Riverside South Phase 8 Block 221 Site Servicing and Stormwater Management Report

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Prepared for:
Richcraft Group of Companies

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Sign-off Sheet

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March 30, 2020

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1.0 INTRODUCTION AND OBJECTIVE

Stantec Consulting Ltd. has been commissioned by Richcraft Group of Companies to prepare the following Site Servicing and Stormwater Management Report and undertake the civil servicing and grading design for Block 221 of the Riverside South Phase 8 Subdivision. The subject property is located within Phase 8 in the Riverside South Community (RSC), in the southeast quadrant of the intersection of Earl Armstrong Road and Ralph Hennessy Avenue within the City of Ottawa, as indicated in **Figure 1.1.** The proposed residential development comprises approximately 1.6 ha of land, and consists of 98 townhomes units, associated private streets, a parking area and a dry pond.

Figure 1.1: Approximate Location of Block 221

The objective of this report is to provide a servicing scenario for the site that is free of conflicts, provides on-site servicing in accordance with City of Ottawa design guidelines, and utilizes the existing local infrastructure in accordance with the various background studies as well as the RSDC Riverside South Phase 8 Site Servicing Report as outlined in **Section 2.0**.

2.0 BACKGROUND

This brief has been prepared in accordance with the following studies.

- Site Servicing Report Riverside Development Corporation Riverside South Phase 8 Part of 980 Earl Armstrong Road, J. L. Richards and Associates Limited, Revised October 2016
- Geotechnical Investigation, Proposed Residential Development, Riverside South Development (Phase 8), Ottawa, Ontario, Golder Associates, July 2015
- City of Ottawa Sewer Design Guidelines, City of Ottawa, October 2012 (including all subsequent technical bulletins)
- City of Ottawa Design Guidelines - Water Distribution, City of Ottawa, November 2011 (including all subsequent technical bulletins)

3.0 POTABLE WATER

A detailed Potable Water Servicing Analysis has been completed and is included in **Appendix A.** The analysis identifies and provides an evaluation of the potable water distribution network provided for Block 221 of the Riverside South Development.

3.1 BACKGROUND

The Block 221 development is proposed within the City of Ottawa pressure zone 2W/2C. The development consists of 98 townhome units.

The site will be serviced via two watermain connections to the 300 mm and 200mm diameter watermains with Ralph Hennessy Avenue and Markdale Terrace respectively (see **Drawing SSP-1**). On site fire protection will be provided by private hydrants located with a maximum of 90m spacing and within 90m of all building entrances. Proposed ground elevations of the site vary from approximately 91.6 m to 93.6 m. Under normal operating conditions, hydraulic gradelines vary from approximately 137.5 m to 147.8 m as confirmed through boundary conditions provided by the City of Ottawa in **Appendix A** and summarized in **Table 3.1** .

Table 3.1: Post-Configuration Boundary Conditions

Connection	Maximum HGL (m)	Peak Hour HGL (m)	Max. Day plus Fire HGL (m) 233 L/s (14,000 L/min)	Max. Day plus Fire HGL (m) 250 L/s (15,000 L/min)	Max. Day plus Fire HGL (m) 267 L/s (16,000 L/min)	Ground Elevation (m)
Ralph Hennessy Avenue (Connection #1)	147.8	145.6	143.7	143.3	142.9	91.60
Markdale Terrace (Connection #2)	147.8	145.6	138.1	137.5	136.9	91.52

1. Boundary conditions for Max. Day plus Fire flow of 267 L/s were interpolated from the values obtained from the City for 233 L/s and 250 L/s

3.2 WATER DEMANDS

Water demands for the development were estimated using the City of Ottawa Design Guidelines - Water Distribution. See **Appendix A** for detailed domestic water demand calculations.

The average day demand (AVDY) for the site was determined to be 1.1 L/s. The maximum daily demand (MXDY) was determined to be 2.7 L/s and was calculated as 2.5 times the AVDY. The peak hour demand (PKHR) totaled 5.9 L/s and was calculated as 2.2 times the MXDY.

Wood frame construction was considered in the assessment for fire flow requirements according to the FUS Guidelines. The FUS Guidelines indicate that low hazard occupancies include apartments, dwellings, dormitories, hotels, and schools, and as such, a low hazard occupancy / limited combustible building contents credit was applied. 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 eight dwelling units or 600 m^2 of building area. Based on calculations per the FUS Guidelines (**Appendix A**), the worst case required fire flows for this development is 267 L/s. Blocks with a total required fire flow greater than 267 L/s have been proposed to be separated by a two-hour firewall (FUS calculations for each proposed Block have been provided in **Appendix A**).

3.3 HYDRAULIC MODEL RESULTS

3.3.1 System Layout

The proposed watermain alignment and sizing for the proposed development is shown in **Figure 2** below with 200mm piping presented in blue.

Figure 2 Proposed Watermain Layout

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 peak hour, average day, 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 during the average day or peak hour scenarios was approximately 75.1 psi (517.7 kPa) and the maximum pressure modeled was approximately 79.5 psi (548 kPa). These pressures are within the serviceable limit of 50 to 80 psi (345 to 552 kPa) as per City of Ottawa guidelines. Results for system pressure at the junction points for average day and peak hour scenarios are displayed in **Figure 3** and **Figure 4** below.

Figure 3 AVDY Pressure Results (psi)

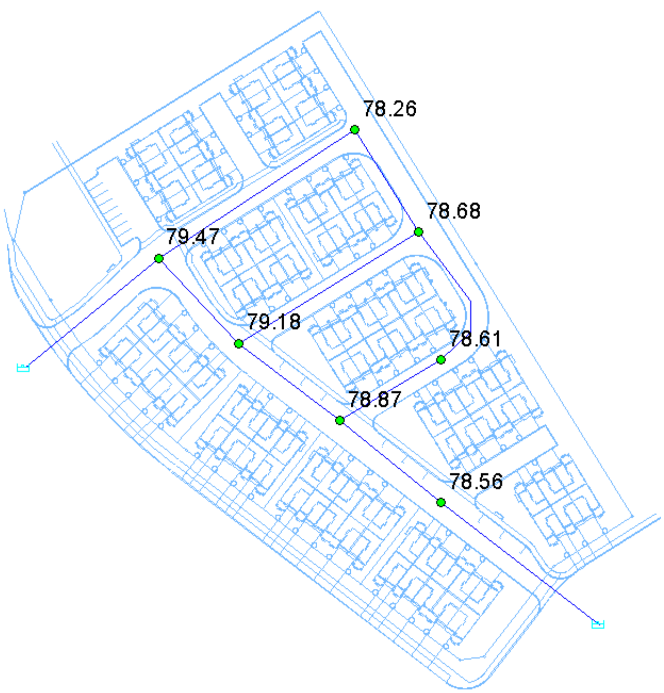


Figure 4 PKHR Pressure Results (psi)

A fire flow analysis was carried out using the hydraulic model to determine the anticipated amount of flow that could be provided for the proposed development under maximum day demands and worst-case fire flow requirements per the FUS methodology (267 L/s at hydrant locations). Results of the modeling analysis indicate that flows in excess of the required fire flow rate 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.** The results for residual pressure in the system at the junction points under a maximum day and fire flow scenario are displayed below in **Figure 5.**

Figure 5 MXDY + 16,000L/min Fire Flow Results (Residual Pressure (psi))

3.3.2 SUMMARY OF FINDINGS

Based on the findings of the hydraulic analysis, 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).

4.0 STORM DRAINAGE

The following sections describe the stormwater management (SWM) design for Block 221 in accordance with the background documents and governing criteria.

4.1 PROPOSED CONDITIONS

The proposed 1.6 ha development is located within Phase 8 of the Riverside South development and comprises 98 townhome units and a SWM dry pond. The storm sewer collection system for the site will discharge to the 1350 mm diameter storm sewer on Ralph Hennessy Avenue that ultimately directs runoff to the existing Riverside South (RS) Pond 1 (see **Drawings SD-1**). Quality control of stormwater runoff from the proposed Block 221 will be provided by the existing downstream RS Pond 1. Detailed grading has been designed to direct overland flows from the proposed development to a dry pond with an emergency overflow to Ralph Hennessy Avenue. Minor areas surrounding the site cannot be graded to drain internally and as such will sheet drain uncontrolled offsite.

4.2 CRITERIA AND CONSTRAINTS

The overall approach for storm servicing and stormwater management for the proposed development was outlined in J. L. Richard's Phase 8 Site Servicing Report (October 2016). Criteria were established by combining current design practices outlined by the City of Ottawa Design Guidelines (2012) as well as the conclusions made within the Phase 8 Servicing Report. The following summarizes the SWM criteria.

- Use of the dual drainage principle. (City)
- Minor system capture rate from Block 221 (Referred to as Block 212 or HD2) to be restricted to 70 L/s. (J. L. Richards – Phase 8)
- Major system overflows up to the 100-year storm from the proposed Block 221 to be stored on-site. (J. L. Richards – Phase 8)
- Size storm sewers to fully capture 2-year storm event under free-flow conditions (i.e. no ponding during 2-year storm event) using 2012 City of Ottawa Sewer Design Guidelines I-D-F parameters. (City) *It should be noted that the minor system target for the proposed site corresponds to a peak flow smaller than the 2-year storm runoff from the proposed site and as such, it is expected that the minor system will not be flowing full and that some ponding will occur during the 2-year storm event.*
- Assess impact of 2-year storm, 5-year storm, the worst case 100-year storm event, and the climate change scenario (worst case 100-year storm event with a 20% increase of rainfall intensity) on the major & minor drainage systems. (City)

- Separation of at least 0.3 m between the 100-year hydraulic grade line (HGL) and building under side of footing (USF) must be provided. (City)
- Maximum 'climate change' HGL to be lower than proposed basement elevations. (City)
- Inlet control devices (ICDs) to have a minimum orifice diameter of 83 mm. (City)
- Depth of flow may extend adjacent to the right-of-way provided that the water level does not touch any part of the building envelope and remains below the lowest building opening during the stress test event (100-year increased by 20%). (City)
- Total maximum depth of flow under static and dynamic conditions shall be less than 0.35 m during the 100-year event. (City)
- There must be at least 30 cm of vertical clearance between the spill elevation on the private street and the lowest building opening that is in the proximity of the flow route or ponding area. (City)
- There must be at least 30 cm of vertical clearance between the spill elevation on rear yard swales and the ground elevation at the building envelope that is in the proximity of the flow route or ponding area. (City)
- Minimum swale grades at 1.5% (subgrade provided for grades < 1.5%). (City)
- Minimum roadway profile grades at 0.5%. (City)
- Minimum roadway slope of 0.1% from crest-to-crest for overland flow route. (City)
- Provide adequate emergency overflow conveyance off-site. (City)

4.3 DESIGN METHODOLOGY

The design methodology for the SWM component of the development is as follows:

- Create a PCSWMM model that generates major and minor system hydrographs and assesses the minor system hydraulic grade line and the major system flow depths.
- Size inlet control devices for the proposed catchbasins to meet the 70 L/s minor system criteria.
- Ensure that the resulting 100-year hydraulic grade line does not encroach within 0.30 m of the proposed underside of footings (USF) for the proposed units.
- Ensure that total dynamic and static surface ponding depths do not exceed 0.35 m during the 100-year storm scenario.
- Size the SWM dry pond to capture post development overflows up to the 100-year storm.
- Confirm that climate change storm simulation does not result in flooding of properties.

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

In keeping with the minor system target peak outflow, Inlet Control Devices (ICDs) or orifice plates have been specified for all catchbasins to limit the inflow to the minor system which outlets to the 1350 mm diameter storm sewer on Ralph Hennessy Avenue. Restricted inlet rates to the sewer are necessary to meet the target peak outflows. Due to the restrictive minor system capture allowance for the site, detailed grading for the proposed development has been designed to direct all overland flows to a surface SWM storage area(dry pond) via a depressed curb in the parking area. The outlet will be controlled by an ICD that will slowly release stormwater into the site's minor storm sewer system.

Drawing SD-1 outlines the proposed storm sewer alignment, ICD locations, drainage divides, SWM surface storage location, and labels. A storm sewer design sheet is included in **Appendix B.1**.

4.4 MODELING RATIONALE

A comprehensive hydrologic modeling exercise was completed with PCSWMM, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure and major system segments. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems' response during various storm events. The following assumptions were applied to the detailed model:

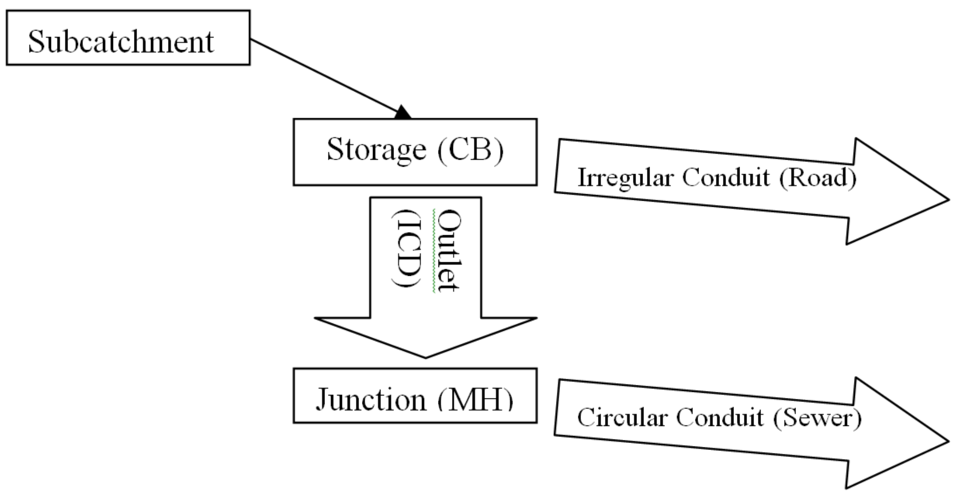
- Hydrologic parameters as per Ottawa Sewer Design Guidelines, including Horton infiltration, Manning's 'n', and depression storage values.
- 3-hour Chicago Storm distribution for the 2-year, 5-year and 100-year analysis.
- To 'stress test' the system a 'climate change' scenario was created by adding 20% of the individual intensity values of the 100-year storm at their specified time step.
- Percent imperviousness calculated based on actual soft and hard surfaces for the proposed catchments and converted to equivalent Runoff Coefficient using the relationship C = (Imp. X 0.7) + 0.2.
- Subcatchment areas are defined from high-point to high-point where sags occur.
- Width parameter was taken as twice the length of the street/swale segment for two-sided catchments and as the length of the street/swale segment for one-sided catchments.
- Catchbasin inflow restricted with inlet-control devices (ICDs) as necessary to maintain the minor system target peak outflow.

- Surface ponding in sag storage calculated based on grading plans (**Drawing SD-1**).

4.4.1 SWMM Dual Drainage Methodology

The proposed development is modeled in one modeling program as a dual conduit system (see **Figure 4.1**), with: 1) circular conduits representing the sewers & circular storage nodes representing manholes; 2) irregular conduits using street-shaped cross-sections to represent the saw-toothed overland road network from high-point to low-point and square storage nodes representing catchbasins and high points. The dual drainage systems are connected via outlet link objects from square storage node (i.e. CB) to circular storage node (i.e. MH), and represent inlet control devices (ICDs). Subcatchments are linked to the square storage nodes on the surface so that generated hydrographs are directed there firstly.

Figure 4.1: Schematic Representing Model Object Roles



Square storage nodes are used in the model to represent catchbasins as well as major system junctions. For square storage nodes representing catchbasins (CBs), the invert of the storage node represents the invert of the CB and the rim represents the maximum allowable flow depth elevation above the storage node (equal to the top of the CB plus an additional 0.35 m or higher). The additional depth has been added to rim elevations to allow routing from one surface storage to the next. Storage nodes that represent catchbasins at sags, are surrounded by two transects that represent the road segments forming the sag. The storage value assigned to the storage node represents the available 100 year ponding volume. If the available storage volume in a storage node is exceeded, flows spill above the storage node and into the sag in the irregular conduits (representing roads). The volume stored within the road sags is represented as flood volume in the model and includes the total static volume and the ponded depth above the node representing the dynamic flow depth. Flow storage volumes exceeding the sag storage available in the transect (roadway) will spill at the downstream highpoint into the next sag and continue routing through the system until ultimately flows either re-enter the minor system or reach the outfall of the major system. Storage nodes representing high points are assigned an invert elevation equal to the transect invert (spill elevation at edge of pavement) and a rim elevation equal to the maximum allowable flow depth elevation above the storage node (equal to the spill elevation at edge of pavement plus an additional 0.35 m). A Storage value of 0 has been assigned to these nodes to disable linear volume calculations. No storage has been accounted for within storage nodes at high points. In this manner, storage will accumulate according to the actual ponding depths before spilling along the roadway conduit, and to the next downstream road conduit.

Inlet control devices, as represented by orifice and outlet links, use a user-specified diameter and discharge coefficient or outflow rate taken from manufacturer's specifications for the chosen ICD model. A minimum orifice diameter of 108 mm has been specified to control outflow from the proposed dry pond.

4.4.2 Boundary Conditions

The detailed PCSWMM hydrology and the proposed storm sewers were used to assess the peak inflows and hydraulic grade line (HGL) in the proposed development. Fixed backwater levels were measured for the trunk sewer at the intersection of Shoreline Drive and Earl Armstrong Road and used at the stubbed outlet as obtained from the Site Servicing Report RSDC Phase 8 (J. L. Richards and Associates Limited, October 2016) and summarized in **Table 4.1.**

Table 4.1: Minor System Boundary Conditions

J. L Richards MH ID / Street Name	Fixed Water Elevation (m)			
	2-year	5-year	100-year	100-year + 20%
MH534 / Ralph Hennessy Avenue	88.39	88.39	88.39	88.39

4.5 INPUT PARAMETERS

Drawing SD-1 summarizes subcatchments used in the analysis of the proposed development and outline the major overland flow path. All parameters were assigned as per applicable City of Ottawa, Ministry of Environment Conservation and Parks (MECP) and background report requirements.

Key parameters for the subject area are summarized below; an example input file is provided for the 100-year, 3hr Chicago storm which indicates all other parameters (see **Appendix B.2**). For all other input files and results of storm scenarios, please examine the electronic model files provided with this report. This analysis was performed using PCSWMM, which is a front-end GUI to the EPA-SWMM engine. Model files can be examined in any program which can read EPA-SWMM files version 5.1.012.

4.5.1 Hydrologic Parameters

Table 4.2 presents the general subcatchment parameters used:

Table 4.2: 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 4.3 presents the individual parameters that vary for each of the proposed subcatchments.

Table 4.3: Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
L100A	0.19	45	3	50.00	0.55
L100B	0.01	9	3	74.29	0.72
L101A	0.35	157	3	82.86	0.78
L102A	0.29	110	3	78.57	0.75
L102B	0.29	96	3	78.57	0.75
L103A	0.08	21	3	84.29	0.79
L104A	0.29	126	3	82.86	0.78
UNC-1	0.03	60	3	0.00	0.20
UNC-2	0.11	78	3	38.57	0.47

Table 4.4 summarizes the storage node parameters used in the model. All catchbasins have been modeled as having an outlet invert as depicted on **Drawings SSP-1**. Static ponding depths, areas, and volumes within the proposed development area are as per **Drawings SD-1**, but are not explicitly included in the PCSWMM model as per methodology presented in **Section 4.4.1**.

Table 4.4: Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation* (m)	Total Depth (m)
L104A-S	90.53	92.26	1.73
L103B-S	92.80	93.15	0.35
L103A-S	91.17	92.90	1.73

Storage Node	Invert Elevation (m)	Rim Elevation* (m)	Total Depth (m)
L102B-S	90.97	92.70	1.73
L102A-S	90.76	92.49	1.73
L101A-S	90.56	92.29	1.73
L100D-S	91.66	92.01	0.35
L100B-S	90.29	92.02	1.73
L100A-S	88.97	91.99	3.02

*The rim of the storage node represents the maximum allowable flow depth elevation above the storage node (equal to the top of the CB plus an additional 0.35 m or higher).

4.5.2 Hydraulic Parameters

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

Storm sewers were modeled to confirm flow capacities, assess hydraulic grade lines (HGLs) and to determine minor system peak outflows to the outlet. The detailed storm sewer design sheet is included in **Appendix B.1**.

Table 4.5 below presents the parameters for the orifice and outlet link objects in the model, which represent ICDs. A coefficient of 0.572 was applied when using orifices to conform to head/discharge curves as supplied by the manufacturer for IPEX Tempest HF model ICDs.

Table 4.5: Orifice and Outlet Parameters for Proposed Catchments

Orifice Name	Catchbasin ID	Tributary Area ID	Minor System Node	ICD Type
L100A-IC	CB L100A-1	L100A	100	IPEX TEMPEST MHF (4.25" ORIFICE)
L101A-IC	CB L101A-1	L101A	101	IPEX TEMPEST LMF 75
L102A-IC	CB L102A-1	L102A	102	IPEX TEMPEST LMF 75
L102B-IC	CB L102B-1	L102B	102	IPEX TEMPEST LMF 75
L103A-IC	CB L103A-1	L103A	103	IPEX TEMPEST LMF 75
L104A-IC	CB L104A-1	L104A	104	IPEX TEMPEST LMF 75
L100B-IC	CB L100B-1	L100B	100	IPEX TEMPEST LMF 80

4.6 MODEL RESULTS AND DISCUSSION

The following section summarizes the key hydrologic and hydraulic model results. For detailed model results or inputs please refer to the example input file in **Appendix B.2** and the electronic model files on the enclosed CD.

4.6.1 Hydrology

Table 4.6 summarizes the orifice/outlet link maximum flow rates and heads across the proposed development.

Table 4.6: Orifice Link Results

Catchbasin ID	Tributary Area ID	ICD Type	5yr Head (m)	100yr Head (m)	5yr Flow (L/s)	100yr Flow (L/s)
CB L100A-1	L100A	IPEX TEMPEST MHF (4.25" ORIFICE)	2.00	2.60	32.4	37.03
CB L101A-1	L101A	IPEX TEMPEST LMF 75	1.46	1.48	6.04	6.09
CB L102A-1	L102A	IPEX TEMPEST LMF 75	1.48	1.51	6.09	6.14
CB L102B-1	L102B	IPEX TEMPEST LMF 75	1.46	1.48	6.05	6.09
CB L103A-1	L103A	IPEX TEMPEST LMF 75	1.42	1.44	5.97	6.00
CB L104A-1	L104A	IPEX TEMPEST LMF 75	1.43	1.44	5.98	6.01
CB L100B-1	L100B	IPEX TEMPEST LMF 80	0.29	0.88	3.08	5.35

4.6.2 Hydraulics

Table 4.7 summarizes the HGL results within the development during the 100-year, 3-hour Chicago storm event and the 'climate change' scenario storm required by the City of Ottawa Sewer Design Guidelines (2012), where 100-year intensities are increased by 20%.

Detailed grading of the site has been completed to ensure that the maximum hydraulic grade line is kept at least 0.30 m below the underside-of-footing (USF) of the adjacent units connected to the storm sewer during the 100-year storm event and below proposed basement elevations during the 'climate change' event.

Table 4.7: Modeled Hydraulic Grade Line Results

STM MH	Prop. USF (m)	100-year 3hr Chicago		100-year 3hr Chicago + 20%	
		HGL (m)	USF – HGL Clearance (m)	HGL (m)	USF – HGL Clearance (m)
100	90.22	88.40	1.82	88.40	1.82
101	90.29	88.43	1.86	88.43	1.86
102	90.58	88.93	1.65	88.93	1.65
103	90.58	89.40	1.18	89.40	1.18

STM MH	Prop. USF (m)	100-year 3hr Chicago		100-year 3hr Chicago + 20%	
		HGL (m)	USF – HGL Clearance (m)	HGL (m)	USF – HGL Clearance (m)
104	90.22	88.71	1.51	88.71	1.51
105	90.22	88.88	1.34	88.88	1.34
106	90.29	88.69	1.60	88.69	1.60
107	90.58	88.88	1.70	88.88	1.70

4.6.3 Overland Flow

Table 4.8 presents the maximum total surface water depths (static ponding depth + dynamic flow) above the top-of-grate of street catchbasins for the 100-year design storm and climate change storm. Based on the model results, the total ponding depth (static + dynamic) does not exceed the required 0.35 m maximum during the 100-year event.

Table 4.8: Maximum Static and Dynamic Surface Water Depths

Storage node ID	Structure ID	Top of Grate Elevation (m)	100-year, 3 hour Chicago		100-year, 3 hour Chicago+20%	
			Max Surface HGL (m)	Total Surface Ponding Depth (m)	Max Surface HGL (m)	Total Surface Ponding Depth (m)
L100A-S	CB L100A-S	91.64	0.00	91.57	0.08	91.72
L100B-S	CB L100B-S	91.67	0.00	91.17	0.00	91.51
L100D-S	CB L100D-S	91.66	0.20	91.86	0.23	91.89
L101A-S	CB L101A-S	91.94	0.09	92.04	0.10	92.05
L102A-S	CB L102A-S	92.14	0.15	92.27	0.16	92.28
L102B-S	CB L102B-S	92.35	0.10	92.45	0.11	92.46
L103A-S	CB L103A-S	92.55	0.04	92.61	0.04	92.61
L104A-S	CB L104A-S	91.91	0.04	91.97	0.05	91.98

4.6.4 Minor and Major System Peak Outflows

Minor system peak flows from the site are directed to the 1350 mm diameter storm sewer on Ralph Hennessy Avenue. Based on the PCSWMM model for the proposed development and the ICD release rates shown on **Table 4.6,** the 100-year minor system peak outflow from the proposed site is equal to 70.3 L/s which meets the 70 L/s minor system target for Block 221 as outlined in J. L. Richards' Phase 8 Servicing Report.

Similarly, major system peak flows from the majority of the site are directed to a dry pond storage system that will fully store the 100-year overflows to be released back into the minor system. The proposed dry pond will be located at the eastern boundary of the site and can store up to a maximum volume of 541m^3.

To allow the minor system outflow to be restricted at a discharge rate of 70L/s, the total required volume stored on site is 522m^3.

Table 4.9 summarizes the storage requirements for the proposed surface storage system.

Table 4.9: Proposed Dry Pond Requirements

Storm Event	Major System Dry Pond Storage (Bottom = 90.35 m)		
	Water Level (m)	Discharge (L/s)	Volume Used (m^3)
100-year, 3 hr Chicago	91.52	37.03	523
5-year, 3 hr Chicago	90.97	32.35	217
2-year, 3 hr Chicago	90.75	30.48	122

The overland flow from a small portion of the site cannot be captured internally and will flow unrestricted toward the adjacent right of ways (Ralph Hennessy Avenue and Markdale Terrace) within the Riverside South Phase 8 Subdivision. These segments of roadway were designed with generous volumes of sag storage that can accommodate the uncontrolled flows from the site as detailed in excerpts from the J. L. Richards Phase 8 Servicing Report included in **Appendix B.4**. During the 100-year storm event, 46.5 L/s of uncontrolled flow will sheet drain offsite from UNC-2 towards Ralph Hennessy Avenue and Markdale Terrace. Stage storage discharge calculations from the J. L. Richards Phase 8 Servicing Report were considered at each roadway sag leading to dry pond 2-P on Markdale Terrace.

A conservative analysis was undertaken where the flow from UNC-2 was added to each street segment between the proposed development and the Phase 8 dry pond 2-P to determine if the downstream major system has sufficient capacity to convey the 100-year uncontrolled runoff from the proposed development (area UNC-2). Based on the analysis described above, it was determined that the combined major system peak flow (Phase 8 plus the uncontrolled 46.4 L/s from the proposed development) is less than the conveyance capacity of each street segment at a depth of 0.35 m as demonstrated in **Table 4.10**.

Table 4.10: Stage Storage Conveyance Calculations

Street Ponding Area ID (J. L. Richards Phase 8)	Available Conveyance at 0.35m depth per J.L. Richards Report (L/s)	Maximum 100yr Conveyance per J.L. Richards Report (L/s)	100yr Conveyance + 46.5 L/s UNC-2 Flow (L/s)
P21	356	96	142

P24-1	851	184	230
P24-2	851	163	213
P25	385	104	150
P26	385	112	158
P27	383	107	153
P28	115	44	90
P29	Spill Over to Dry Pond 2-P		

*Values can be found in the J. L. Richards Phase 8 Servicing Report excerpts in Appendix B.4

Further, the 100-year uncontrolled peak flow from the proposed site was added to the downstream dry pond 2-P to demonstrate that the additional major system inflow is negligible and will not impact the performance of the dry pond (see excerpts from the J. L. Richards Phase 8 Servicing Report included in **Appendix B.4**). During 5-year and 100-year storm events, the major system peak inflow to pond 2-P as per J.L Richards report is 81 L/s and 372 L/s respectively, while the storage used is 45 m^3 and 747 m^3 respectively. Similarly, during a 100-year plus 20% storm event, the major system peak inflow to pond 2-P from Phase 8 is 895 L/s, while the storage used is 927 m^3. The additional 46.4 L/s peak flow generated from the proposed uncontrolled area UNC-2 was added to the maximum 100-year peak inflow to Pond 2P (418.4 L/s) and the storage requirement was interpolated to be 769 m^3 without accounting for major system peak flow reduction due to routing. Dry pond 2-P can store up to a volume of 927 m^3, allowing sufficient additional storage to capture the full 100-year uncontrolled peak flow from the proposed area UNC-2.

5.0 SANITARY SEWER

As shown on **Drawing SA-1**, the proposed Block 221 will be serviced by the 450 mm diameter sanitary sewer on Ralph Hennessy Avenue with a connection to the existing 200mm stub dropped inside the property line. The network of 200mm diameter sanitary sewers is proposed along the private streets. Servicing requirements for Block 221 were outlined in J. L. Richard's Phase 8 Site Servicing Report (October 2016) which included an estimated sanitary peak flow allocation for Block 221 of 3.53 L/s assuming high density residential land use with 60 units/ha and 1.9 person/unit for a total of 189 persons (Site Area = 1.66 ha).

The proposed development consists of eleven townhome blocks compromising 98 back to back townhome units. The sanitary sewer design sheet for the proposed development is included in **Appendix C**. **Table 5.1** shows the proposed sanitary peak flows from Block 221.

Table 5.1: Block 221 Sanitary Peak Outflow Results

Site Area	Number of Units	Population	Average Peaking Factor	Proposed Sanitary Peak Flows (L/s)	Extraneous Peak Flows (L/s)	Total Sanitary Peak Flow (L/s)
1.65	98	265	3.64	2.98	0.55	3.53

1. The above sanitary peak flows are based on 2.7 person/unit for townhomes, 280 L/cap/day average residential flows, and 0.33 L/s/ha for extraneous peak flows.

The above table shows that the proposed sanitary peak flows from Block 221 meet the 3.53 L/s peak flow assumed in J. L. Richard's Phase 8 Site Servicing Report.

The JLR Site Servicing Report for Riverside South Phase 8, there is more than 20 L/s of excess capacity in the 450 mm diameter receiving sewer on Ralph Hennessy and in the sanitary sewers downstream. The design flow for block 221 was based on a conceptual site plan at the time and an estimated population of 189 people. The proposed site plan has a design population of 265 people.

5.1 DESIGN CRITERIA

The following design guidelines were obtained from the City's "Ottawa Sewer Design Guidelines" and were used to estimate wastewater peak flow rates and 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)
- Townhouse unit Population – 2.7 persons/unit (City)

- Extraneous Flow Allowance – 0.33 L/s/ha (City)
- Manhole Spacing – 120 m (City)
- Minimum Cover – 2.5 m (City)
- Per Capita Residential Average Daily Flow – 280 L/p/day (City)

In addition, a peaking factor based on Harmon's Equation was used to determine the residential peak design flows.

6.0 GEOTECHNICAL CONSIDERATIONS

A geotechnical investigation was completed by Golder Associates in July of 2015 for Riverside South Phase 8. The report summarizes the existing soil conditions within the subject area and provides construction recommendations. For details which are not summarized below, please see Golder Report 1418804.

In general soil stratigraphy consisted of a topsoil layer followed by a layer of silty clay to clay overlaying a deposit of silty sand and clayey silt. Based on available geological mapping of the area, bedrock was anticipated at between 10 m – 25 m depth.

Groundwater levels were measured on January 27, 2015 and were estimated to be approximately 1.32 m below existing ground at BH 14-11 which is the borehole closest to the proposed Block 221.

Based on the existing borehole coverage, a grade raise of 1.9m is permissible in Area B where the proposed site is located.

The required pavement structure for local roadways is outlined in **Table 6.1** below.

Table 6.1: Pavement Structure – Local Roadways

Thickness (mm)	Material Description
90	Asphaltic Concrete
150	OPSS Granular 'A' Base
375	OPSS Granular 'B' Type II Subbase

7.0 GRADING AND DRAINAGE

The proposed development site measures approximately 1.6 ha in area. The topography across the site under existing conditions is relatively flat and generally slopes on a northerly direction. The objective of the grading design strategy is to satisfy the stormwater management requirements, adhere to permissible grade raise restrictions as much as possible (see **Section 6.0**), and provide for minimum cover requirements for sewers. The grading design also follows the recommendations outlined in the Site Servicing Report RSDC Phase 8 (J. L. Richards and Associates Limited, October 2016) and directs overland drainage to an onsite storage system which then discharges slowly back into the minor system. Refer to grading plan **Drawing GP-1** for the detailed grading plan of the development.

8.0 EROSION CONTROL DURING CONSTRUCTION

In order to control erosion and migration of sediment-laden runoff during construction, an erosion and sediment control plan will be required for the development. Therefore, an appropriate inspection and maintenance program is necessary to employed by the contractor, and will consider the following goals:

- Implementation of best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s);
- Immediate stabilization and containment of any exposed soil and/or stock piles;
- Minimization of areas to be cleared and grubbed;
- Protection of exposed slopes with plastic or synthetic mulches;
- Provision of sediment traps and basins during dewatering;
- Installation of sediment traps (such as SiltSack® by Terrafix) between catch basins and frames;
- Frequent inspection of all controls during construction and after significant rainfall events (greater than 13 mm) for sediment accumulation and erosion;
- Immediate repair of all noticeable erosion, with investigation into the cause so implementation of mitigation measures to prevent recurrence will be more successful;
- Maintenance of the erosion control measures during construction;
- Preparation of monitoring reports outlining the condition of erosion control works, their overall performance, and any actions such as repairs, replacement, or modification.

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

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

Drawing EC/DS-1 outlines the erosion and sediment control plan.

9.0 APPROVALS

Registration on the Environmental Activity Sector Registry or a Permit to Take Water may be required prior to construction in accordance with Ministry of Environment Conservation and Parks (MECP) requirements. This will be determined by the geotechnical consultant.

Given that the site is currently being developed as one parcel, no Environmental Compliance Approval (ECA) for the sewers or stormwater management is required at this time. Should separate parcels be created in the future, ECAs will be required from the MECP.

10.0 CONCLUSIONS

10.1 WATER SERVICING

Based on the findings of the hydraulic analysis, the proposed private water distribution 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 site will be serviced with watermain connections to the 300 mm diameter watermain on Ralph Hennessy Avenue and the 200 mm watermain on Markdale Terrace.

10.2 STORMWATER SERVICING

- Inlet control devices are proposed to limit inflow from the site area into the minor system to an overall minor system target of 70 L/s;
- 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;
- All dynamic surface water depths are less than or equal to 0.35 m during all design storm events up to the 100-year event;
- A surface storage system is located at the western boundary of the proposed site and is proposed to intercept major system overflows from the site and to contain 100-year overflows and discharge back into the minor system;
- Quality control for runoff from the proposed development will be provided in the existing Riverside South Pond 1.

10.3 SANITARY SERVICING

- The proposed development will generate a total sanitary peak flow of 3.53 L/s. The receiving sewer system has sufficient available capacity to receive the design flows;
- The preferred cover requirement of 2.5 m for the sanitary sewer system will be satisfied in all locations. Design guidelines for slope and velocity have been met within the proposed sewers.

10.4 GRADING

Detailed grading for the site has been designed to allow for the major overland flow to outlet as per the recommendations outlined in the Site Servicing Report RSDC Phase 8 (J. L. Richards and Associates Limited, October 2016) and to adhere to the grade raise restrictions recommended by the Geotechnical Investigation by Golder Associates.