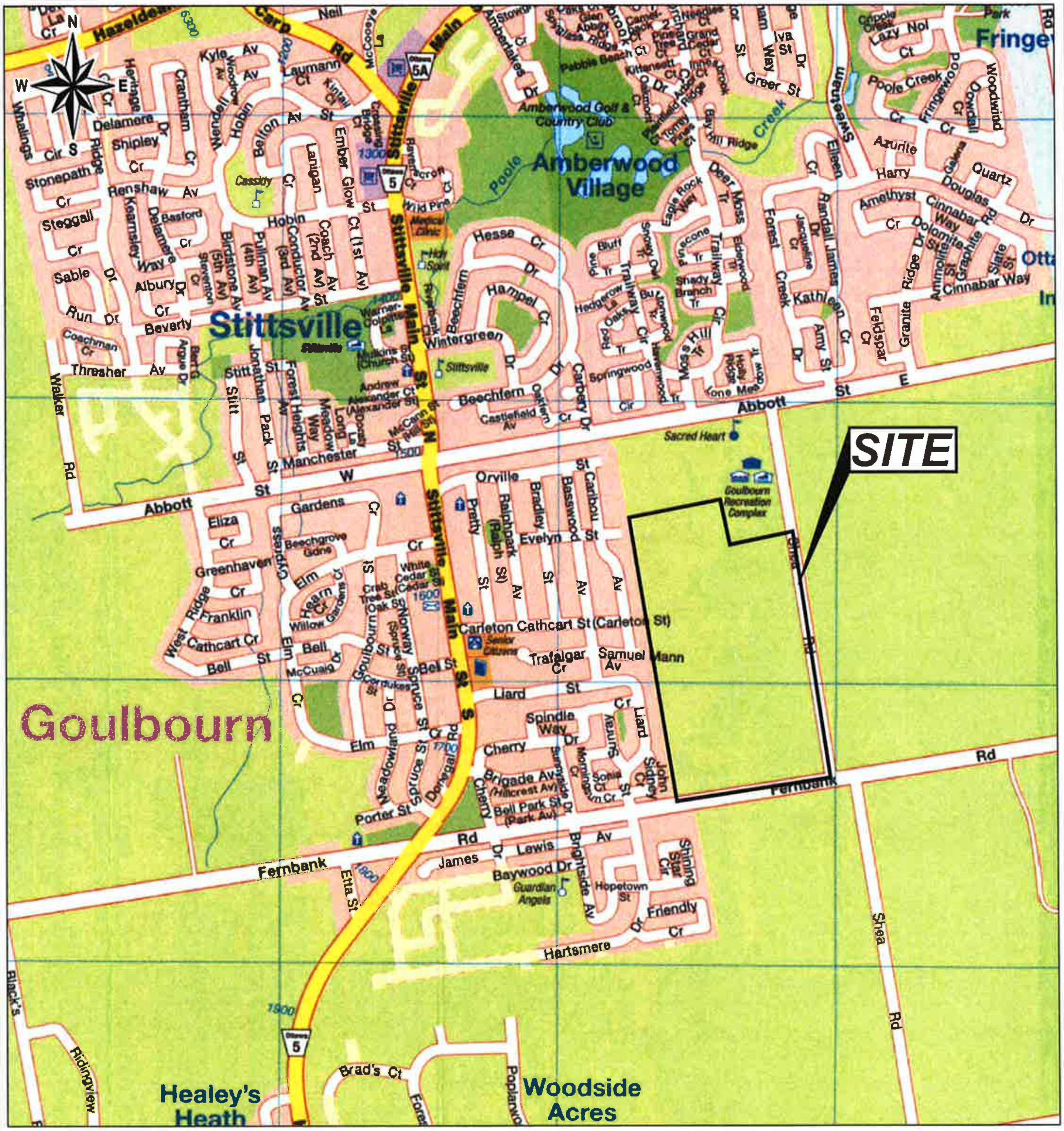
RECORD OF TEST PITS

TEST PIT NUMBER (ELEVATION)	DEPTH (METRES)	DESCRIPTION
TP 11-09 (112.74m)	0.00 – 0.50 0.50 – 0.90 0.90 – 2.90 2.90	TOPSOIL/PEAT Light grey brown CLAYEY SILT with shells (MARL) Grey sandy silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Groundwater encountered at 0.50 metres depth below the existing ground surface.
TP 11-10 (110.74m)	0.00 – 0.15 0.15 – 0.90 0.90	TOPSOIL Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion
TP 11-11 (112.39m)	0.00 – 0.10 0.10 – 0.40 0.40 – 2.60 2.60	TOPSOIL Red brown fine SAND, some silt Brown sandy silt and clayey silt, some gravel with cobbles and boulders (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion <u>Sample</u> <u>Depth (m)</u> 1 0.10 – 0.40
TP 11-12 (112.86m)	0.00 – 0.20 0.20 – 0.50 0.50	TOPSOIL Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion <u>Sample</u> <u>Depth (m)</u> 1 0.30 – 0.50

TABLE 1 (Continued)

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (metres)	Description
TP 11-13 (113.34m)	0.00 – 0.20 0.20 – 0.80 0.80	TOPSOIL Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Groundwater seepage at 0.80 metres depth below ground surface.
TP 11-14 (112.89m)	0.00 – 0.20 0.20 – 0.70 0.70	TOPSOIL Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion.
TP 11-15 (112.01m)	0.00 – 0.18 0.18 – 0.90 0.90	TOPSOIL Grey sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Concentrated water inflow observed from bedrock surface on floor of test pit. Possible former borehole. Possible artesian condition in bedrock.
TP 11-16 (110.25m)	0.00 – 1.00 1.00	Grey sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Test Pit dry upon completion.
TP 11-17	N/A	Not excavated due to test pit location being inaccessible
TP 11-18 (113.23m)	0.00 – 0.50 0.50 – 1.30 1.30	TOPSOIL/PEAT Brown sandy silt and clayey silt, some gravel with cobbles (GLACIAL TILL) Refusal on LIMESTONE BEDROCK Note: Groundwater seepage at 0.50 metres depth below ground surface.

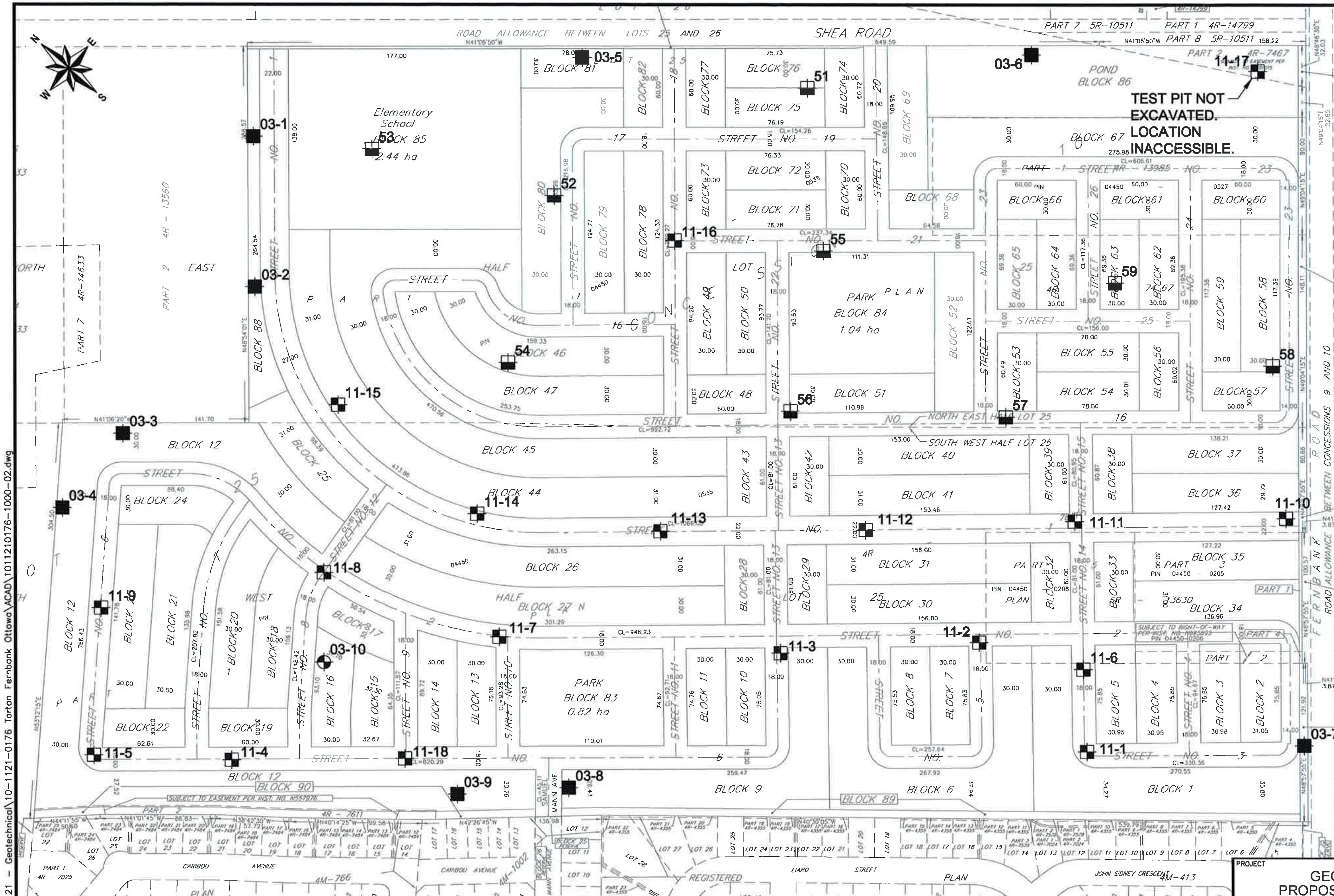


NOTE
THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0176

Golder Associates
Ottawa, Ontario

SCALE	1:20,000	TITLE	KEY PLAN
DATE	30 MAY '11		
DESIGN	J.C.		
CADD	A.B.D.		
FILE No. 1011210176-1000-01.dwg	CHECK		
PROJECT No. 10-1121-0176 REV. 0	REVIEW MIC	GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL DEVELOPMENT	FIGURE 1

FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0176 Tartan Fernbank Ottawa\ACAD\1011210176-1000-01.dwg



LEGEND

- [test pit symbol] APPROXIMATE TEST PIT LOCATION, CURRENT INVESTIGATION
- [filled square] APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD. REPORT NO. 03-1120-028
- [augerhole symbol] APPROXIMATE AUGERHOLE LOCATION, PREVIOUS INVESTIGATION BY GOLDER ASSOCIATES LTD. REPORT NO. 03-1120-028
- [augerhole symbol] APPROXIMATE AUGERHOLE LOCATION, PREVIOUS INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD.

REFERENCE

BASE PLAN PROVIDED IN ELECTRONIC FORMAT BY STANTEC GEOMATICS LTD.

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0176

60 0 60

SCALE 1:3,000 METRES

PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT

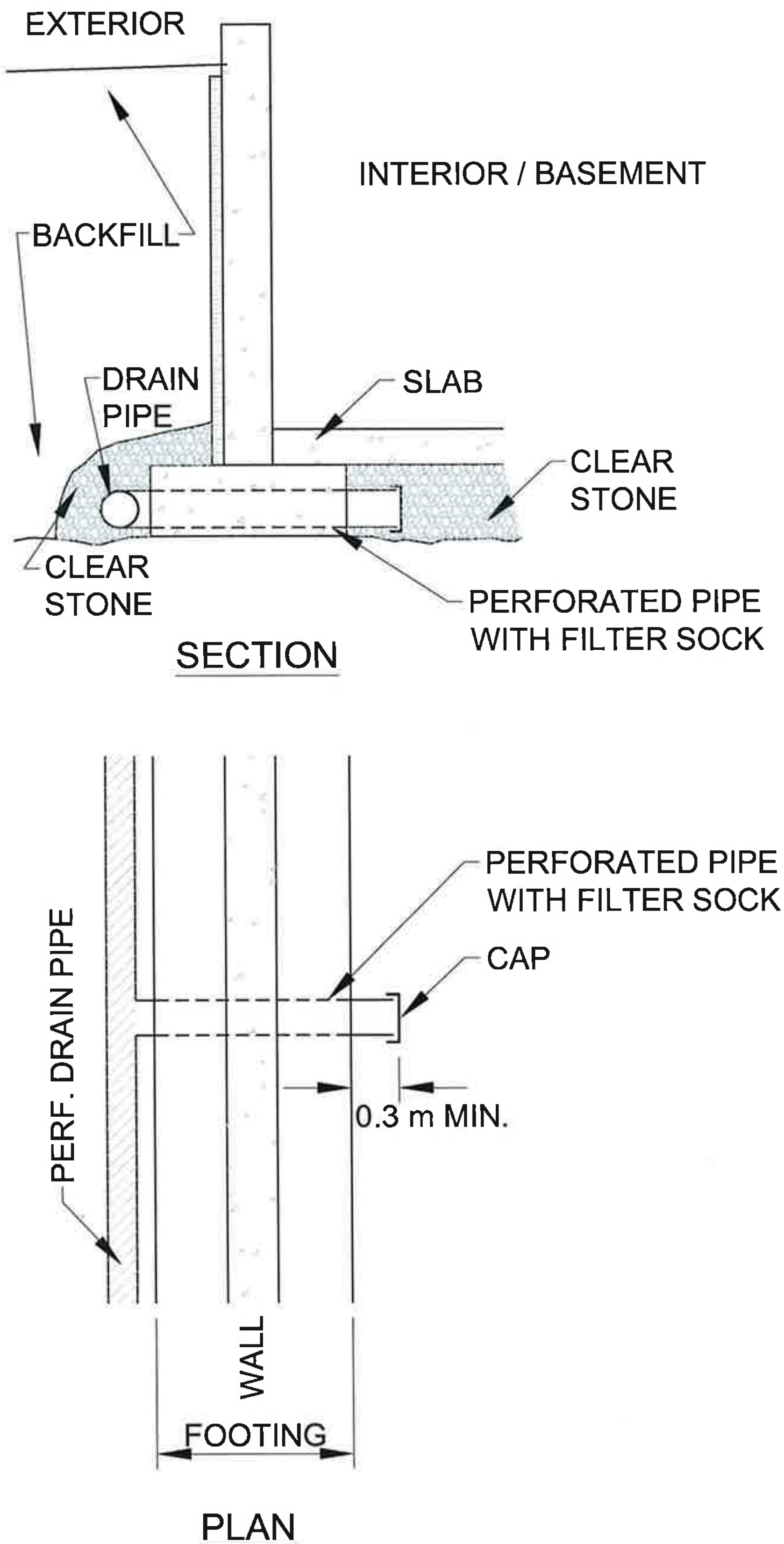
TITLE

SITE PLAN

Golder Associates
Ottawa, Ontario, Canada

PROJECT No. 10-1121-0176			FILE No. 1011210176-1000-02.dwg
DESIGN	J.C.	31 MAY'11	SCALE AS SHOWN REV.
CADD	A.B.D.	31 MAY'11	
CHECK			FIGURE 2
REVIEW	M.I.C.	13 Oct. 2011	

FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0176 Tartan Fernbank Ottawa\CAD\1011210176-1000-02.dwg



Golder Associates Ottawa, Ontario, Canada	SCALE	N.T.S.	TITLE	**HYDRAULIC CONNECTION BETWEEN UNDERSLAB FILL AND DRAINAGE SYSTEM**
	DATE	14 Oct. 2011		
	DESIGN	M.I.C.		
	CAD	M.L.F.		
FILE No. 1011210176-1000-03.dwg	CHECK			FIGURE 3
PROJECT No. 10-1121-0176 REV. 0	REVIEW	MK		

APPENDIX A

Record of Test Pits
Previous Investigation by Golder Associates Ltd.

TABLE 1
RECORD OF TEST HOLES

TEST HOLE NUMBER	DEPTH (METRES)	DESCRIPTION
TP 03-1	0.00 – 0.20	TOPSOIL
	0.20 – 0.70	Red brown fine SAND, some silt
	0.70 – 1.10	Grey brown sandy silt and clayey silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	1.10	Refusal on LIMESTONE BEDROCK
		Note: Test pit dry upon completion
TP 03-2	0.00 – 0.25	TOPSOIL
	0.25 – 0.60	Red brown fine SAND, some silt
	0.60 – 1.45	Grey sandy silt and clayey silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	1.45	Refusal on LIMESTONE BEDROCK
		Note: Groundwater seepage at 1.2 metres
TP 03-3	0.00 – 0.16	TOPSOIL
	0.16 – 0.45	Light brown fine SAND, some silt
	0.45 – 1.50	Brown sandy silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	1.50	Refusal on LIMESTONE BEDROCK
		Note: Test pit dry upon completion
TP 03-4	0.00 – 0.80	Black PEAT
	0.80 – 1.20	Light grey MARL
	1.20 – 1.80	Fine SAND, some silt
	1.80 – 2.40	Grey sandy silt, some gravel with cobbles and boulders (GLACIAL TILL)
	2.40	Refusal on LIMESTONE BEDROCK
		Note: Groundwater seepage at 1.0 metres

TABLE 1 (cont'd)
RECORD OF TEST HOLES

TEST HOLE NUMBER	DEPTH (METRES)	DESCRIPTION
TP 03-5	0.00 – 0.16	TOPSOIL
	0.16 – 0.45	Light brown fine SAND, some silt
	0.45 – 1.46	Brown sandy silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	1.46	Refusal on LIMESTONE BEDROCK
		Note: Test pit dry upon completion
TP 03-6	0.00 – 0.50	Brown silty sand, trace organic material, with cobbles and boulders (FILL)
	0.50 – 0.70	TOPSOIL
	0.70 – 1.10	Red brown fine SAND, some silt
	1.10 – 3.30	Grey sandy silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	3.30	End of test pit
		Note: Groundwater seepage at 2.3 metres
TP 03-7	0.00 – 0.16	TOPSOIL
	0.16 – 0.55	Red brown fine SAND, some silt
	0.55 – 2.36	Grey sandy silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	2.36	Refusal on LIMESTONE BEDROCK
		Note: Test pit dry upon completion
TP 03-8	0.00 – 0.14	TOPSOIL
	0.14 – 0.38	Red brown fine SAND, trace gravel
	0.38 – 1.75	Grey sandy silt, some gravel, with cobbles and boulders (GLACIAL TILL)
	1.75	Refusal on LIMESTONE BEDROCK
		Note: Test pit dry upon completion

TABLE 1 (cont'd)
RECORD OF TEST HOLES

TEST HOLE NUMBER	DEPTH (METRES)	DESCRIPTION
TP 03-9	0.00 – 0.30	Organic material with gravel and cobbles (FILL)
	0.30 – 0.70	Black PEAT
	0.70 – 0.90	Light grey-brown CLAYEY SILT with shells (MARL)
	0.90 – 1.70	Grey SANDY SILT
	1.70 – 1.92	Grey sandy silt, some gravel with cobbles and boulders (GLACIAL TILL)
	1.92	Refusal on LIMESTONE BEDROCK
		Note: Groundwater seepage at 1.1 metres
AH 03-10	0.00 – 0.50	Black PEAT
	0.50 – 0.60	Grey sandy silt and clayey silt, some gravel (GLACIAL TILL)
	0.60	Refusal to augering
		Note: Groundwater seepage at 0.2 metres

APPENDIX B

Record of Test Pits
Previous Investigation by Houle Chevrier Engineering Ltd.

PROJECT 06-384

LOCATION: See Site Plan, Figure 5

DATE OF EXCAVATION: December 12, 2006

RECORD OF TEST PIT 51

SHEET 1 OF 1

DATUM: Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp —O W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Brown fine to medium SAND		0.30					
1				1				
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		1.10					Native Backfill
2	Practical refusal to excavating on possible bedrock End of test pit		2.00					Groundwater inflow observed at 1.80 metres below ground surface on December 12, 2006 Groundwater level in standpipe at 1.47 metres below ground surface on January 4, 2007.
3								
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DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: [signature]

TESTPIT RECORD 06-384 TESTPIT LOGS.GPJ MHECL.GDT 5/28/07

PROJECT 06-384

LOCATION See Site Plan, Figure 5

DATE OF EXCAVATION December 12, 2006

RECORD OF TEST PIT 52

SHEET 1 OF 1

DATUM Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp W Wl 20 40 60 80	ADDITIONAL LAB TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.20					
1								
	Practical refusal to excavating on boulders End of test pit		1.50					No groundwater inflow observed on completion of excavating on December 12, 2006.
2								
3								
4								
5								
6								
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10								

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

PROJECT 06-384

RECORD OF TEST PIT 53

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 5

DATUM: Not Applicable

DATE OF EXCAVATION: December 12, 2006

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp —o W— Wl (20 40 60 80)	ADDITIONAL LAB TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Very stiff to stiff grey brown SILTY CLAY (weathered crust)		0.30					
1	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.70					Native Backfill
2	Practical refusal to excavating on boulders End of test pit Notes: 1. Substantial ground water inflow observed. Test pit filled within a few minutes after excavation.		1.80					Groundwater inflow observed at 0.70 metres below ground surface on December 12, 2006 Groundwater level in standpipe at 0.44 metres below ground surface on January 4, 2007.
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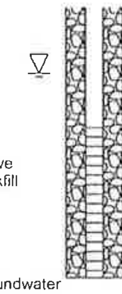
9								
10								



DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

RECORD OF TEST PIT 54

PROJECT 06-384
LOCATION See Site Plan, Figure 5
DATE OF EXCAVATION: December 12, 2006

SHEET 1 OF 1
DATUM Not Applicable
TYPE OF EXCAVATOR Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural V - + Remoulded V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.20					
1	Practical refusal to excavating on bedrock End of test pit		0.85					No groundwater inflow observed on completion of excavating on December 12, 2006.
2								
3								
4								
5								
6								
7								
8								
9								
10								

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG
CHECKED: AC

RECORD OF TEST PIT 55

PROJECT 06-384

LOCATION See Site Plan, Figure 5

DATE OF EXCAVATION December 12, 2006

SHEET 1 OF 1

DATUM Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp —o W— Wl (20 40 60 80)	ADDITIONAL LAB TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.30					
1	Practical refusal to excavating on bedrock End of test pit		0.80					No groundwater inflow observed on completion of excavating on December 12, 2006.
2								
3								
4								
5								
6								
7								
8								
9								
10								

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

TESTPIT_RECORD 06-384 TESTPIT LOGS.GPJ MHECL.GDT 5/28/07

PROJECT 06-384

LOCATION See Site Plan, Figure 5

DATE OF EXCAVATION December 12, 2006

RECORD OF TEST PIT 56

DATUM: Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.20					
	Practical refusal to excavating on bedrock End of test pit		0.75					No groundwater inflow observed on completion of excavating on December 12, 2006
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

TESTPIT RECORD 06-384 TESTPIT LOGS.GPJ MHECL.GDT 5/28/07

RECORD OF TEST PIT 57

PROJECT: 06-384
LOCATION: See Site Plan, Figure 5
DATE OF EXCAVATION: December 12, 2006

SHEET 1 OF 1
DATUM: Not Applicable
TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp —O W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Brown SILTY SAND, some gravel		0.30					
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.50					
1								
2	Practical refusal to excavating on boulders End of test pit		1.90					No groundwater inflow observed on completion of excavating on December 12, 2006.
3								
4								
5								
6								
7								
8								
9								
10								

DEPTH SCALE
1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG
CHECKED: AC

PROJECT 06-384

LOCATION See Site Plan Figure 5

DATE OF EXCAVATION December 12, 2006

RECORD OF TEST PIT 58

SHEET 1 OF 1

DATUM Not Applicable

TYPE OF EXCAVATOR Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp —o W— Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Fine to medium brown SAND, some silt		0.20					
1								∇
	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		1.20					Native Backfill
2								
3	Practical refusal to excavating on boulders End of test pit		2.90					Groundwater inflow observed at 2.70 metres below ground surface on December 12, 2006
4								
5								Groundwater level in standpipe at 0.85 metres below ground surface on January 4, 2007
6								
7								
8								
9								
10								

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED: EG

CHECKED: AC

RECORD OF TEST PIT 59

PROJECT 06-384

LOCATION See Site Plan, Figure 5

DATE OF EXCAVATION: December 12, 2006

SHEET 1 OF 1

DATUM Not Applicable

TYPE OF EXCAVATOR: Hydraulic Excavator

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural V - + Remoulded V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp W Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface							
	TOPSOIL							
	Fine to medium brown SAND		0.20	1				
1	Grey brown silty sand, some clay, gravel, cobbles and boulders (GLACIAL TILL)		0.80					
2								
3								
4	Practical refusal to excavating on boulders End of test pit		3.80					No groundwater inflow observed on completion of excavating on December 12, 2006.
5								
6								
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8								
9								
10								

DEPTH SCALE

1 to 50

Houle Chevrier Engineering Ltd.

LOGGED EG

CHECKED AC

APPENDIX C

Results of Basic Chemical Analysis
Exova Laboratories Report Number 1111177

EXOVA ACCUTEST

REPORT OF ANALYSIS

Exova Accutest

Client: **Golder Associates Ltd. (Ottawa)**
32 Steacie Drive
Kanata, ON
K2K 2A9
Attention: **Ms. Jacquelline Cormier**

Report Number: 1111177
Date: 2011-06-03
Date Submitted: 2011-05-27

Project: 10-1121-0176

P.O. Number:
Matrix: Soil

Chain of Custody Number: 127500

		LAB ID:	883713	883714				GUIDELINE		
		Sample Date:	2011-05-12	2011-05-12						
		Sample ID:	TP 11- 4	TP 11 - 12						
PARAMETER	**UNITS**	**MRL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	%	0.002	0.002	<0.002						
Electrical Conductivity	mS/cm	0.05	0.27	0.11						
pH			7.7	8.1						
Resistivity	ohm-cm	1	3704	9090						
Sulphate	%	0.01	0.02	<0.01						

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL:
Lorna Wilson
AG/Inorganic Lab Supervisor

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
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Golder Associates

Appendix E DRAWINGS