South Nepean Town Centre (SNTC) – Functional Servicing Report

Project #160401085

Prepared for:
Caivan Development Corporation

Prepared by:
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Sign-off Sheet

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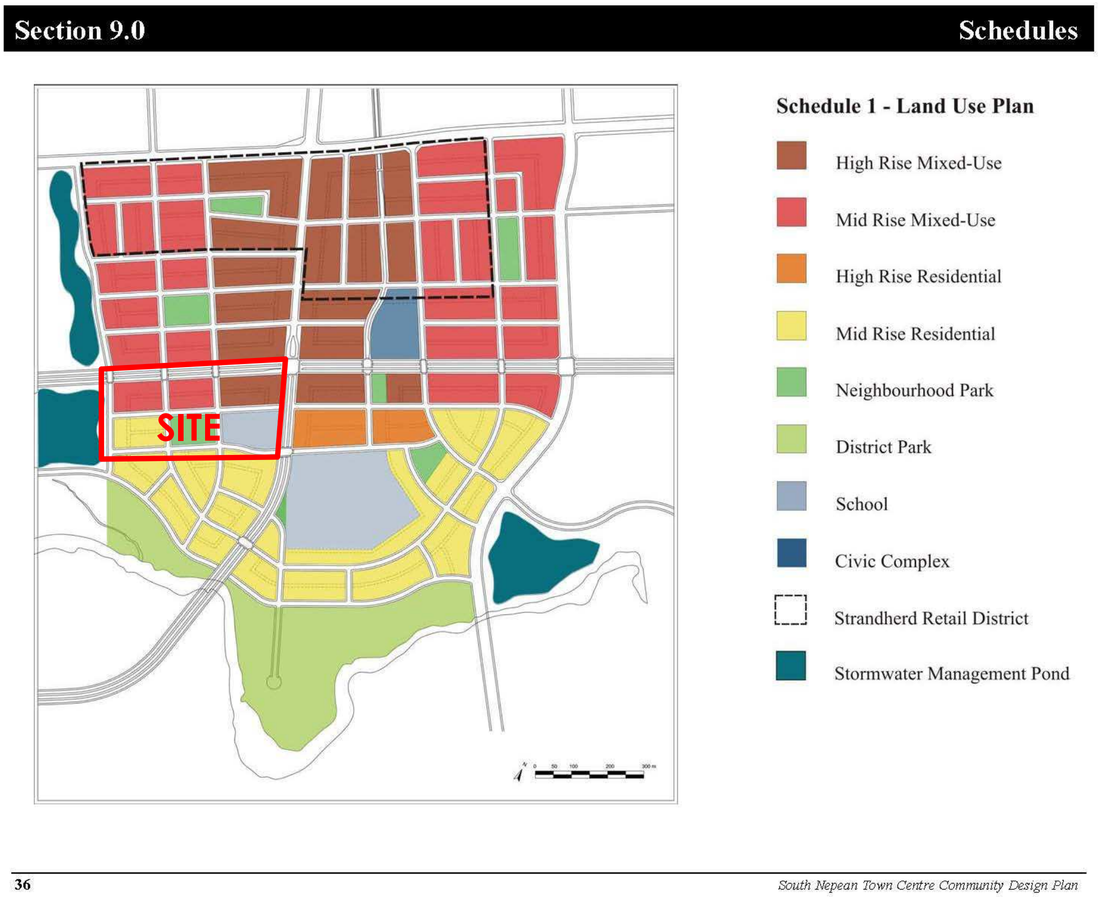
1.0 INTRODUCTION

Caivan Development Corporation has commissioned Stantec Consulting Ltd. to prepare the following Functional Servicing Report for the South Nepean Town Centre (SNTC) development. The subject property is located southwest of the intersection of the future extension of Chapman Mills Drive and Greenbank Road within the South Nepean Town Centre CDP area. The property is currently zoned Development Reserve (DR) and is bordered by Greenbank Road to the east, the Kennedy-Burnett Stormwater Management Pond to the west, a future extension of Chapman Mills Drive to the north, and residential development to the south currently undergoing development review with the City of Ottawa. **Figure 1** identifies the land uses for the SNTC lands as outlined in the South Nepean Town Centre Community Design Plan (CDP). The location of the Caivan development lands in relation to the overall SNTC community is identified in red on the figure. The proposed development comprises 8 development blocks on approximately 12.6ha of land and is to contain a mixture of mid-rise residential to high-rise residential uses, a potential school block, as well as a neighbourhood park.

Figure 1: Approximate Location of South Nepean Town Centre Draft Plan Area



1.1 OBJECTIVE

The intent of this report is to build on the servicing principles outlined in the South Nepean Town Centre CDP and later updated within Fotenn's revised South Nepean Town Centre Planning Rationale for Official Plan Amendment to create a servicing strategy specific to the subject property. The report will establish criteria for future detailed design of the subdivision in accordance with the associated background studies, City of Ottawa Guidelines, and all other relevant regulations.

2.0 BACKGROUND

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

- South Nepean Town Centre Community Design Plan, City of Ottawa Planning and Growth Management Department, July, 2006.
- South Nepean Town Centre Official Plan Amendment, Fotenn Consultants Inc., August, 2014.
- SNTC Lands Assessment of Adequacy of Services for Official Plan Amendment, Stantec Consulting Ltd., January 2014.
- Storm Servicing Options Evaluation – Future Development Lands Adjacent to Kennedy-Burnett SWM Facility, Novatech Engineering Consultants Ltd., October 2015.
- Burnett Lands 3370 Greenbank Road, Site Serviceability and Stormwater Management Report, Novatech Engineering Consultants Ltd., May 2018.
- South Nepean Collector Sewer Alignment & Finalization Report – Phase 2, Novatech Engineering Consultants Ltd., June 2014.
- Draft Kennedy-Burnett Potable Water Master Servicing Study, Stantec Consulting Ltd., March 2014.
- Geotechnical Investigation – Proposed Residential Development – 3288 Greenbank Road, Paterson Group Inc., March 6, 2019.
- City of Ottawa Sewer Design Guidelines, 2nd Ed., City of Ottawa, October 2012
- City of Ottawa Design Guidelines – Water Distribution, Infrastructure Services Department, City of Ottawa, First Edition, July 2010

3.0 POTABLE WATER ANALYSIS

3.1 BACKGROUND

The proposed development is currently located within Zone BARR of the City of Ottawa's water distribution system. This zone is fed by the Barrhaven Pump Station and Barrhaven Reservoir Pump Station, with the Moodie elevated storage tank providing balancing storage for peak flows and demands. The development is located within the future Zone 3C pressure zone, to be completed by the City of Ottawa in the future.

A 400mm diameter watermain exists along Jockvale Road south of the intersection with Greenbank Road, and a 300mm diameter main exists along Jockvale immediately south of Strandherd Drive. Development of the Burnett Lands to the south will require a 300mm watermain connection from the 400mm main within Jockvale Road to the south running up to the shared road at the south of the subject development (Street B), and continuing northwards along Greenbank Road to the existing Jockvale intersection to allow for a looped connection to the area (see **Appendix A** for details). The proposed development will continue this servicing scheme by extending the 300mm main along the realigned Jockvale Road to the northern extent of the property at the future Chapman Mills Road, with intent to connect to the existing main at Strandherd Drive in the future (by others).

3.2 PROPOSED WATERMAIN SIZING AND LAYOUT

The proposed watermain alignment and sizing for this development is shown on **Drawing WTR-1** with 203mm diameter and 305mm diameter piping to conform to the layout shown in the Kennedy-Burnett Potable Water Master Servicing Study. It should be noted that the pipe layout and sizing for the internal mains within the development is preliminary and is to be verified upon detailed hydraulic analysis for the development area.

3.2.1 Ground Elevations

The proposed ground elevations of the development range from approximately 95.0m to 93.5m. Preliminary grading and elevations have been determined for the site and are included on **Drawing GP-1**.

3.2.2 Water Demand

The current draft plan for the development calls for a total of 8 residential blocks, of which 6 are intended for residential development. The six residential blocks lie within CDP areas noted as mid-rise residential and mid-rise mixed-use areas (2-4 and 4-6 storeys), as well as high density mixed-use areas. Net unit density targets have been applied to each block to develop estimated domestic demand rates for the region in consideration of an average townhouse unit population density of 2.7ppu and average apartment population density of 1.8ppu.

Water demands for the development were estimated using the City of Ottawa's Water Distribution Design Guidelines. For residential developments, the average day (AVDY) per capita water demand is 350 L/cap/d. For maximum day (MXDY) demand, AVDY was multiplied by a factor of 2.5 and for peak hour (PKHR) demand, MXDY was multiplied by a factor of 2.2. For commercial and institutional use, the AVDY is based on the area of land use and is shown in the following tables. For MXDY demand, AVDY was multiplied by a factor of 1.5 and for PKHR demand, MXDY was multiplied by a factor of 1.8. The calculated domestic water consumption is represented in **Table 1** and **Table 2** for the SNTC development:

Table 1: Residential Water Demands

Area ID	Units	Person/Unit	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Block 1	220	2.7	594	2.41	6.02	13.23
Block 2	270	2.7	729	2.95	7.38	16.24
Block 3	275	1.8	495	2.01	5.01	11.03
Block 4	63	2.7	170	0.69	1.72	3.79
Block 5	81	2.7	219	0.89	2.21	4.87
Block 6	171	2.7	462	1.87	4.68	10.29
		Total	2669	10.81	27.02	59.45

Table 2: Institutional Water Demands

Area (ha)	Demand	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
1.18	25000	0.03	0.05	0.09

3.2.3 Connection to Existing Infrastructure

Potable water supply will be connected to the watermain stubs located along Street B provided by the adjacent residential development to the south at proposed streets A and C, as well as Jockvale Road, and future connections to Chapman Mills Drive. **Drawing WTR-1** shows the location of the connection points to the existing watermains.

A minimum of two connections to the surrounding 300mm and 200mm watermains will be provided as development proceeds, with no more than 75 units serviced on a temporary basis and no more than 49 units permanently serviced with a single feed as per City guidelines. Development blocks are all anticipated to require 2 service connections per block to meet this requirement.

3.3 HYDRAULIC ASSESSMENT

A hydraulic model was built by Stantec for the Kennedy-Burnett Potable Water Master Servicing Study in June 2014. Burnett lands to the south have been modeled in 2018 by Novatech under current development conditions in consideration of City supplied boundary conditions at the mains along Greenbank Road and at Jockvale. Both models assessed the anticipated pressures in the vicinity at buildout to meet minimum servicing requirements (basic day and peak hour demands). A fire flow analysis was also performed under maximum day conditions (see **Appendix A** for excerpts).

3.3.1 Allowable Pressures

The City of Ottawa Water Distribution Design Guidelines state that the desired range of system pressures under normal demand conditions (i.e. basic day, maximum day, and peak hour) should be in the range of 350 to 552 kPa (50 to 80 psi) and no less than 275kPa (40 psi) at the ground elevation in the streets (i.e. at hydrant level). The maximum pressure at any point in the distribution system is to be no higher than 552kPa (80 psi). As per the Ontario Building Code & Guide for Plumbing, if pressures greater than 552kPa (80 psi) are anticipated, pressure relief measures are required. Under emergency fire flow conditions, the minimum pressure in the distribution system is allowed to drop to 138kPa (20 psi).

3.3.2 Fire Flow

The Master Servicing Study model assessed a maximum fire flow of 10,000 L/min for the area (Option 2A) based on Fire Underwriters Survey (FUS) requirements, with fire flows greater than 15,000 L/min achievable once serviced through lands west of the Kennedy-Burnett SWM Pond.

It should be noted that as per the City's technical bulletin which addresses fire flow (ISDTB-2014-02) for traditional side-by-side towns and row houses constructed in accordance with the OBC, the fire flow requirement shall be capped at 10,000 L/min. As such, the proposed watermain layout in conformance with the Potable Water MSS is expected to perform adequately for the development.

Fire flow assessment will be required at the subdivision approval phase in which local watermains are checked for their ability to provide the objective FUS fire flows, which in turn will be determined based on final unit layouts. Smaller, local internal watermains will need to be assessed and verified as development planning proceeds.

3.4 POTABLE WATER SUMMARY

The proposed piping alignment and sizing is capable of achieving the level of service in the proposed subdivision. Based on the hydraulic analysis created at the Master Servicing level, the following conclusions were made:

- The proposed water distribution system is recommended to include a combination of 305mm and 203mm diameter pipes;
- During peak hour conditions, the proposed system is capable of operating above the minimum pressure objective of 276kPa (40psi);
- During fire conditions, the proposed system is capable of providing sufficient fire flows (10,000L/min and above) while maintaining a residual pressure of 138kPa (20 psi) throughout the development. Sizing of internal mains on local streets will be coordinated to ensure a minimum fire flow of 10,000 L/min will be achieved.

4.0 WASTEWATER SERVICING

4.1 BACKGROUND

As indicated in Novatech's *South Nepean Collector: Phase 2 – Hydraulics Review / Assessment*, wastewater servicing for the development is to be conveyed to the South Nepean Collector via an internal network of free flow gravity sewers. The report identifies the location of the trunk sanitary sewer roughly bisecting the subject property, as well as estimated population densities and residential peak flow rates for the development. The Sanitary Drainage Plan is included in **Appendix B**. The *Hydraulics Review / Assessment* technical memorandum also included an assessment of the residual capacity within trunk based on a full buildout for Phases 2 and 3 of the SNC tributary area.

4.2 DESIGN CRITERIA

As outlined in the City's Sewer Design Guidelines, the following design parameters were used to calculate estimated wastewater flow rates and to preliminarily size on-site sanitary sewers:

- Minimum Full Flow Velocity – 0.6 m/s
- Maximum Full Flow Velocity – 3.0 m/s
- Manning's roughness coefficient for all smooth walled pipes – 0.013
- Single Family Persons per unit – 3.4
- Townhouse Persons per unit – 2.7
- Extraneous Flow Allowance – 0.33 L/s/ha
- Residential Average Flows – 280 L/cap/day
- Institutional Flows – 28,000 L/ha/day
- Manhole Spacing – 120 m
- Minimum Cover – 2.5m

In addition, a residential peak factor based on Harmon's Equation in conjunction with a correction factor of 0.8 was used to determine the peak design flows. Institutional areas were assigned a peaking factor of 1.5 per Ottawa's Sewer Design Guidelines.

Per the CDP and revised during the August 2014 Official Plan Amendment, the contributing area was assessed at a residential density of 100 units/ha for mid-rise 2-4 storey residential areas (Blocks 4, 5 and 6), 200 units/ha for mid-rise 4-6 storey residential areas (Blocks 1 and 2), and 250 units/ha for high-rise residential areas (Block 3). A unit population density of 2.7 persons/unit mid-rise residential and 1.8 persons/unit for high-rise residential areas was applied for a total population of 2,669 (see **Drawing SAN-1**).

4.3 PROPOSED SERVICING

The development will be serviced by a network of gravity sewers which will direct wastewater flows to the centrally located trunk sewer within the proposed Jockvale Road as directed by the CDP. The proposed sanitary sewer design sheet and associated Sanitary Drainage Area Plan can be found in **Appendix B & Appendix E** respectively. The proposed sanitary sewer design indicates a single connection point to the recently constructed trunk sewer. A drop structure will be required to tie in at an existing manhole to avoid direct connection to the trunk concrete pressure pipe.

Peak flows for the development are summarized in **Table 3** below, and have been coordinated with the latest available plans for residential developments to the south including the shared access Street B. Previously allocated flows for the available connection points are noted in **Table 4**. It is of note that the assumed residential unit density and population for the area based on the SNTC CDP is substantially higher than that assessed within the *Hydraulics Review / Assessment*. Given recent changes in peak design and operational design flow parameters within the City Sewer Design Guidelines and ample available capacity within the trunk sewer (see **Appendix B**), the increase in expected population beyond that shown in the *Hydraulics Review / Assessment* is not anticipated to have deleterious impact on downstream receiving sewers (Free flowing capacity of 900.5L/s, peak flow at buildout of 634.2L/s).

Table 3: Wastewater Connection to Jockvale Road

MH ID	Total Area (ha)	Residential Units	Residential Population	Institutional Area (ha)	Park Area (ha)	Total Flow (L/s)
23	8.58	1080	2669	1.18	1.00	29.8

Table 4: Wastewater Connection to Jockvale Road per SNC Phase 2 Hydraulics Review / Assessment Report

MH ID	Total Area (ha)	Residential Units	Residential Population	Institutional Area (ha)	Park Area (ha)	Total Flow (L/s)
80 (A7-D)	11.74	704	1902	0.00	0.00	20.8

5.0 STORMWATER MANAGEMENT

The proposed development encompasses approximately 12.3 ha of land at 59% imperviousness and comprises a school block, designated park land, a high density residential block, and a mix of rear lane town homes and back to back town homes.

Post development runoff from the development will be directed to a shared storm sewer within Street B of the future Claridge development to the south and ultimately to the Kennedy Burnett SWM facility outlet channel. **Drawing STM-1** shows the proposed site drainage areas, the drainage areas from the Claridge development tributary to the shared storm sewer as well as the overall major and minor system flow direction.

The storm drainage objective is to complete a conceptual stormwater management plan for the proposed development that meets all relevant design criteria.

5.1 BACKGROUND AND SWM CRITERIA

The SNTC lands are tributary to the Jock River and must adhere to the stormwater management (SWM) design criteria identified in the *Jock River Reach 1 Subwatershed Study (*lands north of the Jock River) as follows:

- Provide and Enhanced level of water quality treatment (80% TSS removal).
- No quantity control storage is required for flood control purposes.
- No erosion control storage is required to maintain pre-development in-stream erosion condition (for outlets discharging directly to the Jock River).
- All stormwater management facility outlets are to be designed to augment low flows to the extent possible.

As part of a required retrofit for the Kennedy-Burnett SWM Facility to conform to the Ministry of the Environment, Conservation and Parks (MECP) and City of Ottawa standards, the City of Ottawa retained Novatech Engineering to develop and evaluate various storm servicing options for future development south of Strandherd Drive. The results of this analysis are summarized in the report *Storm Servicing Options Evaluation – Future Development Lands Adjacent to Kennedy-Burnett SWM Facility* (Novatech, January 2017).

The preferred option consisted of expanding the Kennedy-Burnett SWM Facility to accommodate future development adjacent to the facility while minimizing the extent of submerged storm sewers and grade raise requirements. As a result, six (6) outfalls to the Jock River with upstream hydrodynamic separators (HDS) will be used to provide water quality treatment for the remaining areas.

The following summarizes the SWM criteria for the proposed site as per the recommendations of Novatech's 2017 Storm Servicing Options Evaluation Report.

- No quantity control is required for flood control purposes.
- Minor system peak flows from the proposed site will be captured and directed to an HDS and will ultimately discharge into the outlet channel for the Kennedy-Burnett SWM Facility.
- Major system peak flows from the proposed site will be directed to the Kennedy-Burnett SWM Facility in the ultimate condition, once the facility has been expanded. In the interim condition, if the proposed development is built before the K-B SWM facility expansion takes place which is tentatively scheduled for 2019-2020, a hydraulic analysis of the existing SWM facility will be required to ensure the water levels in the pond do not negatively impact upstream properties. Conversely, interim measures, in the form of a dry pond to store major system peak flows from the proposed site will be required.

Novatech has been retained by Claridge to complete the design of the development to the south and as such coordination of minor and major system peak flows from the proposed site to Street B is being coordinated with Novatech. Given the street layout of the proposed site as well as grading restrictions, major flows from the proposed site will be directed to Street B within Claridge's development to the south. It is anticipated that 100-year major system overflows from the proposed development to Street B will be negligible if any. However, further coordination will be required with Novatech to ensure conservative assumptions are made.

The following summarizes the SWM criteria and constraints that will govern the detailed design of the proposed development as per the latest City of Ottawa Storm Sewer Guidelines.

- Design using the dual drainage principle.
- Total maximum depth of flow under static and dynamic conditions shall be less than 0.35 m.
- Rear-yard storage is not to be included in calculations.
- 100-year hydraulic grade line (HGL) to be a minimum 0.30 m below lowest building underside of footing elevation.
- 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.
- Assess impact of 2-year storm, 5-year storm, and the worst case 100-year storm events, and climate change scenarios with a 20% increase of rainfall intensity, on the major & minor drainage system.
- Building openings to be above the 100-year water level.
- Provide adequate emergency overflow conveyance off-site.

5.2 POST DEVELOPMENT CONCEPTUAL MODELING RATIONALE

The site is to be designed using the "dual drainage" principle, whereby the minor (pipe) system is designed to convey the peak rate of runoff from the 2-year/5-year design storm and runoff from larger events is conveyed by both minor (pipe) and major (overland) channels, such as roadways and walkways, safely off site without impacting proposed or downstream properties.

In keeping with the 2-year inlet restriction for local streets and 5-year inlet restriction for collector streets (Jockvale Road), inlet control devices (ICDs) or orifice plates will be specified during the detailed design stage for all street and rear yard catchbasins to limit the inflow to the minor system. Restricted inlet rates to the sewer are necessary to prevent the hydraulic grade line from surcharging storm sewers into basements during major storms.

Drawing STM-1 outlines the proposed storm sewer alignment and drainage divides. The major system flows generated from larger events will be safely conveyed to Street B. It is anticipated that negligible 100-year major system overflows will be generated from the areas within the proposed site directed towards the streets within Claridge's development to the south. The majority of the major flows from the proposed site will be directed to the downstream end of Street B that discharges directly into the Kennedy Burnett SWM Facility outlet channel and does not impact the subdivision to the south.

The proposed park, high density residential and school areas (L203C, L205B & L205D) have been assumed to provide on-site storage for storms up to the 100-year storm in order to minimize major system overflows to the Claridge development to the south. All other areas have been assumed to provide 50 m^3/ha of surface storage.

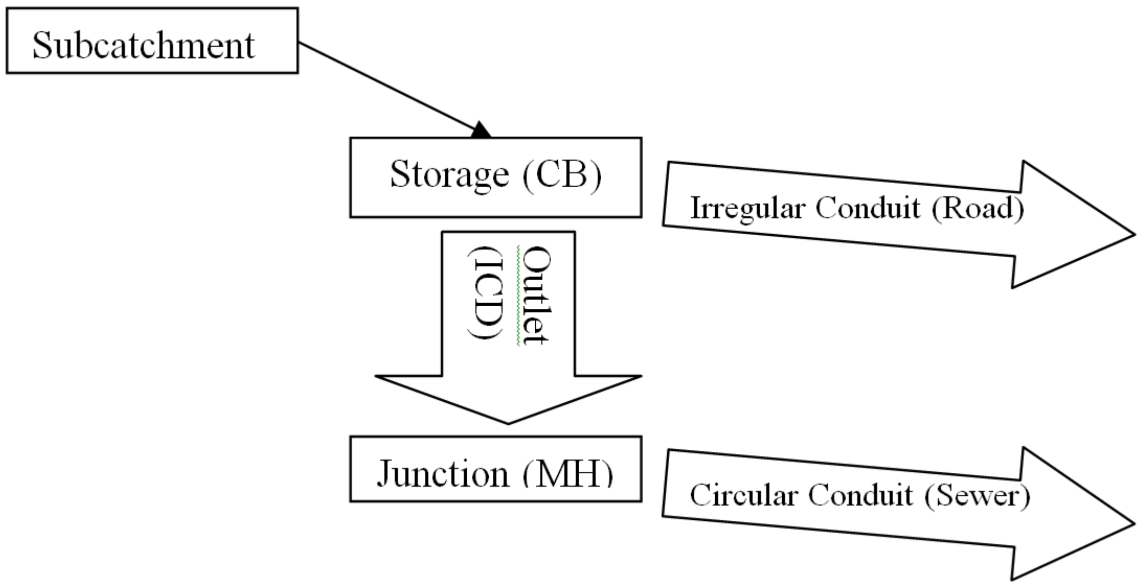
The minor system from the proposed subdivision will be conveyed through the shared storm sewer located within Street B of Claridge's Development to the south which will outlet into the Kennedy Burnett SWM Facility outlet channel (see **Drawing STM-1** for location). Quality control of runoff will be provided through a hydrodynamic separator / Oil-Grit Separator (HDS) designed as part of Claridge's Development to provide 'Enhanced' level of treatment (80% TSS Removal).

Conceptual hydrologic and hydraulic modeling of the proposed site 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. 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.

5.2.1 SWMM Dual Drainage Methodology

The proposed development is modeled in one modeling program as a dual conduit system (see **Figure 2**), 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 2 : 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 1.38 m below the top of the CB and a flow depth of 0.30 m has been assumed on grassed swale segments and of 0.35 m on road segments.

Storage curves in PCSWMM are required to be input as depth-area curves, as such an equivalent area was calculated at a depth of 1.73 m to obtained the surface storage available. All storage was assumed to occur between the top of the CB (1.38 m head) and a 0.35 m depth (1.73 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. Capture curves were defined for each catchment to restrict outlet link flows to the 2-year and 5-year rate for local streets, and collector roads (Jockvale Road) respectively.

5.2.2 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. Other storms including the climate change storm will be used to assess the proposed minor and major systems during the detailed design stage.

5.2.3 Boundary Conditions

Static backwater elevations at the proposed storm outfall to the Kennedy Burnnett SWM Facility outlet channel were obtained from Novatech as 90.55 m for the 2-year storm, 90.92 m for the 5-year storm and 91.58 m for the 100-year storm.

5.2.4 Modeling Parameters

Table 5 presents the general subcatchment parameters used:

Table 5: 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 6 presents the individual parameters that vary for each of the conceptual subcatchments tributary to the storm outlet. Detailed parameter information for the areas within the Claridge Development were obtained from Novatech as shown in **Appendix C2**.

Table 6: Conceptual Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient	Subarea Routing	% Routed	Owner
C203A	0.45	226.0	3.0	64.3%	0.65	OUTLET	100	Cavian
L100A	0.12	80.0	3.0	45.7%	0.52	OUTLET	100	Claridge
L110A	0.15	148.0	3.0	61.4%	0.63	OUTLET	100	Claridge
L110B	0.03	21.0	3.0	38.6%	0.47	PERVIOUS	100	Claridge
L110C	0.11	47.0	3.0	41.4%	0.49	PERVIOUS	100	Claridge
L110D	0.18	101.0	3.0	71.4%	0.70	OUTLET	100	Claridge
L112A	0.20	155.0	3.0	68.6%	0.68	OUTLET	100	Claridge
L112B	0.20	65.0	3.0	31.4%	0.42	PERVIOUS	100	Claridge
L114A	0.23	154.0	3.0	75.7%	0.73	OUTLET	100	Claridge
L116A	0.14	141.0	3.0	67.1%	0.67	OUTLET	100	Claridge
L116B	0.09	36.0	3.0	31.4%	0.42	PERVIOUS	100	Claridge
L118A	0.14	129.0	3.0	65.7%	0.66	OUTLET	100	Claridge
L201A	0.45	226.0	3.0	64.3%	0.65	OUTLET	100	Cavian
L201B	1.71	384.0	3.0	71.4%	0.70	OUTLET	100	Cavian

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient	Subarea Routing	% Routed	Owner
L202A	0.24	134.0	3.0	64.3%	0.65	OUTLET	100	Cavian
L202B	1.10	248.0	3.0	71.4%	0.70	OUTLET	100	Cavian
L203B	0.81	182.0	3.0	71.4%	0.70	OUTLET	100	Cavian
L203C	1.00	225.0	3.0	14.3%	0.30	OUTLET	100	Cavian
L204A	0.28	156.0	3.0	64.3%	0.65	OUTLET	100	Cavian
L204B	1.35	303.0	3.0	71.4%	0.70	OUTLET	100	Cavian
L205A	0.45	226.0	3.0	64.3%	0.65	OUTLET	100	Cavian
L205B	1.10	247.0	3.0	78.6%	0.75	OUTLET	100	Cavian
L205C	0.63	142.0	3.0	71.4%	0.70	OUTLET	100	Cavian
L205D	1.12	253.0	3.0	71.4%	0.70	OUTLET	100	Cavian

1. The width parameter was estimated 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 lumped areas was defined as 225m/ha as per the City of Ottawa Sewer Design Guidelines.

Table 7 summarizes the storage node parameters used in the conceptual model. The major system for Claridge areas within Street B were modeled based on the proposed road design obtained from Novatech and as such, there is no storage assumed within Claridge areas.

Table 7: Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Surface Storage (m^3)	Owner
C203A-S	92.63	94.36	1.73	22	Cavian
CB69-70	91.52	93.46	1.94	0	Claridge
CB39-40	91.49	93.62	2.13	0	Claridge
CB41-42	91.75	93.79	2.04	0	Claridge
RYCB01	92.00	93.95	1.95	0	Claridge
RYCB03	91.50	93.80	2.30	0	Claridge
CB43-44	91.72	93.75	2.03	0	Claridge
RYCB07	91.87	94.06	2.19	0	Claridge
CB45-46	91.73	93.76	2.03	0	Claridge
CB47-48	91.79	93.82	2.03	0	Claridge
RYCB11	89.62	93.73	4.11	0	Claridge
CB49-50	91.97	94.02	2.05	0	Claridge
L201A-S	92.26	93.99	1.73	22	Cavian
L201B-S	92.14	94.22	2.08	85	Cavian
L202A-S	92.58	94.31	1.73	11	Cavian
L202B-S	92.43	94.51	2.08	55	Cavian
L203B-S	92.96	95.04	2.08	41	Cavian
L203C-S	92.41	94.39	1.98	151	Cavian
L204A-S	92.95	94.68	1.73	14	Cavian
L204B-S	93.10	95.18	2.08	68	Cavian
L205A-S	93.14	94.87	1.73	18	Cavian
L205B-S	92.99	95.07	2.08	160	Cavian
L205C-S	93.39	95.47	2.08	32	Cavian

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Surface Storage (m^3)	Owner
L205D-S	92.99	95.07	2.08	164	Cavian

1. The proposed school block (L205D), park area (L203C) and high density residential block (L205B) were assumed to provide on-site storage for all storms up to the 100-year storm. All other areas within the proposed site were assumed to provide 50 m^3/ha of on-site storage.

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 8 summarizes the outlet link maximum flow rates for the 100-year, 3hr Chicago storm event for areas within the proposed development.

Table 8: Conceptual Minor System Capture Rates

Outlet Name	Inlet Node	Outlet Node	Invert Elevation (m)	Level of Service	100-year Minor System Capture Rate (L/s)
C203A-O1	C203A-S	203	92.63	5-year	122.6
L201A-O1	L201A-S	201	92.26	2-year	91.9
L201B-O	L201B-S	201B	92.14	2-year	372.5
L202A-O1	L202A-S	202	92.58	2-year	48.2
L202B-O	L202B-S	202B	92.43	2-year	241.2
L203B-O	L203B-S	203B	92.96	2-year	176.2
L203C-O	L203C-S	203	92.41	2-year	141.1
L204A-O1	L204A-S	204	92.95	2-year	56.7
L204B-O	L204B-S	204B	93.10	2-year	293.5
L205A-O	L205A-S	205	93.14	2-year	91.6
L205B-O	L205B-S	205	92.99	2-year	240.5
L205C-O	L205C-S	205C	93.39	2-year	136.8
L205D-O	L205D-S	205	92.99	2-year	239.5

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 9** below.

Table 9: 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

Degrees	Coefficient
180	0.020

Orifice characteristics for the areas within Claridge Development were obtained from Novatech as shown in **Table 10** below.

Table 10: Claridge Areas Circular Orifice Characteristics

Orifice Name	Inlet CB ID	Outlet MH ID	Diameter (m)	Invert Elevation (m)	Discharge Coefficient	100-year Capture Rate (L/s)
OCB39-40	CB39-40	100	0.127	91.49	0.65	49.12
OCB41-42	CB41-42	102	0.094	91.75	0.65	27.08
OCB43-44	CB43-44	104	0.127	91.72	0.65	49.5
OCB45-46	CB45-46	106	0.102	91.73	0.65	32.23
OCB47-48	CB47-48	108	0.083	91.79	0.65	21.08
OCB49-50	CB49-50	110	0.083	91.97	0.65	20.94
OCB69-70	CB69-70	100	0.200	91.52	0.65	119.06
ORYCB01	RYCB01	102	0.083	92.00	0.65	12.36
ORYCB03	RYCB03	102	0.083	91.50	0.65	21.25
ORYCB07	RYCB07	104	0.094	91.87	0.65	28.32
ORYCB11	RYCB11	108	0.083	89.62	0.65	22.04

5.3 CONCEPTUAL MODELING 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 electronic model files on the enclosed CD.

5.3.1 Proposed Development Conceptual Hydraulic Grade Line Analysis

The 100-year hydraulic grade line (HGL) elevation across the proposed development and Street B in the adjacent Claridge development was estimated using the PCSWMM model for the 100-year, 3 hour Chicago storm. Table 11 below presents the clearance between the trunk sewer HGL and the proposed road grade along the trunk sewer. The site storm sewers will need to be significantly upsized to obtain the required HGL to USF clearance due to the water levels in the receiving channel and the downstream Jock River. The storm sewer design sheet is included in **Appendix C1**.

Table 11: Conceptual HGL Results

STM MH	Prop. Grade (m)	100-Year 3 HR Chicago HGL (m)	Prop. Grade-HGL Clearance (m)
100	93.53	91.69	1.84
102	93.59	91.99	1.60
104	93.88	92.01	1.87
106	93.83	92.06	1.77
108	94.28	92.06	2.22
110	94.51	92.06	2.45
201	93.64	91.71	1.93
201B	93.84	91.77	2.07
202	93.96	91.71	2.25
202B	94.14	91.84	2.30
203	94.00	92.04	1.96
203B	94.14	92.10	2.04
204	94.48	92.05	2.43
204B	94.58	92.14	2.44
205	94.57	92.14	2.43
205C	95.12	92.17	2.95

The model results indicate that there is sufficient clearance between the 100-year HGL and the proposed road grades. Detailed grading of the future developments should be done 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.

5.3.2 Major Flow

Due to the site configuration and grading constraints, major flows from the proposed development will be directed to Street B within the adjacent Claridge Development to the South.

Based on preliminary grading plans and plan and profiles obtained from Novatech for Street B, there are three major system outlets to the adjacent Claridge Subdivision streets and a fifth major system outlet that discharges to the storm outlet channel at the downstream end of Street B.

Table 12: 100-Year Major System Overflows

Major System Outlet	100-Year Major System Overflow (L/s)
Claridge Street 7	1.6
Claridge Street 5	35.6
Claridge Jockvale Road	26.7
Outlet Channel	37.3

The PCSWMM model is based on lumped drainage areas with major system storage represented in storage nodes that overestimate the major system peak flows to Street B. It is anticipated that the actual major system peak flow contribution from the proposed site to the major system outlets will be much lower once detailed grading is completed and the actual road configuration with available sag storage is included in the model during detailed design.

Table 13 shows the preliminary flow depth along Street B for the 100-year, 3 hour Chicago storm.

Table 13: 100-Year, 3hr Chicago Overland Flow Results

Storage Node ID	Drainage Area	Top of Grate Elevation (m)	Rim Elevation (m)	100 year, 3hr Chicago Storm	
				Max Surface HGL (m)	Total Surface Ponding Depth (m)
CB69-70	L100A	93.11	93.46	93.41	0.30
CB39-40	L102A	93.27	93.62	93.50	0.23
CB41-42	L110A	93.44	93.79	93.66	0.22
CB43-44	L112A	93.40	93.75	93.66	0.26
CB45-46	L114A	93.41	93.76	93.69	0.28
CB47-48	L116A	93.47	93.82	93.69	0.22
CB49-50	L118A	93.67	94.02	93.84	0.17
CB69-70	L100A	93.11	93.46	93.41	0.30

5.4 INTERIM CONDITION EXTERNAL DRAINAGE MANAGEMENT

The existing Burnett Municipal Drain, which is tributary to the Jock River bisects the site from north to south. The drain consists primarily of an open channel between the Barrhaven Town Centre and the confluence with the Jock River, with the exception of a portion of the drain that has been piped across an existing driving range south of the Barrhaven Town Centre that is approximately 170 m long.

It is recommended that the drain be re-directed to convey upstream peak flows to an outlet downstream of the proposed site until construction of the surrounding developments has been completed, at which point, it is anticipated that the municipal drain will be formally abandoned. Correspondence related to the abandonment of the municipal drain is included in **Appendix C3**.

6.0 GRADING

The South Nepean Town Centre lands drain predominantly from northeast to southwest. Existing drainage for the development is roughly divided between the centrally located Burnett Municipal Drain (to be abandoned) and uncontrolled sheet flows to the Jock River / Kennedy Burnett SWMF. The CDP and additionally the Kennedy-Burnett SWMF Servicing Options report provided preliminary grading for the development which has been included for reference in **Appendix E**. For the purposes of this report a conceptual grading plan has also been prepared which takes into account anticipated overland flow conveyance, cover over proposed sewers, and grade raise restrictions as identified in the geotechnical investigation (see **Section 10.0**). The conceptual grading plan has been provided for reference in **Appendix E**. A detailed grading design will be developed at the time of final design. Detailed grading will adhere to all requirements as outlined in the City of Ottawa guidelines.

The conceptual grading plan (**Drawing GP-1**) identifies grade raise restrictions identified in the geotechnical investigation. Areas where grades are expected to exceed the maximum permissible grade raise will be subject to either a pre-loading/surcharge program, or lightweight fill and/or other approved means outside of proposed rights-of-way to reduce the risks of unacceptable long-term post construction differential settlements.

7.0 UTILITIES

7.1 HYDRO

Accessible Hydro infrastructure exists along the eastern boundaries of the site via existing plant within Greenbank Road, in addition to the hydro infrastructure to be installed within the residential development to the south by others. Exact size, location and routing of hydro utilities will be finalized after design circulation. Transformer locations and positioning of required utility easements will be identified in the detailed design stage.

7.2 ENBRIDGE GAS

Similarly to Hydro, existing gas infrastructure exists at the eastern boundary of the subject site. Exact size, location and routing of gas infrastructure will be finalized after design circulation.

7.3 TELECOMMUNICATIONS

Both Bell and Rogers are expected to be able to service the subdivision. Infrastructure locations and easement requirements will be identified as part of the Composite Utility Planning process, following design circulation.

8.0 APPROVALS

Ontario Ministry of Environment, Conservation and Parks (MECP) Environmental Compliance Approvals (ECAs, formerly Certificates of Approval (CofA)) under the Ontario Water Resources Act will be required for proposed storm and sanitary sewers and inlet control devices (Transfer of review) for the proposed development. The Rideau Valley Conservation Authority should be circulated on such submissions so that CA sign-off may be given and submission of the ECAs may proceed. Permit from the Conservation Authority will be required for filling of the existing Burnett Municipal Drain.

An MECP Permit to Take Water (PTTW) may be required for the site as some of the proposed works may be below the groundwater elevation shown in the geotechnical report. The geotechnical consultant shall determine whether a PTTW is required at the detailed design stage / prior to construction.

9.0 EROSION CONTROL

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

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit extent of exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with plastic or synthetic mulches.
6. Provide sediment traps and basins during dewatering.
7. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
8. 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:

9. Verification that water is not flowing under silt barriers.
10. Clean and change silt traps at catch basins.

An erosion control plan for the development has been previously created by DSEL for the purposes of earthworks operations on-site, and a network of cutoff swales and a temporary sediment control basin has been installed to provide appropriate erosion and sediment control. It is proposed to maintain the current erosion control scheme, and migrate the sediment basin and cutoff swales eastwards along with construction phasing for the development.

Refer to Erosion and Sediment Control Plan included in **Appendix E** for the proposed location of silt fences, cutoff swales, temporary sediment basins and other erosion control structures.

10.0 GEOTECHNICAL INVESTIGATION

A geotechnical investigation for the development was completed by Paterson Group Inc. in March 2019. The report summarizes the existing soil conditions within the subject area and construction recommendations. For details which are not summarized below, please see the original Paterson report included in **Appendix D**.

Subsurface soil conditions within the subject area were determined through field investigations in February 2019 and October 2012. In total 11 boreholes were drilled and 8 test pits excavated throughout the subject lands. In general soil stratigraphy consisted of topsoil and/or a silty clay deposit overlaying glacial till. Bedrock was estimated to occur between depths of 5-15m. The thickness of the existing topsoil ranged from 250 to 400mm.

Groundwater levels were encountered between 0.55m and 3.32m in depth. It is expected that construction may occur below the existing groundwater table and therefore a permit to take water may be required.

Based on the observed soil conditions, a grade raise restriction of between 1.4m and 3.0m above existing grade was recommended for housing / roadways, and between 0.8m and 2.5m for apartment buildings. Areas where grades are expected to exceed the maximum permissible grade raise will be subject to either a pre-loading/surcharge program, or lightweight fill and/or other approved means outside of proposed rights-of-way to reduce the risks of unacceptable long-term post construction differential settlements.

11.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the preceding information, the following conclusions are summarized below:

11.1 POTABLE WATER ANALYSIS

- During peak hour conditions, the proposed system is expected to operate above the minimum pressure objective of 276kPa (40psi);
- The proposed system is capable of providing sufficient fire flow while maintaining a residual pressure of 138kPa (20 psi) in all areas based on hydraulic analysis done at the Master Servicing level. A final hydraulic analysis is to be completed at time of detailed design;
- As the development proceeds northwards, additional water transmission and available fire flows may necessitate connection to the watermain within existing Jockvale Road to the north, as determined by detailed hydraulic analysis for the development.

11.2 WASTEWATER SERVICING

The South Nepean Town Centre development will be serviced by a network of gravity sewers which will direct wastewater flows to the existing centrally located South Nepean Collector within the proposed extension of Jockvale Road. The proposed sanitary sewer design indicates one connection point to the existing SNC, with a total estimated peak outflow of 29.8L/s. Peak flows are expected to be well within the capacity of the existing SNC Phase 2 sewer.

11.3 STORMWATER MANAGEMENT

- The proposed stormwater management plan is in compliance with the goals specified in the background reports and the 2012 City of Ottawa Sewer Guidelines
- Inlet control devices will be proposed to limit inflow from the site area into the minor system to the 2-year storm (5-year for collector roads) event based on City of Ottawa IDF curves.
- The storm sewer hydraulic grade line will be maintained at least 0.30 m below the underside of footing in the subdivision during design storm events up to the 100-year storm.
- All dynamic surface water depths are to be less than 0.35 m during all storm events up to the 100-year storm.
- Minor system peak flows from the proposed site will be captured and directed to an HDS for quality control, and will ultimately discharge into the outlet channel for the Kennedy-Burnett SWM Facility.
- The proposed park, high density residential and school areas (L203C, L205B & L205D) have been assumed to provide on-site storage for storms up to the 100-year storm in order to minimize major system overflows to the Claridge development to the south. All other areas have been assumed to provide 50 m^3/ha of surface storage.

11.4 GRADING

A conceptual grading plan has been prepared taking into account required overland flow conveyance, cover over sewers, hydraulic grade line requirements, and grade raise restrictions as identified in the geotechnical investigation. A detailed grading design will be developed at the time of final design. Detailed grading will adhere to all requirements as outlined in the City of Ottawa guidelines.

11.5 UTILITIES

Utility infrastructure exists in the general area of the subject site. Exact size, location and routing of utilities will be finalized at the detailed design stage.