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CD c/w PDFs of report, appendices, and drawings, as well as watermain hydraulic model files.
1.0 Introduction

Stantec Consulting Ltd. has been commissioned by Richcraft Homes ltd. to prepare the following servicing study in support of a proposal to develop 741 Bernard Street, situated on the east side of Bernard Street north of Hardy Avenue and south of the intersection of Bernard Street and Prince Albert Street. The site is located in the City of Ottawa and is indicated in Figure 1. The conceptual site development plan used for the purpose of this servicing brief consists of three, four-storey terrace homes residential blocks. Two blocks are composed of 16 units and the central block houses 19 units. Parking lots are located to the east of the first block, and to the south of blocks two and three, as illustrated on Drawing SGP-1. The 0.60 ha (1.48 acre) site was previously a school site, and the school building has been demolished. In March 2003, Stantec prepared a stormwater management report in support of two apartment buildings to be constructed on the site; however, these buildings were never constructed.

For the purpose of this report, it is assumed that Bernard Street runs North-South. The intent of this study is to provide site servicing and grading information in support of the site plan control application.
1.1 REVISIONS

Revisions were made to this report based on the comments received from the City of Ottawa on April 25, 2012. One of the recommendations by the City was to include a climate change stress test for the stormwater system. Rainfall intensities to the site were increased by 20% to model the effects of climate change on storm events. The site design was then evaluated at these increased intensities to provide reasonable assurance that flooding would not occur. It was determined that surface flooding would not occur in the climate change storm scenario. Various other changes were made as requested to the engineering drawings and specifications.
2.0 Site Grading

The existing topography of the site is gently sloped to the southwest corner of the property falling approximately 1.5m from northeast to southwest across the site. The greatest elevation change occurs through the eastern half of the site. The existing residential properties to the north currently sit at a higher elevation than the site with a portion of their drainage spilling onto the site. Under existing conditions a portion of the site drains to the southwest property corner and to Bernard Street, with the remainder of the site draining south to an existing ditch-inlet catchbasin.

Several factors need to be taken into consideration for site grading.

i. Appropriate depth of cover to be provided for site services in accordance with the Ottawa Sewer Design Guidelines.

ii. Ensure drainage patterns and overland flow route run west towards Bernard Street.

iii. Existing boundary conditions are to be maintained.

Drawing SGP-1 illustrates the proposed grading plan.
3.0 Water Servicing

3.1 BACKGROUND

3.1.1 Pressure Zone and Available Infrastructure

The subject property lies within the City of Ottawa’s 1E pressure zone. Water supply is delivered to the subject property from a 150mm diameter watermain in Bernard Street. A direct connection to the watermain is proposed to service the site, as illustrated on Drawing SSP-1. Two 150mm watermain services are proposed to connect to the existing 150 mm watermain in Bernard Street in order to provide looping of a portion of the proposed watermain.

3.1.2 Water Demand Calculations

Water demand calculations were based on the 2010 City of Ottawa Water Distribution Design Guidelines. The estimated population count for the site is 138 persons based on a density of 2.7 persons per unit. The unit counts for the proposed development area were provided by the developer for each type of residential block. During normal operating conditions it is estimated that the average daily, maximum daily and peak hourly domestic water demands for the site will be approximately 0.56 L/s, 1.40 L/s and 3.12 L/s respectively. Detailed water demand calculations are included in Appendix A.

3.1.3 Pressure Criteria

The City of Ottawa Water Distribution Design Guidelines state the design objective for system pressures under normal demand conditions shall remain between within the range of 345 and 550 kPa (50 to 80 psi) during average and maximum daily demand conditions. Under peak hour demand conditions the minimum allowable pressure is 276 kPa (40psi) at the ground elevation in the streets (i.e. at hydrant level). Under emergency and fire flow conditions, the minimum pressure in the distribution system is allowed to drop to 140kPa (20 psi). However as per the Ontario Plumbing Code, services to buildings with pressures that are expected to exceed 550kPa (80psi) require the use of pressure reducing valves. Ground elevations of the proposed development are presented in Drawing GP-1.

3.1.4 Hydraulic Boundary Conditions

The City of Ottawa provided watermain hydraulic boundary conditions and the available fire flow for 741 Bernard Street, in an email dated November 21, 2011. This data indicated a static pressure of 496kPa (72 psi) and a minimum pressure of 400kPa (58 psi) during peak hour conditions (See Appendix E).
3.1.5 Fire Flows

Based on correspondence from the City of Ottawa, fire flows of up to 7,320L/min (122L/s) can be provided at the proposed connection location on Bernard Street near Hardy Avenue. As required by the City of Ottawa Guidelines, the required fire flow for the proposed development was calculated as per the Fire Underwriter’s Survey (FUS) fire flow requirement calculation procedures. The calculations assumed ordinary construction with firewall separations between each unit and no sprinkler systems. Based on these assumptions the estimated required fire flow was 5,000 L/min (see Appendix A). The Ontario Building Code (OBC) fire flow calculation procedure could also be applied for this infill development to determine whether sufficient fire flow is available from the existing watermain. The fire flow calculations for the OBC method assumed a building construction of “Combustible with no fire resistance ratings”, the resulting minimum required fire flow was 2,700 L/m (see Appendix A). Therefore, the available fire flow of 7,320 L/min is sufficient to meet the minimum fire flow required as per both the Fire Underwriter’s Survey (5,000 L/min) and the Ontario Building Code (2,700 L/min).

3.2 HYDRAULIC ANALYSIS

In order to verify that pressure criteria are maintained within the system for the proposed development a hydraulic model of was run for the three demand scenarios. The analysis included average day (AVDY), peak hour (PKHR), static pressure check and maximum day plus fire flow (MXDY+FF).

Hydraulic boundary conditions were provided by the City and reported the maximum available fire flow of 122 L/s (7,320 L/min) at 20 psi, peak hour and average day conditions. The boundary condition at a fire flow of 5,000 L/min (83 L/s) was determined by interpolating between the PKHR head (109.6m) and the 20psi (82.8m) at the maximum fire flow. The resulting estimated boundary condition for the MXDY + FF scenario was 91.4m. The boundary conditions applied in the model were as follows:

AVDY & Maximum Pressure Check: 119.40 m
PKHR: 109.60 m
MXDY + FF (83 L/s): 91.40 m

A hydraulic analysis model was run using EPANET Z-v0.5 hydraulic modeling software. A fixed head reservoir was used in the model to simulate the boundary conditions. The reservoir was located at the proposed connection to Bernard Street. A copy of the model has been provided on the CD accompanying this report.

For the MXDY+FF analysis the FUS fire flow rate of 5,000 L/min was applied at the proposed hydrant on-site.

Any buildings greater than 2 storeys in height will require jet pumps in order to achieve sufficient pressures. Pumps should be sized by mechanical engineers.
3.2.1 Average Day Demand Results

The results of the AVDY analysis are presented in Appendix A.2. A fixed head reservoir elevation of 119.4 m located at the proposed connection to Bernard Street, was used to simulate the boundary conditions when the network is experiencing low demands and minimal head losses.

The resulting local minimum and maximum pressures were 71 psi (490 kPa) and 72 psi (496 kPa), respectively. These pressures are within the allowable pressure range of 40 to 80 psi (275kPa to 552kPa) as recommended by the City’s Water Distribution Design Guidelines. Based on the modeling results all services within the proposed development will meet the Ontario Plumbing Code requirements.

3.2.2 Peak Hour Demand Results

The results of the PKHR analysis are presented in Appendix A.2. A fixed head reservoir elevation of 119.6 m at the service connection was used to simulate the boundary conditions when the network is experiencing peak demands and high head losses.

The resulting local minimum and maximum pressures were 57 psi (393 kPa) and 58 psi (400 kPa), respectively. These minimum pressures are above the minimum allowable pressure of 40 psi (275kPa) as required by the City of Ottawa’s Water Distribution Design Guidelines. It is noted that the 4 storey terrace homes and condo flats will require jet pumps in order to achieve sufficient pressures. Pumps should be sized by mechanical engineers.

3.2.3 Maximum Day + Fire Flow Results

The City’s Water Distribution Design Guidelines require a minimum pressure of 20 psi (140kPa) to be maintained at all points in the distribution system under fire flow conditions. For this analysis a fire flow demand of 83 L/s, as determined from FUS calculations, was applied to the proposed fire hydrant on site. The location of the hydrant is indicated in Drawing SP-1 (EPANET model node ID: Junc 1).

The fire flow results in Table 3-1 show that fire flows of 83L/s (5,000L/min) can be supplied within the proposed local network while maintaining a residual pressure greater than 20 psi (140kPa).

<table>
<thead>
<tr>
<th>Node ID</th>
<th>Elevation</th>
<th>Demand</th>
<th>Head</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m</td>
<td>LPS</td>
<td>m</td>
<td>psi</td>
</tr>
<tr>
<td>Junc 3</td>
<td>69.10</td>
<td>0.00</td>
<td>89.03</td>
<td>19.93</td>
</tr>
<tr>
<td>Junc 4</td>
<td>68.90</td>
<td>0.00</td>
<td>88.77</td>
<td>19.87</td>
</tr>
<tr>
<td>Junc 5</td>
<td>68.85</td>
<td>0.44</td>
<td>88.11</td>
<td>19.26</td>
</tr>
<tr>
<td>Junc 6</td>
<td>68.68</td>
<td>0.52</td>
<td>85.23</td>
<td>16.55</td>
</tr>
<tr>
<td>Junc 7</td>
<td>69.00</td>
<td>0.44</td>
<td>85.23</td>
<td>16.23</td>
</tr>
</tbody>
</table>

Table 3-1: Results for Watermain Layout, MXDY Fire Flow = 83 L/s
3.3 RELIABILITY/LOOPING

As per Section 4.3.1 of the City of Ottawa Water Distribution Guidelines, watermain looping is required for all developments consisting of 50 units or more. The proposed single feed watermain services only 35 units which is less than the allowable 49 units. The remaining 16 units are serviced of the proposed looped watermain.
4.0 Wastewater Servicing

As illustrated on Drawing SSP-1, a 225mm diameter sanitary sewer exists within Hardy Avenue. It is proposed to make a 200mm dia. sanitary sewer connection to this existing sewer to service the proposed residential development.

Peak wastewater generation from the proposed development was estimated based on an estimated building population of 107 residents using the City of Ottawa Sewer Design Guidelines, November 2004; see Drawing SAN-1 for outline of sanitary drainage area and population count.

It is estimated that the proposed development will produce a peak wastewater flow rate of approximately 1.93L/s including an extraneous flow rate of 0.20L/s (see Appendix B). The sanitary sewer design sheet is included in Appendix B.
5.0 Storm Servicing and Stormwater Management

Pre-development conditions at the site are such that the majority of the stormwater runoff from the site area (0.46 ha) drains to an existing ditch-inlet catchbasin at the southern property line. The remaining portion of the site drains towards the southwest corner of the property and onto Bernard Street/Hardy Avenue. As illustrated on Drawing SSP-1, a 300mm diameter storm sewer exists in Hardy Avenue and drains west to Isidore Street. A 300mm diameter storm sewer connection to this sewer is proposed to service the development.

Stormwater runoff from the proposed development will be directed to the Hardy Avenue storm sewer. The estimated 5-year target release rate for the site with a runoff coefficient of 0.40 (as per City correspondence) is 47 L/s. However, due to existing sewer constraints the maximum available capacity of the existing 300mm storm sewer is 41 L/s (see Appendix C.1 for analysis). Flows from the neighbouring properties to the north and a portion of Bernard Street and the front yards of the lots along the west side of Bernard Street across from the site, will be controlled and conveyed to the Hardy Avenue storm sewer. The overall post-development runoff coefficient for the area draining to Hardy Avenue is 0.61 for a 5 year storm event. This overall post-development runoff coefficient would be increased to 0.71 for a 100 year storm event.

The following stormwater management criteria have been provided through consultation with City of Ottawa staff:

- In discussions with the City, flow restrictions to meet the 5 year target flows at C=0.4, at a time-of-concentration (tc) = 20 minutes, are required. All stormwater runoff from the development towards the Bernard Street right of way (ROW) up to and including the 100-year event is to be restricted to this flow rate.

- Due to capacity restrictions in the existing Hardy Avenue sewer however, the release rate will be controlled to less than the required 5-year pre-development rate. Flows up to the 100-year event are to be restricted to 41 L/s to ensure adequate sewer capacity.

- All unreleased flows and up to and including the 100-year storm, must be detained on site. Sufficient onsite storage will be necessary to retain stormwater accumulation in excess of the maximum allowable release rate. The site is divided into nine (9) storm drainage areas indicated on Drawing SSP-1.

- Flows draining from neighbouring properties onto the site and a portion of Bernard Street, are to be captured, controlled and conveyed to the Hardy Avenue storm sewer.

Since the rooftops of the proposed buildings are peaked, no stormwater will be stored on the rooftops.
Where possible, site catchments were designed with storage components. Maximum 100-year ponding limits are shown in Drawing SD-1 to illustrate that building entrances are at least 0.3m above the ponding limits. Stormwater flows generated from neighbouring properties will be collected and restricted to meet the allowable release rate to the Hardy Avenue storm sewer. Inflows of stormwater runoff generated within drainage area 8 along Bernard Street to the Hardy Avenue storm sewer will be restricted to protect the storm sewer and nearby connecting basements. Additional flows will not be stored due to existing grading restrictions. These excess flows will drain overland along Hardy Avenue to be picked-up in the downstream section of the sewer where sufficient capacity exists.

Table 5-1 summarizes the maximum release rates and storage volumes for each tributary catchment during the 5 and 100 year events. As per section 5.4.5.2.1 of the City of Ottawa Sewer Guidelines (2004), post-development runoff coefficients have been increased by 25% when analyzing the 100 year return period event.

### Table 5-1: Summary of Catchment Storage

<table>
<thead>
<tr>
<th>Area ID</th>
<th>Release Rate (L/s)</th>
<th>Storage Volume (m³)</th>
<th>Release Rate (L/s)</th>
<th>Storage Volume (m³)</th>
<th>Available Storage (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.9</td>
<td>0.44</td>
<td>1.9</td>
<td>2.26</td>
<td>5.3</td>
</tr>
<tr>
<td>2</td>
<td>10.5</td>
<td>12.6</td>
<td>10.5</td>
<td>34.9</td>
<td>35.8</td>
</tr>
<tr>
<td>3</td>
<td>3.9</td>
<td>26.0</td>
<td>3.9</td>
<td>61.1</td>
<td>107.5</td>
</tr>
<tr>
<td>4&amp;5</td>
<td>3.0</td>
<td>22.3</td>
<td>3.0</td>
<td>60.7</td>
<td>64.2</td>
</tr>
<tr>
<td>6</td>
<td>3.9</td>
<td>19.1</td>
<td>3.9</td>
<td>45.8</td>
<td>107.5</td>
</tr>
<tr>
<td>7</td>
<td>2.5</td>
<td>2.64</td>
<td>2.5</td>
<td>7.45</td>
<td>7.5</td>
</tr>
<tr>
<td>8</td>
<td>14.8</td>
<td>-</td>
<td>14.8</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

*Note: Areas 3 and 6 have a combined available storage of 107.5m³, with a combined storage requirement of 106.9m³ (61.1m³+45.8m³) in the 100 year scenario.

As shown in Table 5-1, there is sufficient storage available to contain ponding on-site during the 5 and 100 year events without spilling off-site.

Due to grading restrictions on-site, catchment area 9 was designed without a storage component. Area 9 flows offsite uncontrolled and is not tributary to the Hardy Avenue storm sewer. Areas that discharge offsite without entering the proposed stormwater management system are considered “non-tributary”. Table 5-2 summarizes the peak uncontrolled 5 and 100 year catchment release rates for the uncontrolled area 9.

### Table 5-2: Non-Tributary Areas

<table>
<thead>
<tr>
<th>Area ID</th>
<th>Area (ha)</th>
<th>Tc (min)</th>
<th>C₅(5-Year)</th>
<th>Q₅(5-Year) (L/s)</th>
<th>C₅(100-Year)</th>
<th>Q₅(100-Year) (L/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>0.037</td>
<td>10</td>
<td>0.2</td>
<td>2.14</td>
<td>0.25</td>
<td>4.6</td>
</tr>
</tbody>
</table>
Detailed Modified Rational Method stormwater management calculations are included in Appendix C.2. Table 5-3 and Table 5-4 summarize flow rates and storage requirements for the 741 Bernard Street site during the peak total flow from the site and external areas tributary to the most upstream portion of the Bernard Street sewer. Note that some totals may not sum exactly due to rounding.

Table 5-3: 5 year estimated post-development release rate to Hardy Avenue and maximum storage requirements

<table>
<thead>
<tr>
<th>Catchment ID</th>
<th>Area (ha)</th>
<th>C (5 Year)</th>
<th>Q\text{\text{release}} (L/s)</th>
<th>V\text{\text{stored}} (m}^3)</th>
<th>tc at Max Volume (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.045</td>
<td>0.20</td>
<td>1.9</td>
<td>0.4</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>0.117</td>
<td>0.90</td>
<td>10.5</td>
<td>12.6</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>0.133</td>
<td>0.90</td>
<td>3.9</td>
<td>26.0</td>
<td>45</td>
</tr>
<tr>
<td>4</td>
<td>0.172</td>
<td>0.36</td>
<td>3.0</td>
<td>22.3</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>0.116</td>
<td>0.32</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>0.107</td>
<td>0.90</td>
<td>3.9</td>
<td>19.1</td>
<td>35</td>
</tr>
<tr>
<td>7</td>
<td>0.026</td>
<td>0.90</td>
<td>2.5</td>
<td>2.6</td>
<td>15</td>
</tr>
<tr>
<td>8</td>
<td>0.135</td>
<td>0.59</td>
<td>14.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Total to sewer</strong></td>
<td></td>
<td></td>
<td><strong>40.5</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0.037</td>
<td>0.20</td>
<td>2.1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>0.888</strong></td>
<td><strong>0.61</strong></td>
<td><strong>42.6</strong></td>
<td><strong>83.0</strong></td>
<td>-</td>
</tr>
</tbody>
</table>
Table 5-4: 100 year estimated post-development release rate to Hardy Avenue and maximum storage requirements

<table>
<thead>
<tr>
<th>Catchment ID</th>
<th>Area (ha)</th>
<th>C (100 Year)</th>
<th>Q_{release} (L/s)</th>
<th>V_{stored} (m^3)</th>
<th>tc at Max Volume (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.045</td>
<td>0.25</td>
<td>1.9</td>
<td>2.3</td>
<td>20</td>
</tr>
<tr>
<td>2</td>
<td>0.117</td>
<td>1.00</td>
<td>10.5</td>
<td>34.9</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>0.133</td>
<td>1.00</td>
<td>3.9</td>
<td>61.1</td>
<td>80</td>
</tr>
<tr>
<td>4</td>
<td>0.172</td>
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<td>3.0</td>
<td>60.7</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td>0.116</td>
<td>0.40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>0.107</td>
<td>1.00</td>
<td>3.9</td>
<td>45.8</td>
<td>60</td>
</tr>
<tr>
<td>7</td>
<td>0.026</td>
<td>1.00</td>
<td>2.5</td>
<td>7.5</td>
<td>30</td>
</tr>
<tr>
<td>8</td>
<td>0.135</td>
<td>0.74</td>
<td>14.8</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Total to sewer</td>
<td></td>
<td></td>
<td>40.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0.037</td>
<td>0.25</td>
<td>4.6</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>0.888</td>
<td>0.71</td>
<td>45.1</td>
<td>212.1</td>
<td></td>
</tr>
</tbody>
</table>

As demonstrated in Table 5-3 and Table 5-4, sufficient storage is available to restrict sewer inflows to 41.0L/s and total site flows to less than the target rate of 47L/s. Therefore, sufficient stormwater retention is available on-site in order to meet the specified criteria for the development.

Flows are restricted with inlet control devices (ICDs) installed within the catchbasin and catchbasin manhole structures. All ICDs are installed upstream of building service connections and therefore a hydraulic analysis of the storm sewer was not completed as the design does not present a risk to basement flooding. An ICD summary table is included below in Table 5-5 and on Drawing SSP-1. See Appendix C.2 for the storm sewer design sheet.

Table 5-5: Summary of ICDs

<table>
<thead>
<tr>
<th>Structure ID</th>
<th>Location Inside Structure</th>
<th>Catchment Area ID</th>
<th>100 yr Ponding Depth (m)</th>
<th>Head (m)</th>
<th>ICD Type</th>
<th>ICD Model</th>
<th>Max Release Rate (L/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBMH1.1</td>
<td>west</td>
<td>1</td>
<td>0.10</td>
<td>1.90</td>
<td>IPEX</td>
<td>Tempest LMF-40</td>
<td>1.9</td>
</tr>
<tr>
<td>CBMH2.1</td>
<td>south</td>
<td>4/5</td>
<td>0.00</td>
<td>1.05</td>
<td>IPEX</td>
<td>Tempest LMF-55</td>
<td>3.0</td>
</tr>
<tr>
<td>CB3.1</td>
<td>north</td>
<td>2</td>
<td>0.30</td>
<td>1.90</td>
<td>IPEX</td>
<td>Tempest LMF-90</td>
<td>10.5</td>
</tr>
<tr>
<td>CB3.2</td>
<td>north</td>
<td>3</td>
<td>0.30</td>
<td>2.05</td>
<td>IPEX</td>
<td>Tempest LMF-55</td>
<td>3.9</td>
</tr>
<tr>
<td>CB3.3</td>
<td>west</td>
<td>6</td>
<td>0.30</td>
<td>2.05</td>
<td>IPEX</td>
<td>Tempest LMF-55</td>
<td>3.9</td>
</tr>
<tr>
<td>CB5.1</td>
<td>north</td>
<td>7</td>
<td>0.15</td>
<td>2.20</td>
<td>IPEX</td>
<td>Tempest LMF-45</td>
<td>2.5</td>
</tr>
<tr>
<td>CB5.2</td>
<td>north</td>
<td>8</td>
<td>0.00</td>
<td>2.20</td>
<td>IPEX</td>
<td>Tempest LMF-75</td>
<td>7.5</td>
</tr>
<tr>
<td>CB5.3</td>
<td>south</td>
<td>8</td>
<td>0.00</td>
<td>2.10</td>
<td>IPEX</td>
<td>Tempest LMF-75</td>
<td>7.3</td>
</tr>
</tbody>
</table>
5.1 CLIMATE CHANGE STRESS TEST

Based on the recommendations given in technical bulletin ISDTB-2012-1 issued by the City of Ottawa on January 31st, 2012, the storm sewer and storage volume capacity for this site was tested with increased rainfall intensities. An increase in intensity of 20% was used to model the potential effects of climate change on storm events. The 100 year intensities in the Modified Rational Method spreadsheet were increased by 20%, and the resulting volume increases were evaluated to determine if surface flooding of the adjacent buildings would occur at the site. Although overland spilling from the catchment area would occur above the spill-crest elevation, a ‘worst-case’ scenario where the crest was assumed to be blocked was used to determine the maximum volume that would be generated within the catchment. Table 5-6 shows the increase in volume and depth during the climate change scenario versus the 100 year design event.

Table 5-6: Climate Change Stress Test

<table>
<thead>
<tr>
<th>Storm Catchment Area</th>
<th>100 yr Ponding Volume (m$^3$)</th>
<th>100 yr (+20% intensity) Volume (m$^3$)</th>
<th>100 yr Ponding Depth (m)</th>
<th>100 yr (+20% intensity) Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area 1</td>
<td>2.20</td>
<td>3.12</td>
<td>0.10</td>
<td>0.15</td>
</tr>
<tr>
<td>Area 2</td>
<td>34.90</td>
<td>45.28</td>
<td>0.30</td>
<td>0.32</td>
</tr>
<tr>
<td>Area 4 &amp; 5</td>
<td>64.20</td>
<td>76.05</td>
<td>0.00</td>
<td>0.22</td>
</tr>
<tr>
<td>Area 3 &amp; 6</td>
<td>106.10</td>
<td>135.87</td>
<td>0.30</td>
<td>0.33</td>
</tr>
<tr>
<td>Area 7</td>
<td>7.50</td>
<td>9.21</td>
<td>0.15</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Almost all of the ponding depths for the 100 year +20% intensity storm are within a few centimetres of the regular 100 year storm ponding depths. Only Areas 4 & 5 would have a significant change. The volume created by the increase in intensity would no longer be able to be stored entirely in the storage pipe, and some surface ponding would occur. However, this surface ponding should not cause any surface flooding.

Since it is already a City requirement that elevations at the building units are to be 0.3 m above the spill crest of the drainage areas, the increase in depths shown above will not result in surface ponding reaching the units under the climate change scenario.

The ponding areas for the 100 year +20% intensity storm, along with the previously calculated ponding areas for the 5 and 100 year storms, can be found on Drawing SSP-1. The stormwater calculations used to prepare this climate change stress test can be found in Appendix C.3.

Table 5-7 shows the maximum release rate for the ICDs in each catchment area under the climate change conditions. Of our proposed sewers, the smallest pipe has a capacity of 43.9 L/s, which is still capable of conveying the peak inflow of 41.15 L/s from the inlet control devices under gravity conditions in the climate change storm scenario without surcharging.
SERVICING BRIEF, RICHCRAFT HOMES, 741 BERNARD STREET, OTTAWA, ON
June 14, 2012

Table 5-7: Climate Change Summary of ICDs

<table>
<thead>
<tr>
<th>Structure ID</th>
<th>Catchment Area ID</th>
<th>Max Release Rate (L/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBMH1.1</td>
<td>1</td>
<td>1.90</td>
</tr>
<tr>
<td>CBMH2.1</td>
<td>4/5</td>
<td>3.10</td>
</tr>
<tr>
<td>CB3.1</td>
<td>2</td>
<td>10.70</td>
</tr>
<tr>
<td>CB3.2</td>
<td>3</td>
<td>3.90</td>
</tr>
<tr>
<td>CB3.3</td>
<td>6</td>
<td>3.90</td>
</tr>
<tr>
<td>CB5.1</td>
<td>7</td>
<td>2.85</td>
</tr>
<tr>
<td>CB5.2</td>
<td>8</td>
<td>7.50</td>
</tr>
<tr>
<td>CB5.3</td>
<td>8</td>
<td>7.30</td>
</tr>
<tr>
<td><strong>Total Release Rate</strong></td>
<td><strong>41.15</strong></td>
<td></td>
</tr>
</tbody>
</table>

For the downstream existing sewers, the City has provided a figure of 41.0 L/s for the conveyance capacity of the existing sewer. The downstream sewers will then see an excess of 0.4% of the gravity free flow capacity of the pipe from the climate change scenario. Therefore, it can be expected that the sewer will convey the peak ICD flow with a surcharge of only a few centimetres; this is not expected to have an impact on the surrounding units.
6.0 Utilities

A 50mm diameter Gas service is available in Bernard Street and Bell, hydro and cable services currently exist overhead along the south and north property lines. Connections will be made to these existing services. The appropriate utility companies will be contacted at the site design stage to coordinate servicing to the proposed development.
7.0 Approvals

Ontario Ministry of Environment Environmental Compliance Approval (formerly Certificates of Approval (CofA)) under the Ontario Water Resources Act will be required for both the proposed storm and sanitary sewer connections to existing sewers within the Hardy Avenue ROW.

A MOE Permit to Take Water (PTTW) may be required for the site as some of the proposed works are below the groundwater elevation shown in the geotechnical report. The geotechnical consultant shall determine whether a PTTW is required.

The subject site is not adjacent to any floodplain or watercourse, and no modifications are proposed that would require an application for alteration of a watercourse from the local Conservation Authority under the Lakes and Rivers Improvement Act. There are no municipal drains adjacent to this site and no other approvals are required from other regulatory agencies.
8.0  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. 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 filter cloth 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:

1. Verification that water is not flowing under silt barriers.
2. Clean and change filter cloth at catch basins.

Refer to Drawing EC-1 for the proposed location of silt fences, straw bales, filter cloth and other erosion control structures.
9.0 Geotechnical Investigation

A geotechnical investigation was completed by Paterson Group Ltd. in April 2003. The report summarizes the existing soil conditions within the subject area and construction recommendations.

Subsurface soil conditions within the subject area were determined from six (6) boreholes distributed across the site area. Borehole depths ranged from 2.3 to 4.5m and locations are indicated in the test hole location plan in Appendix D. In general soil stratigraphy consisted of topsoil or pavement structure and fill, overlaying glacial till. Boreholes 3 and 4 indicated silty sand layers above the glacial till. Weathered shale or black shale bedrock was encountered in all boreholes at depths ranging from 1.98 to 2.84m below ground surface. The thickness of the existing topsoil at BH1 to BH4 ranged from 50 to 100mm. The existing asphalt at BH5 and BH6 had a thickness of 40 and 100mm, respectively.

Standpipes were installed in BH1 through BH6 and were monitored on April 8, 2003 for groundwater elevations. Groundwater levels ranged from 1.6 to 2.4m below ground surface.

The required pavement structures for the heavy-duty access areas and the parking lot areas are outlined in Tables 9.1 and 9.2 below.

Table 9-1: Pavement Structure – Heavy-duty/access lanes

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>HL-3 Wear Course</td>
</tr>
<tr>
<td>50</td>
<td>HL-8 Binder Course</td>
</tr>
<tr>
<td>150</td>
<td>OPSS Granular ‘A’ crushed stone</td>
</tr>
<tr>
<td>300</td>
<td>OPSS Granular ‘B’ Type II</td>
</tr>
<tr>
<td></td>
<td>Subgrade – either fill, insitu soil or OPSS Granular B Type I or II material place over insitu soil or fill.</td>
</tr>
</tbody>
</table>

Table 9-2: Pavement Structure – Parking Areas

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>HL-3 Wear Course</td>
</tr>
<tr>
<td>150</td>
<td>OPSS Granular ‘A’ crushed stone</td>
</tr>
<tr>
<td>300</td>
<td>OPSS Granular ‘B’ Type II</td>
</tr>
<tr>
<td></td>
<td>Subgrade – either fill, insitu soil or OPSS Granular B Type I or II material place over insitu soil or fill.</td>
</tr>
</tbody>
</table>

Geotechnical Investigation Report Excerpts are included in Appendix D. Refer to original report for additional details.
10.0 Conclusions

Water distribution for the site is available through a direct connection to a 150mm diameter watermain in Bernard Street. It is estimated that the demands for the proposed development will be in the order of **0.56 L/s**, **1.40 L/s** and **3.12 L/s** for average day, maximum day and peak hour respectively. The existing 150mm diameter watermain provides sufficient flow such that normal operating pressures remain within the City of Ottawa limits and that pressures are maintained above 140 kPa during MXDY + fire flow scenario.

Fire flow requirements were calculated using the Fire Underwriter’s Survey to be 5,000L/min; available fire flow is more than required, at 7,320 L/min.

A 200mm lateral connection to an existing 225mm diameter sanitary sewer within Hardy Avenue is proposed to service the development. It is estimated that the proposed development will produce a peak wastewater flow rate of approximately **1.93 L/s**.

Stormwater flows are conveyed to the existing 300mm diameter storm sewer within Hardy Avenue. The post-development release rate to the Hardy Avenue storm sewer for the 5-year and 100-year storms is **40.5 L/s**, which is below the target rate and available free flow capacity in the downstream sewer. Through a combination of surface and subsurface storage, the total stormwater flow release rate for the 5-year storm is restricted to **42.6 L/s**, including flows from external areas, which is below the pre-development rate of 47.0 L/s. The total calculated peak release rate under the 100-year storm condition is **45.1 L/s** including flows from external areas, also below the pre-development rate. The site also passes the climate change stress test for increased rainfall intensities, and the resulting stormwater flows.

Gas servicing (50mm diameter main) exists within the Bernard Street right-of-way. Hydro, Bell and cable servicing exists on overhead pole lines along the north and south property lines of the site. No issues are anticipated with respect to utilities for the proposed site.

MOE Environmental Compliance Approval will be required for the proposed storm and sanitary sewer connections; a Permit to Take Water may be required (to be determined by Geotechnical Engineer). No other approvals from other regulatory agencies are required.
All of which is respectfully submitted,

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