SERVICING AND STORMWATER MANAGEMENT REPORT
École Élémentaire Catholique Horizon-Jeunesse

349 Olmstead Street, Ottawa, Ontario

This document includes:

- Stormwater Management Report
- Watermain Analysis
-Assessment of Adequacy of Public Services
- Erosion and Sediment Control Brief (Plan Requirements Shown on Drawing C002)

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APPENDIX

1. GENERAL

1.1 Executive Summary

This report outlines site servicing criteria and civil engineering calculations pertaining to the servicing of a proposed new elementary school on a 5.6 ha site located at 349 Olmstead Street, in Ottawa. The site is bounded by residential development on the east, McArthur Avenue on the south, Olmstead Street on the west, and Jeanne Mance Street on the north. The new building will replace an existing school on the site, with the existing building remaining open while the new building is constructed. An adjoining existing building, with an entrance off of McArthur Avenue, houses a training centre for Conseil des Écoles Catholiques du Centre-Est, and will remain in place. The existing school building will be demolished following the opening of the new school building. The main entrance to the school will remain off of Olmstead Street.

The proposed work is divided into three phases. Phase 1, scheduled for the summer of 2016, will involve interior modifications in preparation for the building of the new school, and the related installation of new water, gas and communication services. The existing water, gas and communication services presently enter the west side of the existing building, and need to be relocated to the McArthur frontage in order to allow for continued servicing of the building during the new building phase. Phase 2 will be for the building of the new school on the west side of the existing building, and is scheduled to start in the fall of 2016. Demolition of the old school building will be done in Phase 3, starting in the fall of 2017.

The current building has one 150mm diameter water service off of the watermain on Olmstead Street. There are two sanitary services, one of 200mm diameter from the southwest corner of the existing school which connects to a manhole at the intersection of Olmstead and McArthur, and one of 225mm diameter, which is on the west side of the school, and connects to the City manhole at the intersection of Gladu and Olmstead. The former service will be removed, and the latter service is proposed to be maintained.

There are multiple storm services on the site, with discharges to the municipal sewers on all three fronting streets. The large northeastern section of the site, containing the old running track and sports field area, will not be altered, and will continue to drain via a storm service outlet to Jeanne Mance. In the centre north area of the site, an existing 375mm storm service connected to Jeanne Mance will continue to be used for the north central area. Also on the Jeanne Mance frontage, there is a single site catch basin with a connection to Jeanne Mance, and this catch basin will also be maintained.

The southeastern section of the site, including the existing east parking area, the frontage along McArthur, and the training centre portion of the building also will not be changed (other than some minor sidewalk installation), and will continue to drain via two storm sewer services discharging to McArthur, one to the east and one to the west.

On the Olmstead frontage, there is a single 300mm storm service connected to the City main at the intersection of Olmstead and Maple. This outlet will continue to be used for the proposed northwest area of the site. A second storm service is proposed to service the new school building and the front area, and will connect to the City main at the intersection of Olmstead and Gladu.

As per the City of Ottawa Sewer Design Guidelines, the redeveloped site will be required to provide storm water management quantity control. As noted in Section 4, runoff from the site is further restricted based on the existing capacity of the receiving municipal storm sewers.

Existing services and proposed service removals are noted on Drawing C002, along with sediment and erosion control requirements. The proposed grading is shown on Drawings C003 to C006, and servicing for the site is shown on Drawings C007 to C010. Drawing C011 provides a site drainage area plan for the proposed conditions.

The format of the report matches that of the development servicing study checklist found in section 4 of the City of Ottawa's Servicing Study Guidelines for Development Applications, November 2009.

1.2 Date and Revision Number

This version of the report is the first issue, dated May 16, 2016.

1.3 Location Map and Plan

Drawing C001 provides a location plan of the site. The entire existing site, including surveyed boundary is shown on Drawing C002. The architectural site plan, and the civil grading and servicing plans provide a detailed description of the proposed site layout and site work. The municipal address is provided on all drawings.

1.4 Adherence to Zoning and Related Requirements

The property is in conformance with zoning and related requirements, and is already designated as a school site.

1.5 Pre-Consultation Meetings

A pre-consultation meeting was held with representatives of the City of Ottawa, the School Board, and the consultant design team on December 2, 2015. A subsequent meeting between WSP and the City's infrastructure review engineer was held on February 4, 2016 to review possible storm water management options.

1.6 Higher Level Studies

The design for servicing has been undertaken in conformance with, and utilizing information from, the following documents:

- Ottawa Sewer Design Guidelines, October 2012.

- Ottawa Design Guidelines – Water Distribution, July 2010 and Technical Bulletin ISD 2010-02 Revisions to Water Design Guidelines.

- Untitled drainage area plan providing areas and allowable runoff coefficients, as determined by the City of Ottawa.

1.7 Statement of Objectives and Servicing Criteria

The objective of the site servicing is to meet the ultimate requirements for the development of the school site while adhering to the stipulations of the applicable higher level studies and City of Ottawa servicing design guidelines.

1.8 Available Existing and Proposed Infrastructure

Existing and proposed water, sanitary and storm infrastructure was briefly described in Section 1.1 above, and additional details are provided in subsequent sections for these utilities.

Communication services are currently provided off of McArthur, although one of the building entry points is on the west side of the existing school building. The existing natural gas service is off of Olmstead, also entering the west side of the existing school. Proposed communication and gas services will enter the McArthur side of the building.

Vehicular access to the site from McArthur will remain unchanged. The driveway loop on the Olmstead frontage will be removed. A single entrance to the parking lot for the school at the intersection of Olmstead and Maple will remain, and a second access point to this parking lot will be added from Jeanne Mance.

1.9 Environmentally Significant Areas, Watercourses and Municipal Drains

There are no environmentally significant areas, watercourses or municipal drains on or in close proximity to the site.

1.10 Concept Level Master Grading Plan

As the design is being submitted for site plan approval, the grading plan has been developed to the final design level. Existing grading along the site boundaries was identified in the topographic survey prepared in 2015. The grading plan confirms the feasibility of the proposed stormwater management, drainage, soil removal and fills. Site boundary proposed elevations

match with existing elevations on the site boundaries except where minor changes are made to permit new driveways.

1.11 Impacts on Private Services

There are no existing domestic private services (septic system and well) located on the site. There are no neighbouring properties using private services.

1.12 Development Phasing

The development will be phased, as the new building will be constructed at the west side of the site while the existing school is still in operation in the existing building. The existing building will be demolished following the transfer of the school operations to the new building. The new building can be developed without interference with the use of the present building and parking, with the exception of the 2016 Phase 1 interior renovations and service alterations.

A new water service will be provided in Phase 1. The existing sanitary service will be maintained. The new school storm service will be provided during Phase 2. All remaining new storm sewer work (which is all confined to the site), will be undertaken during Phase 3, following demolition of the old school building. The site plan does indicate possible future development of portable classrooms. The granular pads associated with the future free-standing portables and the impervious area of the portable classrooms has been taken into account in the stormwater management calculations. The population of the school is not expected to change from the existing condition, and is already well below the capacity of the existing building.

1.13 Geotechnical Study

A geotechnical investigation report has been prepared by exp Services Inc., and its recommendations have been taken into account in developing the civil engineering drawings. Copies of the geotechnical report are submitted with the City of Ottawa site plan application. This document will also be included with the contract documents to be issued for construction, and the recommendations of the report will be referenced in the construction specifications.

1.14 Drawing Requirement

The submitted Site Plan from Edward J. Cuhaci and Associates Architects provides a metric scale, north arrow, location plan, name of Owner, contact information for owner's representative, property limits including bearings and dimensions, existing and proposed structures and parking areas, easements, rights of way, and adjacent street names. Similar information is provided on the engineering plans submitted for site plan approval.

2. WATER SERVICING

2.1 Consistency with Master Servicing Study and Availability of Public Infrastructure.

No changes are required to the City's water distribution system to allow servicing for this property. City watermains and public hydrants are available on McArthur and Olmstead. The hydrant on McArthur will continue to service the existing training centre portion of the building that fronts onto McArthur. The new school building fronts onto Olmstead, and will be served by the existing hydrant on the southwest corner of Olmstead and Gladu. The fire department connection for the new school will be at the main Olmstead entrance, and is within 45 metres of the existing hydrant. No increase in domestic water demand is anticipated due to the use of the property being unchanged. The addition of a full sprinkler system in the new school building will increase the piped fire flow requirements in comparison to the existing school building.

2.2 System Constraints and Boundary Conditions

There are no known system constraints pertaining to the proposed development.

The following hydrant test results were provided by the City of Ottawa:

Hydrant 040 (McArthur): Static pressure 66 psi. Dynamic pressure >59 psi. Test flow 1278 USgpm (4837 L/min). Estimated flow at 20 psi of 3532 USgpm (13,369 L/min).

Hydrant 141 (Olmstead): Static pressure 68 psi. Dynamic pressure >59 psi. Test flow 1163 USgpm (4402 L/min). Estimated flow at 20 psi of 2871 USgpm (10,766 L/min).

2.3 Confirmation of Adequate Domestic Supply and Pressure

Domestic water supply requirements are much lower than fire supply requirements. Based on the water supply parameters listed above for the hydrant on McArthur, the available water supply can easily supply a suitable flow and pressure for the school.

2.4 Confirmation of Adequate Fire flow Protection

The calculated fire flow demand (FUS (Fire Underwriters Survey – Water Supply for Public Fire Protection) method) was determined separately for the new school building and for the existing training centre, each of which will have fire department connections.

For the new school, the estimated gross floor area is 3481.5 m^2, with non-combustible construction, limited combustible contents and a fully automated sprinkler system. Exposure charges of 5% for the west and 25% for the east were assumed due to the proximity of adjacent buildings. The estimated fire flow demand using the FUS method was 5950 L/min, which is well below the available flow of 10,766 L/min at 20 psi from the Olmstead hydrant.

For the existing training centre, the gross floor area is 3101.7 m^2, with normal construction, limited combustible contents and no reduction for a sprinkler system (although there is a standpipe system). An exposure charge of 25% for the west was assumed. The estimated fire

flow demand using the FUS method was 13,000 L/min, which is below the available flow of 13,369 L/min at 20 psi available from the McArthur hydrant.

2.5 Check of High Pressures

High pressure is not a concern based on the recorded static and dynamic pressures at the hydrants on Olmstead and McArthur, which ranged between 59 and 68 psi.

2.6 Phasing Constraints

As the existing water service room will need to be demolished to permit the construction of the new school, a replacement water service is needed prior to construction of the new building. Accordingly, some interior renovations and replacement of the water, gas, and communication services is proposed to be undertaken in Phase 1 of the project during the summer of 2016.

The new service is proposed to come off the 400mm main on the north side of McArthur, and enter the south side of the existing building. The proposed service size is 200mm diameter, as recommended by the mechanical designer. The existing 150mm diameter water service off of Olmstead will be removed from service and blanked.

2.7 Reliability Requirements

A shut off valve will be provided for the new building water service at the McArthur property line. Water can be supplied to the service from both the east and west.

2.8 Need for Pressure Zone Boundary Modification

There is no need for a pressure zone boundary modification.

2.9 Capability of Major Infrastructure to Supply Sufficient Water

The current infrastructure is capable of meeting the anticipated building demand.

2.10 Description of Proposed Water Distribution Network

A 200 mm diameter water service is proposed to be provided into the school site as proposed by the mechanical engineer. No private hydrants are required for the new building.

2.11 Off-site Requirements

No off-site improvements to existing watermains, feedermains, pumping stations, or other water infrastructure are required to meet the pipe service demands for the site.

2.12 Calculation of Water Demands

Water demands were calculated as described in Sections 2.3 and 2.4 above.

2.13 Model Schematic

As the water works consist of a single building service, a model schematic is not required.

3. WASTEWATER SERVICING

3.1 Design Criteria

The City of Ottawa Sewer Design Guidelines recommend a sanitary flow allowance of 50,000 L/ha/day for institutional uses, with a peaking factor of 1.5. The area of the school site is 5.62 ha. The peak flow allowed for the site calculated using the guidelines is therefore 3.25 L/s. The extraneous flow allowance is 0.28 L/s/ha, raising the peak estimated allowable flow to 4.82 L/s.

3.2 Consistency with Master Servicing Study

The existing sanitary service from the school building which outlets to the municipal sanitary sewer at Olmstead and Gladu, is proposed to be maintained. This service is reported to be a minimum of 200mm diameter. A CCTV investigation has been requested to confirm the condition of this sewer, and its suitability for continued use. The portion of the sewer within the footprint of the new school building will be replaced as part of the plumbing design. A new monitoring manhole is proposed to be placed on the service near the Olmstead property line.

The Ottawa Sewer Design Guidelines also provide estimates of sewage flows based on per capita unit rates, using both student and staff numbers. The anticipated average flow based on a projected school population of 390 students, plus an allowance for 200 staff at the school and training centre (at an average rate of 60 L/cap/d for elementary students and staff) is 0.41 L/s. Applying the peaking factor of 1.5, and adding the extraneous flow, the estimated peak flow is 2.19 L/s. This is less than the allowable flow for the site calculated in Section 3.1 above using the Sewer Design Guidelines, and considerably lower than the capacity of the existing 200mm minimum diameter service from the building to the monitoring manhole, and out to the receiving sewer.

3.3 Review of Soil Conditions

Based on the geotechnical report by exp Services Inc., there are no specific local subsurface conditions that suggest the need for a higher extraneous flow allowance.

3.4 Description of Existing Sanitary Sewer

The outlet for the sanitary service from the site is an existing sanitary manhole at the intersection of Olmstead and Gladu. Downstream of this manhole, a 300mm diameter sewer conveys the sewage to the west within the Gladu right of way.

3.5 Verification of Available Capacity in Downstream Sewer

As the proposed new school will have considerable less capacity than the former school, and will also utilize flow efficient fixtures, the anticipated sanitary flow will be less than under previous conditions. No capacity issues are therefore anticipated in the downstream sewer.

3.6 Calculations for Sanitary Sewers

Based on a combination of measured and estimated inverts, the existing building sanitary service has a slope of approximately 0.9%. The estimated capacity of a 200mm sewer at 0.9% slope is 32.5 L/s, which is well above the anticipate flow.

3.7 Description of Proposed Sewer Network

As noted, the only proposed addition to the existing sanitary sewer network is the addition of a monitoring manhole near the Olmstead property line.

3.8 Environmental Constraints

There are no previously identified environmental constraints that impact the sanitary servicing design in order to preserve the physical condition of watercourses, vegetation, or soil cover, or to manage water quantity or quality.

3.9 Pumping Requirements

The proposed development will have no impact on existing pumping stations and will not require new pumping facilities.

3.10 Force-Mains

No force-mains are required specifically for this development.

3.11 Emergency Overflows from Sanitary Pumping Stations

No pumping stations are required for this site.

3.12 Special Considerations

Site investigations have not yielded the need for special considerations for sanitary sewer design related to contamination, corrosive environments, or any other issue.

4. STORMWATER SERVICING

4.1 Description of Drainage Outlets and Downstream Constraints

The available drainage outlets from the school site are the existing 300/500mm diameter storm sewer on Jeanne Mance, the 300mm storm sewer at Olmstead and Maple, the 450mm storm

sewer at Olmstead and Gladu, the 300mm storm sewer on McArthur (west side of site) and the 375mm storm sewer on McArthur (east side of the site).

The City of Ottawa provided an area drainage plan indicating contributing areas and runoff coefficients. This information, as it pertains to the site, has been replicated on Drawing SKC1 included in the appendix to this report. The desired allowable flow release to the receiving sewer systems is the flow generated by a 2 year storm event, utilizing the areas and runoff coefficients noted on Drawing SKC1.

The development of the site to date, particularly on the McArthur frontage has not been consistent with the restrictions noted in the previous paragraph. The McArthur storm sewer network consists of two separate storm sewers fronting on the school site, one flowing east and one flowing west. The east system combines with the Jeanne Mance storm sewer at the intersection of Blake and Lacasse. The west system combines with the drainage from Olmstead at the intersection of McArthur and Cyr. Under existing conditions both sewer segments on McArthur receive greater flow from the school site than is desired. As the site drainage area contributing to McArthur is not within the section of the site that is being modified, additional controls are being proposed for the Jeanne Mance and Olmstead outlets to compensate for the excess flow going to McArthur.

With reference to Drawing SKC1, the allowable release rates to the receiving storm sewers can be calculated using the Rational Method, assuming a 2 year rainfall intensity of 77.10 mm/hour for a 10 minute time of concentration.

Drainage Outlet	Sub-Area No. on Drawing SKC!	Area (ha)	Runoff Coefficient	2 year Peak Flow (L/s)
McArthur West	A1	0.262	.55	30.9
Olmstead – Gladu	A2	1.082	.55	127.6
Olmstead – Maple	A3	1.082	.55	127.6
Jeanne Mance	A4	0.354	.45	34.1
Jeanne Mance	A5	0.798	.45	77.0
Jeanne Mance	A6	0.091	.55	10.7
Jeanne Mance	A7	0.445	.50	47.7
Jeanne Mance	A8	1.110	.40	95.2
McArthur East	A9	0.131	.75	21.1
McArthur East	A10	0.213	.75	34.2
McArthur East	A11	0.049	.70	7.4

Based on the above table, the desired allowable flows to the outlets are:

McArthur West 30.9 L/s
Olmstead – Gladu 127.6 L/s

Olmstead – Maple	127.6 L/s
Jeanne Mance	264.7 L/s
McArthur East	62.7 L/s

4.2 Analysis of Available Capacity in Existing Public Infrastructure

For the existing condition, the site of 5.6188 ha consists of 2.3969 ha of impervious area and 3.2219 ha of pervious area. The weighted average runoff coefficient is 0.527, based on assigning a runoff coefficient of 0.9 for the impervious area and 0.25 for the pervious area. Under 100 year conditions, the runoff coefficients for impervious and pervious areas are increased to 1.0 and 0.3125 respectively, resulting in a weighted average coefficient of 0.606. Using the Rational Method, and assuming a 10 minute time of concentration, the existing 2 year peak flow for the site is 634.7 L/s, and the existing 100 year flow is 1,694.4 L/s.

Previous design documents have noted the placement of ICD's on the central north drainage area going to Jeanne Mance, and on the recently added east parking lot next to the training centre. The majority of the site has uncontrolled flow.

Under proposed conditions, the impervious area is reduced to 1.9616 ha, the pervious area is increased to 3.5873 ha, and a gravel area of 0.0699 ha has been added for the portable classrooms. The weighted average runoff coefficient is 0.483 for the 2 year condition, and is 0.560 for the 100 year condition. The site will therefore generate less runoff than under present conditions. In addition, considerable retention storage and additional flow controls will be added to reduce the 100 year discharge rate to 623.8 L/s. Details of the stormwater management system area provided in subsequent sections.

As flows will be reduced in comparison to existing conditions, and will be restricted to approximately the 2 year release rate up to 100 year conditions, no adverse impact to the existing municipal storm sewer system is anticipated.

4.3 Grading, Drainage and Servicing Drawings

Drawings C003 to C006 provide proposed grading and drainage, and includes existing grading information. Drawing C007 to C010 indicate the receiving storm sewers and site storm sewer network. A drainage sub-area plan is provided on Drawing C011, with a breakdown of sub-area information. Sub-area information is also provided on the storm sewer design sheet attached to the appendix to this report. The servicing drawings also provide the location of roof drains and overflow scuppers for the proposed school building. Drawing SCK2 in the appendix to this report provides details of the roof ponding areas, depths and available storage volumes.

4.4 Water Quantity Control Objective

The water quantity objective for the site is to limit the flow release to approximately 613.5 L/s, with each of the outlets ideally matching the limits noted in Section 4.1 above. Excess flows

above this limit for the school site up to those generated by the 100 year storm event from drainage on the school site are temporarily stored on the site.

No provision is required on the school's site to accommodate any flow from the adjacent lands. All flows exceeding the defined minor system capacity and on-site storage capability will enter the major system, with overflow to the City right of way on the adjacent streets. The overland overflow locations are noted on the grading design drawings.

4.5 Water Quality Control Objective

No water quality objectives have been noted for the site by regulatory authorities.

4.6 Description of Storm Water Management Concept for Quantity Control

The drainage system consists of a series of existing and proposed manholes, catchbasins and storm sewers leading to the outlets on McArthur, Olmstead and Jeanne Mance. With reference to the drainage area plan on Drawing CO11, sub-areas 16, 18, part of 19, and 35 do not direct drainage to any site sewers, or directly to off-site storm sewers, but discharge stormwater via overland flow to the fronting streets.

No changes are proposed to the drainage systems on the McArthur frontage (including roof drainage from the portion of the existing building to remain), or to the system serving the large sports field and running track area in the northeast section of the site. Nevertheless, flow controls will be imposed in the areas that are being redeveloped in order to compensate for flows from uncontrolled areas at McArthur and at the running track area.

The concept for quantity control is to provide detention storage on the roof and ground surface areas, in conjunction with flow control devices, in order to reduce outlflows to the desired levels. For the McArthur frontage area, and the north east playing field, where it is not practical to introduce flow control or new storage, additional storage is added to adjacent areas draining to the same watersheds.

All flow calculations were based on the Rational Method, assigning runoff coefficients of 0.9 to impervious areas, 0.7 to gravel areas (located at the proposed portables), and 0.25 for the landscaped areas. Under the 100 year conditions used in the storage calculations, the runoff coefficients were increased to 1.0 for the impervious areas, 0.875 for the gravel areas, and 0.3125 for the landscaped areas. Flows are calculated using the formula $Q = 2.78 \times C \times I \times A$, where

- Q = flow in litres per second. C = runoff coefficient
- I = rainfall intensity (from City of Ottawa Sewer Design Guidelines)
- A = drainage area in hectares

Rainfall intensities were used as provided in the Ottawa Sewer Design Guidelines.

The site drainage has three existing separate outlets to Jeanne Mance, one existing outlet to Olmstead, and two existing outlets to McArthur. All of these existing storm sewer outlets will be maintained for the redeveloped site. One new storm outlet will be added for the new school

building, and will discharge to the existing municipal storm sewer at the intersection of Olmstead and Gladu.

The location and description of ICD's is noted on Drawing C009. A summary of the surface ponding for detention storage is provided on Drawing C010. Calculations are provide in Section 4.10 of this report.

4.7 Set-Back from Sewage Disposal Systems, Water Courses, and Hazard Lands

As there are no adjacent or on-site private sewage disposal systems, watercourses or hazard lands, there are no required setbacks.

4.8 Pre-Consultation with Ontario Ministry of the Environment and Climate Change, and Conservation Authority

Pre-consultation with the Ontario Ministry of the Environment and Climate Change, and the Rideau Valley Conservation Authority was initiated early in the design process. A further submission of the final design as submitted for site plan application will be made.

No formal responses have been received to date from either agency. The proposed development is expected to be exempt from a requirement for an Environmental Compliance Approval from the MOECC, as it is an institutional use connected to a City separated storm sewer, and drains only a single site.

4.9 Consistency with Higher Level Studies

As noted previously, the site is anticipated to have to meet the design constraints associated with the capacity of the existing municipal storm sewers, which are more stringent than the constraints included in the Ottawa Sewer Design Guidelines for site redevelopment when there are no additional constraints associated with local sewers.

4.10 Storage Requirements and Conveyance Capacity

Flow calculations and storage requirements are presented in this section for each of the stormwater outlet locations for the site.

McArthur West

As noted in Section 4.1 above, the desired flow release from the site to the McArthur west storm sewer is 30.9 L/s, based on a 2 year, 10 minute storm event. The contributing drainage area from the site to McArthur west consists of sub-areas 19 and 21A (refer to Drawing C011), and is estimated as 0.5901 ha, consisting of 0.1113 ha of impervious surface area, 0.1896 ha of roof area, and 0.2892 ha of pervious area. The weighted average runoff coefficient is 0.581, and is increased to 0.663 for the 100 year condition. Using the Rational Method, the 100 year, ten minute peak flow rate is calculated to be 194.7 L/s.

The proposed condition is essentially unchanged from the existing condition other than the addition of a pathway from the McArthur sidewalk to the building. As this frontage is otherwise

not being changed, and the existing contributing section of the building roof is also not being altered, there is no reasonable opportunity for introducing flow control or detention storage. The storage that would be required to attenuate the 100 year flow to 30.9 L/s is calculated using the Modified Rational Method as outlined in the table below.

McArthur West Discharge Area: Area = 0.5901 ha. Runoff coefficient = 0.663
Release rate = 30.9 L/s.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	263.9	30.9	233.0	69.9
10	179.0	194.7	30.9	163.8	98.3
15	146.8	159.7	30.9	128.8	115.9
20	119.95	130.5	30.9	99.6	119.5
25	103.85	113.0	30.9	82.1	123.2
30	91.9	100.0	30.9	69.1	124.4
35	82.58	89.8	30.9	58.9	123.7
40	75.15	81.7	30.9	50.8	121.9

A minimum storage volume of 124.4 m^3 is indicated for the 100 year event in order to restrict the discharge to 30.9 L/s. Additional storage and flow control will be introduced on the stormwater discharges to Olmstead in order to compensate for the unrestricted flow to McArthur west.

McArthur East

As noted in Section 4.1 above, the desired flow release from the site to the McArthur west storm sewer is 62.7 L/s, based on a 2 year, 10 minute storm event. The contributing drainage area from the site to McArthur east consists of sub-areas 21B, 33A, 33B and 35 (refer to Drawing C011), and is estimated as 0.6174 ha, consisting of 0.5278 ha of impervious surface area and 0.0896 ha of pervious area. The impervious area includes an allowance for the future parking lot that falls within the boundary of sub-area 33B. The weighted average runoff coefficient is 0.806, and is increased to 0.900 for the 100 year condition.

The proposed condition is essentially unchanged from the existing condition other than the addition of a small portion of sidewalk at the junction between the existing and new parking. As this area is otherwise not being changed, and the existing contributing section of the building roof is also not being altered, there is no reasonable opportunity for introducing additional flow control or detention storage.

Sub-areas 33A and 35 have uncontrolled flow.

Sub-area 33A consists of 0.1516 ha, with 0.1151 ha of impervious area, and 0.0365 ha of pervious area, with a weighted average runoff coefficient of 0.744. Under 100 year conditions, the runoff coefficient is increased to 0.834. Using the Rational Method, the 100 year, 10 minute release rate from sub-area 33A is 62.9 L/s.

Sub-area 35 consists of 0.0069 ha, all of which is pervious, with a weighted average runoff coefficient of 0.250. Under 100 year conditions, the runoff coefficient is increased to 0.3125. Using the Rational Method, the 100 year, 10 minute release rate from sub-area 35 is 1.1 L/s.

Sub-area 33B has an existing ICD rated at 10 L/s, located at CBMHEX5 in the east parking lot. Based on information available for the design of that parking lot, the detention storage available is 263 m^3. This area totals to 0.3373 ha, with 0.2911 ha of impervious area, and 0.0462 ha of pervious area. The runoff coefficient is 0.811, and increases to 0.906 for the 100 year condition. The Modified Rational Method is used to check the storage needed to reduce the flow to 10 L/s from this sub-area.

Sub-area 33B: Area = 0.3373 ha. Runoff coefficient = 0.906. Release rate = 10 L/s.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	206.1	10	196.1	58.8
10	179.0	152.1	10	142.1	85.3
15	146.8	124.7	10	114.7	103.2
20	119.95	101.9	10	91.9	110.3
25	103.85	88.2	10	78.2	117.3
30	91.9	78.1	10	68.1	122.6
35	82.58	70.2	10	60.2	126.4
40	75.15	63.8	10	53.8	129.1
45	69.05	58.7	10	48.7	131.5
50	63.95	54.3	10	44.3	132.9
55	59.62	50.7	10	40.7	134.3
60	55.89	47.5	10	37.5	135.0
65	52.65	44.7	10	34.7	135.3
70	49.79	42.3	10	32.3	135.7
75	47.26	40.1	10	30.1	135.5
80	44.99	38.2	10	28.2	135.4

A storage volume of 135.7 m^3 is indicated for the 100 year event in order to restrict the discharge to 10 L/s. This is less than the available storage capacity of 263 m^3, so no increase in the existing storage capacity is needed.

The total discharge from sub-areas 33A, 35 and 33B to McArthur east is 62.9 + 1.1 + 10 = 74 L/s. As this value is 11.3 L/s higher than the desired release rate of 62.7 L/s, additional restrictions will be placed on the discharges to Jeanne Mance.

Olmstead - Gladu

As noted in Section 4.1 above, the desired flow release from the site to the Olmstead-Gladu storm sewer is 127.6 L/s, based on a 2 year, 10 minute storm event. The contributing drainage area from the site to Olmstead-Gladu consists of sub-areas 16, 17, 18, 20 and 36 (refer to

Drawing C011), and is estimated as 0.2931 ha, consisting of 0.2174 ha of impervious surface area and 0.0757 ha of pervious area. The weighted average runoff coefficient is 0.732, and is increased to 0.822 for the 100 year condition.

Sub-areas 18 and 36 have uncontrolled overland flow, and consist of 0.0451 ha, with 0.0175 ha of impervious area, and 0.0276 ha of pervious area, with a weighted average runoff coefficient of 0.502. Under 100 year conditions, the runoff coefficient is increased to 0.579. Using the Rational Method, the 100 year, 10 minute release rate from sub-areas 18 and 36 is 13.0 L/s.

Sub-areas 16 and 17 also have uncontrolled flow but are connected to a storm sewer. These areas consist of 0.0524 ha, with 0.0043 ha of impervious area, and 0.0481 ha of pervious area, with a weighted average runoff coefficient of 0.303. Under 100 year conditions, the runoff coefficient is increased to 0.369. Using the Rational Method, the 100 year, 10 minute release rate from sub-areas 16 and 17 is 9.6 L/s.

Sub-area 20 is the new school roof area of 0.1956 ha. Flow control drains will be provided. The roof is divided into 7 sectors as illustrated in Drawing SKC2, found in the appendix to this report, which also indicates the roof drain locations, controlled flow rates, ponding depths, ponding areas, contributing drainage areas and ponding volumes. Potential volumes were determined using the inverted pyramid formula (Volume = surface area x depth /3). Actual available volumes were determined using the Modified Rational Method to balance the incoming rainfall versus the controlled release rate.

The lowest flow rate that can be achieved at an individual roof drain is 0.63 L/s.

Roof Area R1: Ponding area = 17 m^2. Contributing area = 23 m^2. Depth = 40mm. Potential volume = 0.23 m^3. The 100 year, 10 minute flow rate is 1.14 L/s, which is less than the minimum controlled flow rate of 0.63 L/s per drain. No flow control is necessary for R1.

Roof Area R2: Ponding area = 143 m^2 – skylight area of 26 m^2 = 117 m^2. Contributing area = 152 m^2. Depth = 150mm. Potential volume = 5.85 m^3. Release rate is set at 0.63 L/s for each of the two drains.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	10.25	1.26	8.99	2.70
10	179.0	7.56	1.26	6.30	3.78
15	146.8	6.20	1.26	4.94	4.45
20	119.95	5.07	1.26	3.81	4.57
25	103.85	4.39	1.26	3.13	4.69
30	91.9	3.88	1.26	2.62	4.72
35	82.58	3.49	1.26	2.23	4.68
40	75.15	3.18	1.26	1.92	4.61

In Area R2, 4.72 m^3 of storage is available.

Roof Area R3: Ponding area = 116 m^2. Contributing area = 125 m^2. Depth = 150mm. Potential volume = 5.8 m^3. Release rate is set at 0.63 L/s for each of the two drains.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	8.43	1.26	7.17	2.15
10	179.0	6.22	1.26	4.96	2.98
15	146.8	5.10	1.26	3.84	3.46
20	119.95	4.17	1.26	2.91	3.49
25	103.85	3.61	1.26	2.35	3.52
30	91.9	3.19	1.26	1.93	3.47

In Area R3, 3.52 m^3 of storage is available.

Roof Area R4: Ponding area = 254 m^2. Contributing area = 270 m^2. Depth = 150mm. Potential volume = 12.7 m^3. Release rate is set at 0.63 L/s for each of the two drains.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	18.21	1.26	16.95	5.09
10	179.0	13.44	1.26	12.18	7.31
15	146.8	11.02	1.26	9.76	8.78
20	119.95	9.00	1.26	7.74	9.29
25	103.85	7.79	1.26	6.53	9.79
30	91.9	6.90	1.26	5.67	10.21
35	82.58	6.20	1.26	4.94	10.37
40	75.15	5.64	1.26	4.38	10.51
45	69.05	5.18	1.26	3.92	10.58
50	63.95	4.80	1.26	3.54	10.62
55	59.62	4.48	1.26	3.22	10.63
60	55.89	4.20	1.26	2.94	10.58

In Area R4, 10.63 m^3 of storage is available.

Roof Area R5: Ponding area = 645 m^2 – 38 $m^2$for RTU units = 607 m^2. Contributing area = 998 m^2. Depth = 150mm. Potential volume = 30.35 m^3. Release rate is set at 1.89 L/s for each of the three drains.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	18.21	5.67	61.63	18.50
10	179.0	13.44	5.67	43.99	26.39
15	146.8	11.02	5.67	35.06	31.55
20	119.95	9.00	5.67	27.61	33.13
25	103.85	7.79	5.67	23.14	34.71
30	91.9	6.90	5.67	19.83	35.69
35	82.58	6.20	5.67	17.24	36.20
40	75.15	5.64	5.67	15.18	36.43
45	69.05	5.18	5.67	13.49	36.42
50	63.95	4.80	5.67	12.07	36.20

In Area R5, 36.43 m^3 of storage could theoretically be achieved, but available storage is limited by the roof geometry to a maximum of 30.35 m^3.

Roof Area R6: Ponding area = 327 m^2. Contributing area = 353 m^2. Depth = 150mm. Potential volume = 16.35 m^3. Release rate is set at 0.63 L/s for each of the two drains.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	23.81	1.26	22.55	6.76
10	179.0	17.57	1.26	16.31	9.79
15	146.8	14.41	1.26	13.15	11.83
20	119.95	11.77	1.26	10.51	12.61
25	103.85	10.19	1.26	8.93	13.39
30	91.9	9.02	1.26	7.76	13.97
35	82.58	8.10	1.26	6.84	14.36
40	75.15	7.37	1.26	6.11	14.66
45	69.05	6.78	1.26	5.52	14.90
50	63.95	6.28	1.26	5.02	15.06
55	59.62	5.85	1.26	4.59	15.15
60	55.89	5.48	1.26	4.22	15.19
65	52.65	5.17	1.26	3.91	15.25
70	49.79	4.89	1.26	3.63	15.25

In Area R6, 15.25 m^3 of storage is available.

Roof Area R7: Ponding area = 30 m^2. Contributing area = 35 m^2. Depth = 40mm. Potential volume = 0.4 m^3. Release rate is set at 0.63 L/s for each of the two drains.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	2.36	1.26	1.10	0.33
10	179.0	1.74	1.26	0.48	0.29
15	146.8	1.43	1.26	0.17	0.15
20	119.95	1.17	1.26	-	-

In Area R7, 0.33 m^3 of storage is available.

The total storage available on the roof is 4.72 + 3.52 + 10.63 + 30.35 + 15.25 + 0.33 = 64.8 m^3.

The controlled roof release rate is 1.14 + 1.26 + 1.26 + 1.26 + 5.67 + 1.26 + 1.26 = 13.11 L/s.

For the entire Olmstead-Gladu area, the combined release rate is the sum of 13.1 L/s from the roof, 13.0 L/s from sub-areas 18 and 36, and 9.6 L/s from sub-areas 16 and 17, for a total of 35.7 L/s.

Olmstead – Maple

As noted in Section 4.1 above, the desired flow release from the site to the Olmstead-Maple storm sewer is 127.6 L/s, based on a 2 year, 10 minute storm event. The contributing drainage area from the site to Olmstead-Maple consists of sub-areas 1 to 3, 5 - 9, 13 to 15, and 22 to 26 (refer to Drawing C011), and is estimated as 0.9938 ha, consisting of 0.6360 ha of impervious surface area, 0.2879 ha of pervious area, and 0.0699 ha of gravel surface. The weighted average runoff coefficient is 0.698, and is increased to 0.792 for the 100 year condition.

The rate of flow release from this sector is controlled using an ICD installed at STMH13, which is located at the driveway entrance off of Olmstead at the northwest corner of the site. The release rate of 66 L/s at this location will result in surface ponding at 16 locations, denoted as Locations A through P on the ponding table provided on Drawing C010. The ponding in all these locations will be controlled to a maximum elevation of 64.27m, which is the overflow elevation from the site at the parking lot entrance off of Olmstead. The ponding depth will be a maximum of 300mm in landscaped areas, and 200mm in paved areas. The delineation of the ponding areas is indicated on the grading and servicing drawings.

The total volume of ponding available for the Olmstead-Maple contributing area is 244.6 m^3. The Modified Rational Method is used to confirm that the release rate of 66 L/s will result in storage being generated that is similar to the volume available.

Olmstead-Maple Area: Area = 0.9938 ha. Runoff coefficient = 0.792. Release rate = 66 L/s.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	530.8	66.0	464.8	139.4
10	179.0	391.7	66.0	325.7	195.4
15	146.8	321.2	66.0	255.2	229.7
20	119.95	262.5	66.0	196.5	235.8
25	103.85	227.2	66.0	161.2	241.8
30	91.9	201.1	66.0	135.1	243.2
35	82.58	180.7	66.0	114.7	240.9
40	75.15	164.4	66.0	98.4	236.2

Storage generated is 243.2 m^3, which is similar to the measured volume available of 244.6 m^3.

Jeanne Mance

As noted in Section 4.1 above, the desired flow release from the site to the Jeanne Mance storm sewer is 264.7 L/s, based on a 2 year, 10 minute storm event. There are three separate piped outlets from the site to Jeanne Mance. Two of these are existing drainage outlets serving sub-areas 4 and 34, which will not be altered, and will not be flow controlled. The third outlet is at CBMHEX3 in the north central area of the site, and flow controls will be placed at several new locations contributing to this outlet.

Sub-area 4 consists of 0.0501 ha, comprised of 0.0014 ha of impervious area, and 0.0487 ha of pervious area, with a weighted average runoff coefficient of 0.268. Under 100 year conditions, the runoff coefficient is increased to 0.332. Using the Rational Method, the 100 year, 10 minute release rate from sub-area 4 is 8.3 L/s. This flow is collected and released at CBEX16.

Sub-area 34 consists of 1.8587 ha, comprised of 0.0410 ha of impervious area, and 1.8177 ha of pervious area, with a weighted average runoff coefficient of 0.264. Under 100 year conditions, the runoff coefficient is increased to 0.328. For this sub-area, the 100 year peak flow is calculated using a 21 minute time of concentration due to the long inlet path. The distance from the furthest point to the piped system entry is 86 m, and the average slope is 2.07%. Using Appendix 5-D of the Ottawa SDG, the inlet time is estimated as 21 minutes, which corresponds to a rainfall intensity of 116.3 mm/hour. The peak runoff rate from sub-area 34 is calculated to be 197.1 L/s. This flow is released via MHSTMEX2.

The uncontrolled release rate to Jeanne Mance consists of 8.3 L/s from sub-area 4, 197.1 L/s from sub-area 34, and 11.3 L/s allowed for the over contribution at McArthur East. These values are subtracted from the allowable release rate of 264.7 L/s to determine the allowable controlled release rate of 48.0 L/s at CBMHEX3.

The release rate of 48 L/s will be allocated to the sub-areas 10 to 12, and 27 to 32. New detention ponding will be created at CBMH10, CB2 and CB3 in the new parking lot on the north

side of the training centre, and in the landscaped area to the north of this parking lot at CBMH12 and CB's 19 to 22. ICD release rates at each of the control points were selected to match the achieved storage to the volume created by the grading design.

Controlled Area at CB19 (sub-area 11): Area = 0.2471 ha, with 0.0415 ha impervious, and 0.2056 ha pervious, with a weighted average runoff coefficient of 0.359. Under 100 year conditions, the runoff coefficient is increase to 0.428.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
30	91.9	27.0	2.0	25.0	45.0
35	82.58	24.3	2.0	22.3	46.8
40	75.15	22.1	2.0	20.1	48.2
45	69.05	20.3	2.0	18.3	49.4
50	63.95	18.8	2.0	16.8	50.4
55	59.62	17.5	2.0	15.5	51.2
60	55.89	16.4	2.0	14.4	51.8
65	52.65	15.5	2.0	13.5	52.7
70	49.79	14.6	2.0	12.6	52.9
75	47.26	13.9	2.0	11.9	53.6
80	44.99	13.2	2.0	11.2	53.8
85	42.95	12.6	2.0	10.6	54.1
90	41.11	12.1	2.0	10.1	54.5
95	39.43	11.6	2.0	9.6	54.7
100	37.90	11.1	2.0	9.1	54.6

At CB19, 54.7 m^3 of storage is available.

Controlled Area at CB20 (sub-area 32): Area = 0.2343 ha, with 0.0410 ha impervious, and 0.1933 ha pervious, with a weighted average runoff coefficient of 0.363. Under 100 year conditions, the runoff coefficient is increase to 0.433.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
30	91.9	25.9	1.8	24.1	43.4
35	82.58	23.3	1.8	21.5	45.1
40	75.15	21.2	1.8	19.4	46.6
45	69.05	19.5	1.8	17.7	47.8
50	63.95	18.0	1.8	16.2	48.6
55	59.62	16.8	1.8	15.0	49.5
60	55.89	15.8	1.8	14.0	50.4
70	49.79	14.0	1.8	12.2	51.2
80	44.99	12.7	1.8	10.9	52.3
90	41.11	11.6	1.8	9.8	52.9
100	37.90	10.7	1.8	8.9	53.4
110	35.20	9.93	1.8	8.13	53.7
120	32.89	9.28	1.8	7.48	53.9
130	30.90	8.71	1.8	6.91	53.9
140	29.15	8.22	1.8	6.42	53.9
150	27.61	7.79	1.8	5.99	53.9
160	26.24	7.40	1.8	5.60	53.8

At CB20, 53.9 m^3 of storage is available.

Controlled Area at STMH3 (sub-areas 28 to 31, with storage at CBMH10, CB2 and CB3): Area = 0.1782 ha, with 0.1128 ha impervious, and 0.0654 ha pervious, with a weighted average runoff coefficient of 0.661. Under 100 year conditions, the runoff coefficient is increase to 0.748.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	89.9	13.5	76.4	22.9
10	179.0	66.3	13.5	52.8	31.7
15	146.8	54.4	13.5	40.9	36.8
20	119.95	44.4	13.5	30.9	37.1
25	103.85	38.5	13.5	25.0	37.5
30	91.9	34.1	13.5	20.6	37.1
35	82.58	30.6	13.5	17.1	35.9
40	75.15	27.8	13.5	14.3	34.3

Upstream of STMH3, 37.5 m^3 of storage is available, with ponding at CBMH10, CB2 and CB3.

Controlled Area at CBMH12 (sub-area 27): Area = 0.1993 ha, with 0.0086 ha impervious, and 0.1907 ha pervious, with a weighted average runoff coefficient of 0.278. Under 100 year conditions, the runoff coefficient is increase to 0.342.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	46.0	12.6	33.4	10.0
10	179.0	33.9	12.6	21.3	12.8
15	146.8	27.8	12.6	15.2	13.7
20	119.95	22.7	12.6	8.1	9.7

At CBMH12, 13.7 m^3 of storage is available.

Controlled Area at CB22 (sub-area 10): Area = 0.2070 ha, with 0.0332 ha impervious, and 0.1738 ha pervious, with a weighted average runoff coefficient of 0.354. Under 100 year conditions, the runoff coefficient is increase to 0.423.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	59.1	16.4	42.7	12.8
10	179.0	43.6	16.4	27.2	16.3
15	146.8	35.7	16.4	19.3	17.4
20	119.95	29.2	16.4	12.8	15.4
25	103.85	25.3	16.4	8.9	13.3
30	91.9	22.4	16.4	6.0	10.8

At CB22, 17.4 m^3 of storage is available.

Controlled Area at CB21 (sub-area 12): Area = 0.1497 ha, all pervious, with a runoff coefficient of 0.25. Under 100 year conditions, the runoff coefficient is increase to 0.3125.

Duration Minutes	Intensity mm/hr	Q L/s	Q Released L/s	Difference	Storage m^3
5	242.6	31.6	1.7	29.9	9.0
10	179.0	23.3	1.7	21.6	13.0
15	146.8	19.1	1.7	17.4	15.7
20	119.95	15.6	1.7	13.9	16.7
25	103.85	13.5	1.7	11.8	17.7
30	91.9	12.0	1.7	10.3	18.5
35	82.58	10.7	1.7	9.0	18.9
40	75.15	9.77	1.7	8.07	19.4
45	69.05	8.98	1.7	7.28	19.7
50	63.95	8.32	1.7	6.62	19.9
55	59.62	7.75	1.7	6.05	20.0
60	55.89	7.27	1.7	5.57	20.0

At CB21, 20.0 m^3 of storage is available.

The total release rate to Jeanne Mance is 253.4 L/s, which is the sum of the uncontrolled and controlled release rates, summarized as follows:

Location	Release Rate (L/s)
CBEX16	8.3
MHSTMHEX2	197.1
CB19	2.0
CB20	1.8
CBMH12	12.6
C22	16.4
CB21	1.7
STMH3	13.5

For the site as a whole, the proposed release rates and desired release rates under 100 year conditions are:

	Proposed Release	Allowed Release
McArthur West	194.7 L/s	30.9 L/s
Olmstead Gladu	35.7 L/s	127.6 L/s
Olmstead Maple	66.0 L/s	127.6 L/s
McArthur East	74.0 L/s	62.7 L/s
Jeanne Mance	253.4 L/s	264.7 L/s
Total	623.8 L/s	613.5 L/s

The total proposed release rate of 623.8 L/s is less than 2% above the desired release rate of 613.5 L/s. The difference arises primarily at the McArthur West outlet, where existing flows are considerably higher than the desirable flow. As the difference is minor, and represents an existing condition, it is recommended that this variation be accepted.

4.11 Watercourses

There are no watercourses on or adjacent to the site.

4.12 Pre and Post Development Peak Flow Rates

The pre-development condition of the site is fully developed. The 5.6188 ha site was comprised of 2.3969 ha of impervious area, and 3.2219 ha of landscaped area. The pre-development runoff coefficient is 0.527.

Peak flow rates can be calculated for the 5.6188 hectare site using the Rational Method assuming a runoff coefficient of 0.527 and a time of concentration of 10 minutes. Rainfall intensities are provided in the Ottawa Sewer Design Guidelines, and are 77.10 mm/hour for the two year event, and 179 mm/hour for the 100 year event.

Pre-development 2 year peak flow = 2.78 x C x I x A = 2.78 x 0.527 x 77.1 x 5.6188 = 634.7 L/s.

Pre-development 100 year peak flow = 2.78 x C x I x A = 2.78 x 0.606 x 179 x 5.6188 = 1694.4 L/s.

Post-development peak flow rates under 100 year conditions are provided in Section 4.10 above. The estimated peak flow is 623.8 L/s, which compares favourably to the existing 2 year release rate from the site.

The drainage area plan on Drawing C011, Sections 4.6 and 4.10 above, and the storm sewer design sheets describe the post-development drainage areas and extent of imperviousness.

4.13 Diversion of Drainage Catchment Areas

Existing outlet locations have been maintained for drainage, and flows to the east and west sectors of the municipal drainage system have been balanced to generally meet the capacity constraints of these systems.

4.14 Minor and Major Systems

Proposed minor and major systems are shown on Drawing C007 to C010, and have been described in previous sections of the report. The minor site storm sewer system is described on the attached storm sewer calculation sheet. The stormwater management facility consists of a series of basins, each connected to a common sewer system leading to a series of controlled flow outlets. The stormwater storage ponding areas are immediately next to one another, and thus flow can cascade between these basins in the event of any blockage of their outlets prior to overflow occurring to neighbouring lands.

4.15 Downstream Capacity Where Quantity Control Is Not Proposed

This checklist item is not applicable to this proposed development as quantity control is provided.

4.16 Impacts to Receiving Watercourses

No adverse measurable impact is anticipated to downstream receiving watercourses due the separation of the site from the eventual receiving watercourse as a result of discharge through

City owned storm sewers and stormwater management facilities. The flow attenuation and quality treatment being implemented on the site will improve upon existing conditions.

4.17 Municipal Drains and Related Approvals

There are no municipal drains on the site or associated with the drainage from the site.

4.18 Means of Conveyance and Storage Capacity

The means of flow conveyance and storage capacity are described in Sections 4.6, 4.10 and 4.14 above.

4.19 100 Year Flood Levels and Major Flow Routing

The maximum 100 year ponding levels will vary with the location on the site, and will be determined by the site overland overflow elevations noted on the grading drawings. All ponding levels are well below the floor level of the existing and proposed buildings.

4.20 Hydraulic Analysis

Hydraulic calculations for the site storm sewers are provided in the storm sewer design sheet.

4.21 Erosion and Sediment Control Plan

This document addresses the City of Ottawa's requirement for an Erosion and Sediment Control Plan for the proposed construction.

Drawing C002 includes requirements for the Contractor to implement Best Management Practices to minimize erosion and sediment release during construction activities. Specific measures are dictated including a geotextile under catch basin grates, and a temporary silt fence on the down gradient property lines.

The Architect, as lead consultant, is responsible for ensuring contractual compliance with the construction specifications, including erosion and sediment control. The Engineer will be retained to provide periodic site observations and will therefore also monitor the condition of the erosion and sediment control measures.

It is anticipated that the measures outlined above will prove adequate for erosion and sediment control. Site inspection personnel will have the authority based on the Contract Documents to require additional control measures as necessary should the contractor's operations result in soil tracking or other offsite transfer of sediment and soil.

4.22 Identification of Floodplains

There are no designated floodplains on the site of this development.

4.23 Fill Constraints

There are no fill constraints applicable to this site related to any floodplain. In general, the site grading has been designed to match existing grading where possible, and no significant grade raises are proposed.

5. APPROVAL AND PERMIT REQUIREMENTS

The proposed development is subject to site plan approval and building permit approval.

No approvals related to municipal drains are required.

No permits or approvals are anticipated to be required for the School Board from the Ontario Ministry of Transportation, National Capital Commission, Parks Canada, Public Works and Government Services Canada, or any other provincial or federal regulatory agency.

6. CONCLUSION CHECKLIST

6.1 Conclusions and Recommendations

It is concluded that the proposed development can meet all provided servicing constraints and associated requirements. It is recommended that this report be submitted to the City of Ottawa in support of the application for site plan approval.

6.2 Comments Received from Review Agencies

No comments pertaining to the completed design have been received to date.

6.3 Signature and Professional Stamp

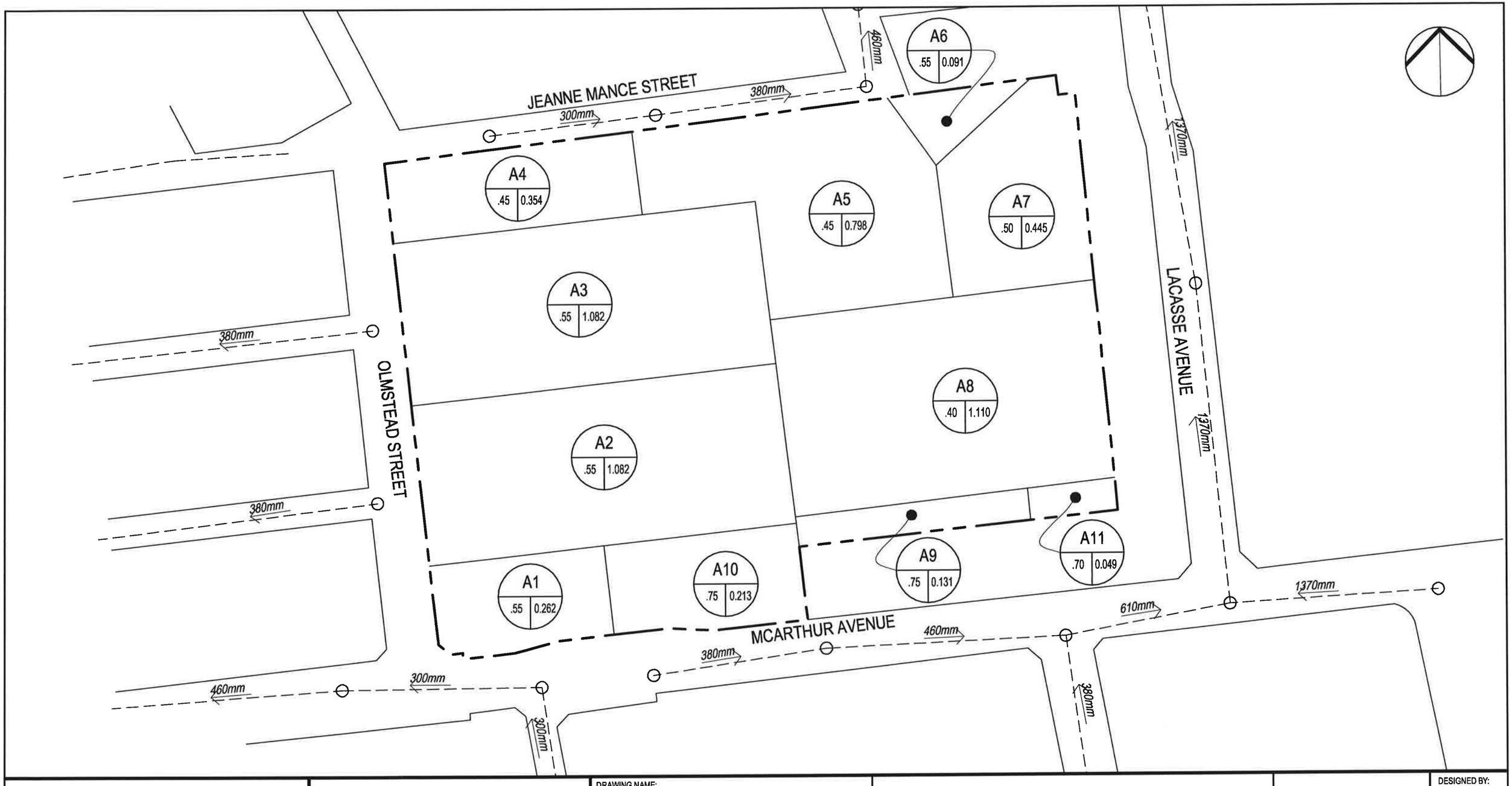
Report prepared by: WSP James C. Johnston, P.Eng. 2611 Queensview Drive, Suite 300 Ottawa, Ontario K2B 8K2	

APPENDIX

DRAWING SKC1 – DRAINAGE AREAS AND RUNOFF COEFFICIENTS BASED ON MUNICIPAL SEWER DESIGN

DRAWING SKC2 – NEW SCHOOL – ROOF PONDING

STORM SEWER DESIGN SHEETS



WSP
2611 QUEENSVIEW DRIVE, SUITE 300
OTTAWA (ONTARIO) CANADA K2B 8K2
TELEPHONE: (613) 829-2800
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PROJECT: HORIZON-JEUNESSE

ADDRESS: 349 OLMSTEAD ST. OTTAWA, ON

PROJECT NO.: 151-08425-00

DRAWING NAME: DRAINAGE AREAS AND RUNOFF COEFFICIENTS BASED ON MUNICIPAL SEWER DESIGN

DATE: APRIL 29, 2016

SCALE: 1 : 1500

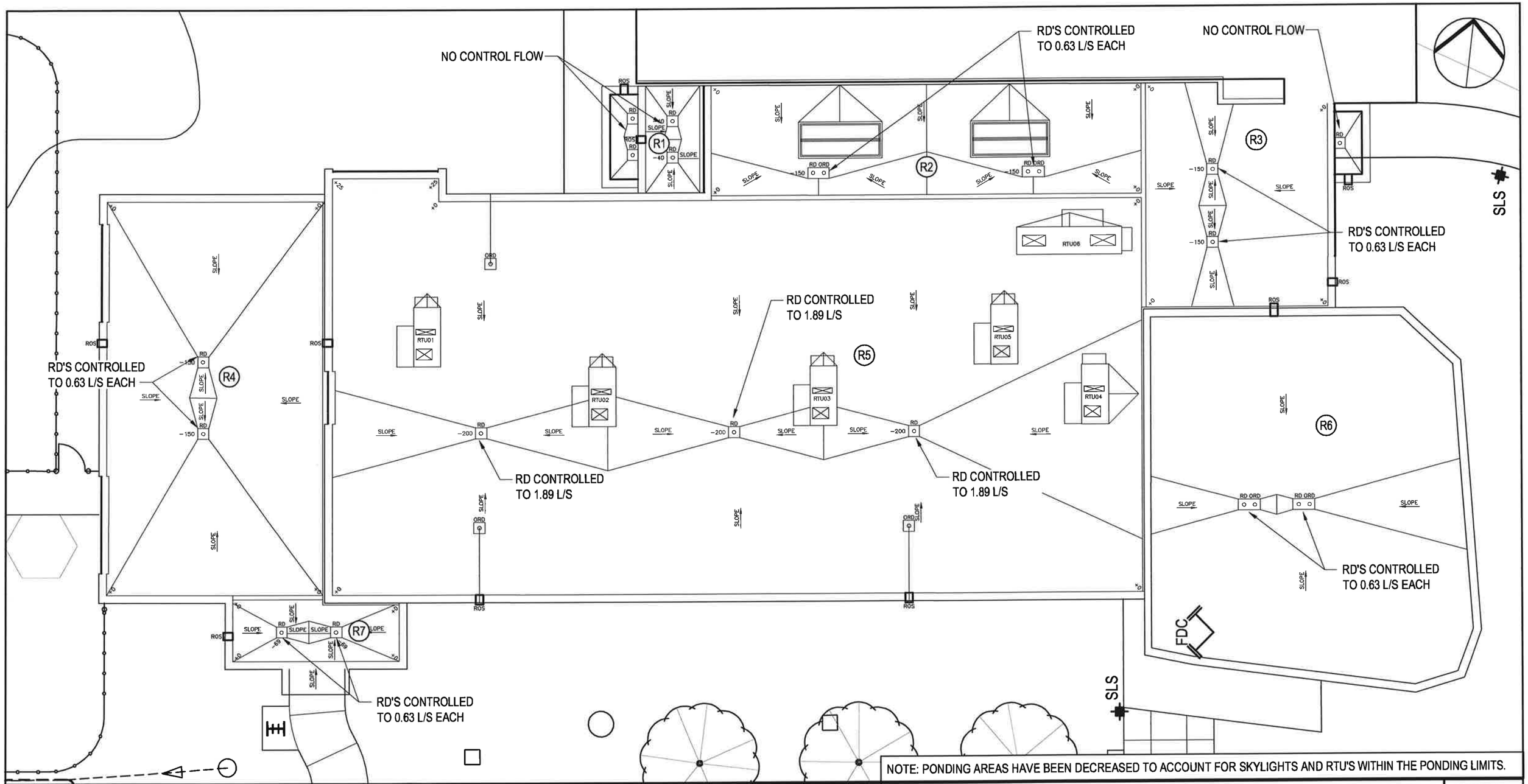
REVIEWED BY: J.J

A0 (.00 | 0.000)
- AREA No.
- AREA (HECTARES)
- ALLOWED RUNOFF COEFFICIENT

DESIGNED BY: J.J

DRAWN BY: B.N.

SHEET: SKC1



NOTE: PONDING AREAS HAVE BEEN DECREASED TO ACCOUNT FOR SKYLIGHTS AND RTU'S WITHIN THE PONDING LIMITS.

WSP

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PROJECT: HORIZON-JEUNESSE

ADDRESS: 349 OLMSTEAD ST. OTTAWA, ON

PROJECT NO.: 151-08425-00

DRAWING NAME: NEW SCHOOL - ROOF PONDING

DATE: MAY 13, 2016

SCALE: 1 : 250

REVIEWED BY: J.J

ROOF PONDING TABLE

ROOF ZONE	CONTRIBUTING AREA (m²)	NO. OF DRAINS	PONDING AREA (m²)	PONDING DEPTH (m)	AVAILABLE VOLUME (m³)
R1	23	2	17	0.040	0.23
R2	152	2	117	0.150	5.85
R3	125	2	116	0.150	5.80
R4	270	2	254	0.150	12.70
R5	998	3	607	0.150	30.35
R6	353	2	327	0.150	16.35
R7	35	2	30	0.040	0.40

DESIGNED BY: J.J

DRAWN BY: B.N.

SHEET: SKC2

WSP

Storm Sewer Calculation Sheet (Rational Method) – Flow to Jeanne Mance and McArthur

Sub-Area	LOCATION		AREA (Ha)			FLOW					Dia. (mm)	Design Slope (%)	Design Length (m)	Capacity (L/s)	Velocity (m/s)	Time of Flow (min.)	Ratio Q/Q full	Note
	From Node	To Node	R = 0.9	R = 0.25	R = 0.7	Local 2.78 AC	Accum. 2.78 AC	Time of Conc.	Rainfall Intensity	Peak Flow Q (L/s)								
29	CB1	CBMH10	.0012	.0260	-	.0211	.0211	10	104.4	2.2	200	1.0	10	34.2	1.06	.16	.06	
28	CBMH10	STMH4	.0383	.0132	-	.1050	.1261	10.16	103.4	13.0	200	0.58	11	26.1	0.80	.23	.50	
30	CB2	Main	.0300	.0156	-	.0859	.0859	10	104.4	9.0	200	1.0	9.5	34.2	1.06	.15	.26	
31	CB3	Main	.0433	.0106	-	.1157	.1157	10	104.4	12.1	200	1.0	10.5	34.2	1.06	.17	.35	
-	STMH4	STMH3	-	-	-	-	.3277	10.39	102.2	33.5	300	0.7	32.5	84.4	1.16	.47	.40	
-	STMH3	CBMH2	-	-	-	-	.3277	10.86	99.9	32.7	300	0.5	36	71.3	0.98	.61	.46	
27	CBMH12	CBMH2	.0086	.1907	-	.1541	.1541	10	104.4	16.1	300	0.5	50.5	71.3	0.98	.86	.26	
32	CB20	CBMH2	.0410	.1933	-	.2369	.2369	10	104.4	24.7	200	1.0	3.5	34.2	1.06	.06	.72	
-	CBMH2	CBMH1	-	-	-	-	.7187	11.47	97.0	69.7	375	0.8	38.5	90.2	1.24	.52	.77	
12	CB21	CBMH1	-	.1497	-	.1040	.1040	10	104.4	10.9	200	1.0	4.5	34.2	1.06	.07	.32	
10	CB22	CBMH1	.0332	.1738	-	.2039	.2039	10	104.4	21.3	250	0.8	27.5	55.5	1.10	.42	.38	
-	CBMH1	CBMHEX3	-	-	-	-	1.0266	11.99	94.7	97.2	375	0.6	32	141.7	1.24	.43	.69	
11	CB19	CBMHEX3	.0415	.2056	-	.2467	.2467	10	104.4	25.8	200	1.0	15.5	34.2	1.06	.24	.75	
-	CBMHEX3	City	-	-	-	-	1.2733	12.42	92.9	118.3*	375	1.0	16	182.9	1.60	.16	.65	Existing
34	MHSTEX2	City	.0410	1.8177	-	1.3659	1.3659	20	70.25	96.0	300	.59	22	77.5	1.06	.35	1.24	Existing
4	CBEX16	City	.0014	.0487	-	.0373	.0373	10	104.4	3.9	200	6.0	13.5	83.8	2.58	.09	.05	Existing
19/21A	McArthur	West	.3009	.2892	-	.9538	.9538	10	104.4	99.6	250	2.8	25	103.5	2.0	.21	.96	Existing
21B/33	McArthur	East	.5278	.0827	-	1.3780	1.3780	10	104.4	143.9**	200	1.0	25	34.2	1.06	.39	2.1	Existing
35	McArthur	East	-	.0069	-	.0048	.0048	10	104.4	0.5	-							Existing overland.

Definitions:
Q = 2.78 AIR, where
Q = peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Notes:
* Controlled flow of 46.7 L/s.
**Partially controlled to 72.9 L/s.

Time of concentration for existing McArthur storm sewer outlets was arbitrarily assumed at 10 minutes.

Designed: JCJ

PROJECT: École Horizon-Jeunesse

LOCATION: 349 Olmstead Street, Ottawa, ON

File Ref.: 151-08425-00

Date: May 16, 2016

WSP

Storm Sewer Calculation Sheet (Rational Method) – Flow to Olmstead

Sub-Area	LOCATION		AREA (Ha)			FLOW												Note
	From Node	To Node	R = 0.9	R = 0.25	R = 0.7	Local 2.78 AC	Accum. 2.78 AC	Time of Conc.	Rainfall Intensity	Peak Flow Q (L/s)	Dia. (mm)	Design Slope (%)	Design Length (m)	Capacity (L/s)	Velocity (m/s)	Time of Flow (min.)	Ratio Q/Q full	
20	Building	STMH11	.1956	-	-	.4894	.4894	10	104.4	51.1***			6.0					
17	CB14	STMH11	.0027	.0297	-	.0274	.0274	10	104.4	2.9	200	1.0	12.0	34.2	1.06	.19	.08	
16	CB13	STMH11	.0016	.0184	-	.0168	.0168	10	104.4	1.8	200	1.0	7.0	34.2	1.06	.11	.05	
-	STMH11	City	-	-	-	-	.5336	10	104.4	55.7	300	0.34	25	58.4	0.80	.39	.95	Gladu
18	Overland		.0063	.0276	-	.0349	.0349	10	104.4	3.6	-	-	-	-	-	-	-	
36	Overland		.0112	-	-	.0280	.0280	10	104.4	2.9	-	-	-	-	-	-	-	
26	CB4	CBMH5	.0365	.0012	-	.0922	.0922	10	104.4	9.6	200	1.0	13	34.2	1.06	.20	.28	
22	DCB5	CBMH5	.0784	.0144	-	.2062	.2062	10	104.4	21.5	250	1.0	13	62.0	1.22	.18	.35	
25	CBMH5	CBMH6	.0284	.0018	-	.0723	.3707	10.2	102.8	38.2	300	0.34	27.5	58.4	0.8	.57	.65	
9	CB6	CBMH6	.0525	.0417	.0648	.2864	.2864	10	104.4	29.9	250	1.0	7	62.0	1.22	.10	.48	
24	CB7	Main	.0522	.0253	-	.1482	.1482	10	104.4	15.5	200	1.0	11	34.2	1.06	.17	.45	
8	CB8	Main	.0150	.0698	.0051	.0960	.0960	10	104.4	10.0	200	1.0	13	34.2	1.06	.20	.29	
23	CB9	Main	.0273	.0325	-	.0909	.0909	10	104.4	9.5	200	1.0	7	34.2	1.06	.11	.28	
14	CB10	Main	.0340	-	-	.0851	.0851	10	104.4	8.9	200	1.0	3	34.2	1.06	.05	.26	
-	CBMH6	STMH7	-	-	-	-	1.0773	10.77	100.3	108.1	300	1.2	49.5	109.2	1.50	.55	.99	
3	CB17	CBMH9	.0597	.0022	-	.1509	.1509	10	104.4	15.8	200	1.0	14	34.2	1.06	.22	.46	
1	CB18	CBMH9	.0409	.0261	-	.1205	.1205	10	104.4	12.6	200	1.0	17	34.2	1.06	.27	.37	
2	CBMH9	CBMH8	.0484	.0174	-	.1332	.4046	10.27	102.8	41.6	300	0.34	20.5	58.4	0.8	.43	.71	
7	CB16	CBMH8	.0358	-	-	.0896	.0896	10	104.4	9.4	200	1.0	14	34.2	1.06	.22	.27	
5	CB15	Main	.0388	.0021	-	.0985	.0985	10	104.4	10.3	200	1.0	15.5	34.2	1.06	.24	.30	
6	CBMH8	STMH7	.0387	-	-	.0968	.6895	10.7	100.6	69.4	300	0.5	18.5	71.3	0.98	.31	.97	
13	CB11	Main	.0273	-	-	.0683	.0683	10	104.4	7.1	200	1.0	6	34.2	1.06	.09	.21	
15	CB12	Main	.0221	.0534	-	.0924	.0924	10	104.4	9.6	200	1.0	14.5	34.2	1.06	.23	.28	
-	STMH7	STMH13	-	-	-	-	1.9275	11.32	97.7	188.3	300	3.5	20	188.7	2.59	.13	1.0	
-	STMH13	City	-	-	-	-	1.9275	11.45	97.1	187.2****	300	3.8	15	196.5	2.70	.09	.95	Maple

Definitions:
Q = 2.78 AIR, where
Q = peak Flow in Litres per second (L/s)
A = Areas in hectares (ha)
I = Rainfall Intensity (mm/h)
R = Runoff Coefficient

Notes:
*** Controlled flow of 13.1 L/s.
****Controlled flow of 66 L/s.

Designed: JCJ

PROJECT: École Horizon-Jeunesse
LOCATION: 349 Olmstead Street, Ottawa, ON
File Ref.: 151-08425-00
Date: May 9, 2016
Sheet No. 2 of 2