SERVICING AND STORMWATER MANAGEMENT REPORT
École Secondaire Catholique Paul-Desmarais Dome

5315 Abbott 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 C001)

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Revision 3 – July 20, 2018

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1. GENERAL

1.1 Executive Summary

This report outlines site servicing criteria and civil engineering calculations pertaining to the servicing of a proposed sports dome and associated pavilion at an existing secondary school constructed in 2015/2016. The report also includes information from the servicing report prepared in 2014 for the original site development. The site is 6 hectares in size, with the present school building located in the southeast corner of the site. The proposed dome facility will be constructed in the northwest corner of the site, north of the existing bus loop, on land presently partly occupied by the main sports field for the school. The site is located within the proposed Fernbank community, located near the eastern limit of the existing community of Stittsville.

Abbott Street was extended to the site in 2015. Water and sanitary mains presently exist within the Abbott Street right of way, and the existing school building is connected to these services. The existing site water service will be extended to service the pavilion, the dome and a new private hydrant. The sanitary service for the pavilion will require pumping. A forcemain will carry sanitary sewage from the pavilion to a manhole near the site driveway entrance, and a gravity sewer will convey the sewage to the existing sanitary manhole SANMH2 at the south east corner of the property. A piped storm drainage outlet on the east side of the site discharges to a temporary ditch within the right of way of the future Robert Grant Avenue, on route to off-site interim stormwater quantity and quality control facilities designed for the use of the school site. The interim facility will be replaced in the future by a larger communal stormwater treatment facility, which will service the school site as well as other properties within the Fernbank community. The developer is responsible for obtaining all required regulatory approvals associated with these facilities.

Modifications will be made to the existing on-site storm sewer system and storm water management system in order to accept the increase in impervious surfaces arising from the dome construction. No change will be made to the storm outlet or allowable flow release rate.

The proposed grading and servicing for the site are shown on civil drawings C002 to C005. Drawing C001 provides a site drainage area plan, sediment and erosion control, and related engineering notes and details.

This report was prepared utilizing servicing design criteria obtained from the original site development, the City of Ottawa, and Novatech Engineering Consultants Ltd. (the consultant for the community developer), and outlines the design for water, sanitary wastewater, and stormwater facilities, including stormwater management.

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 servicing report is the third revision, incorporating city review comments and site plan modifications, and is dated July 20, 2018.

1.3 Location Map and Plan

The civil engineering drawings C001 to C005 include the municipal address, site boundary, and site layout for grading and servicing. A location plan is shown on Drawing C001. The architectural site plan provides a detailed description of the site layout.

1.4 Adherence to Zoning and Related Requirements

The property and project will be in conformance with zoning and related requirements, subject to confirmation by the City of Ottawa. The dome facility is considered a Recreation and Athletic Facility, which is an acceptable use under the current zoning.

1.5 Pre-Consultation Meetings

A pre-consultation meeting for the dome project were held with representatives of the City of Ottawa, Conseil des Écoles Catholique du Centre-Est, and the consultant design team on February 3, 2017.

1.6 Higher Level Studies

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

- Servicing design information provided by Novatech Engineering, which is based on the Master Servicing Study for the Fernbank Community Design Plan, June 2009. Confirmation was received from Mark Bissett of Novatech Engineering in October 2017 that the water and sanitary sewage demands for the proposed dome project will not have an adverse impact on the neighbourhood servicing design.

- Ottawa Sewer Design Guidelines, October 2012.

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

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 with the dome project, while adhering to the stipulations of the applicable higher level studies and City of Ottawa servicing design guidelines. The site plan includes allowances for additional parking, expansions to the school building, and a new bus loop from the future street on the east side of the site. The servicing design has allowed for these future site plan modifications, although the presented storm drainage network and ponding is based on the interim condition.

1.8 Available Existing and Proposed Infrastructure

The existing services for the present school will not be altered. The storm sewer network in the north part of the size will be changed to allow for the dome, but the outlet 975mm diameter storm sewer will not be changed. The existing 150mm water service will be replaced with a 200mm private watermain from the south side of the existing building (at the site water entry point) to the new pavilion and dome, and will service a new private hydrant and a 100mm diameter service to the building. Due to the distance of the pavilion from the present site sanitary outlet, a combination forcemain and gravity sewer will be provided to convey sanitary sewage from the pavilion to an existing private sanitary manhole at the southeast corner of the site. This manhole discharges to the sanitary trunk sewer on Abbott Street. Off-site facilities have been provided by the developer for stormwater quantity and quality control, and for conveyance of the stormwater. Presently, the 975 mm discharge pipe outlets to a constructed channel on the future Robert Grant Avenue. A storm sewer on this street is expected in the future. Stormwater quantity control is required on the site, and detention storage will be reconfigured and increased to account for the dome project.

Site access is presently from Abbott Street. A future bus loop connection off of Robert Grant Avenue is anticipated on the site plan, but is not currently scheduled for construction. Additional catch basins and sewer modifications are expected to be necessary for this future change.

1.9 Environmentally Significant Areas, Watercourses and Municipal Drains

The proposed changes to the site will not require any additional approvals or amendments to approvals pertaining to environmentally significant areas, watercourses or municipal drains.

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. The existing and proposed grading are shown on Drawing C002 - Grading Plan. Existing grading information is based on a topographic survey of the site completed in 2017. No changes in grading are proposed beyond the site boundaries, and in the vicinity of the existing City storm sewer on the west side of the site. The proposed grading plan confirms the feasibility of the proposed stormwater management system, drainage, soil removal and fills.

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

No scheduled development phasing has been detailed for the site. The site plan does indicate possible future development of portable classrooms, a replacement bus loop and building expansions. These additional impervious areas have been taken into account in the stormwater management calculations, assuming the worst case scenario that all future building additions, bus loop, full parking and eight portable classrooms will be in place. During the interim, higher numbers of portable classrooms are expected as noted on the site plan, but these numbers would be reduced when the building expansion takes place. Historically, this School Board has experienced substantial growth at their school sites, and inclusion of larger amounts of potential impervious area is considered a reasonable precaution.

1.13 Geotechnical Study

A geotechnical investigation report was prepared by exp Services Inc. for the original school construction, and a new report has been prepared in 2017 for the dome project. No additional geotechnical information was required for the design of the modified site services, including paving. This geotechnical reports will be included with the contract documents to be issued for construction, and the recommendations of the reports will be referenced in the construction specifications. Flexible joints on piped services at the building walls have also been noted on the civil engineering drawings to allow for possible differential settlement.

1.14 Drawing Requirement

The submitted Site Plan from Edward J. Cuhaci and Associates 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 amendment.

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, either existing or as proposed in the Fernbank Community Design Plan – Master Servicing Study, to allow for water servicing for this property. The 400 mm watermain which services the property, including the proposed buildings, is already in place.

An existing 200mm watermain extends to the property from Abbott Street in the southeast corner of the site, and supplies a private hydrant in that area. The existing school building has a

150mm diameter water service, with a water entry room in the southwest corner. The 150mm water service will be replaced with a 200mm private watermain in order to convey the required fire flow to the pavilion.

There is an option to supply the domestic water needs of the pavilion directly from the existing school water entry room, but this would require both a 75mm diameter domestic supply line and a 150mm private watermain being extended to the pavilion area. It is not anticipated that this arrangement would provide sufficient flow. The proposal is therefore to supply both the domestic supply and fire fighting requirements from the 200mm diameter proposed private watermain extension.

2.2 System Constraints and Boundary Conditions

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

Available system conditions are established based on hydraulic head information provided by the City of Ottawa, as indicated in Appendix D. Hydraulic head values were provided as follows: Peak Hour = 155.6m, Max HGL = 161.3m, and Max Day + Fire = 155.6m. The finished ground elevation at the existing 400mm municipal watermain connection is approximately 104.7m, which at a watermain burial depth of 2.4m results in an elevation head of 102.3m. This results in pressure heads between 53.3m and 59.0m. The proposed floor of the pavilion is at an elevation of 104.8m, and therefore will operate under similar conditions to the present school.

Static pressures in the range of 76 to 84 psi can therefore be expected under the operating heads described above. Minimal head loss will occur in the proposed 200mm private watermain extension due to the relatively low water demands of the pavilion.

2.3 Confirmation of Adequate Domestic Supply and Pressure

The mechanical engineer has suggested a peak domestic water demand of 86 USgpm or 5.4 L/s for the pavilion building. The estimated demand for the school building based on the ultimate demand was 12.9 L/s, but based on 2 years of City of Ottawa water bills provided by the school board, the maximum day demand from the school is only 0.32 L/s. Both demands are relatively low considering the size of the site, and are not expected to be concurrent. The dome will be used during school hours for students, and therefore will not result in an increased population or demand during the daytime. During evening hours and weekends, the dome will be occupied, but the school will generally be empty or used minimally.

The Master Servicing Study was completed for the Fernbank Community Design Plan allowing for the development of a secondary school for the subject parcel of land. The Study did not require any modification in the size of the watermain on Abbott Street even at the ultimate development of the community. It is therefore reasonable to assume that the watermain can supply the expected demand for the school and the dome facility, both of which will be constructed early in the development of the community.

2.4 Confirmation of Adequate Fire Flow Protection

The fire demand for the pavilion is calculated in this report based on the Fire Underwriter's Survey method for a 581 m^2 building of combustible construction without a sprinkler system. A standpipe system will be provided for the pavilion. A dry standpipe system with freestanding fire cabinets will be provided for the dome.

For the existing school building, a peak fire demand of 23.65 L/s was estimated by the mechanical engineer using OBC requirements. Using the Fire Underwriter's Survey (FUS) method, a recommended fire demand of 9,000 L/min was estimated for the school building in the original site servicing report.

The pavilion will be a single storey structure, of wood frame (combustible) construction, with low fire hazard contents and no sprinkler system. The estimated fire demand using the FUS method is 8,000 L/min (133 L/s), assuming a 20% increase for the proximity of the dome. Calculations are provided in Appendix D. The proposed watermain network was modeled using EPANET software to confirm the system could deliver 8,000 L/min to the new building area. A printout from the model is also provided in Appendix D.

The dome structure will be serviced by an exterior dry standpipe system as determined by the mechanical designer, with a demand of 500 USgpm (1890 L/min). As this demand is lower than the 8,000 L/min demand estimated in the previous paragraph, the demand for the dome structure is easily met by the proposed system.

Section 8.3.2 of the Master Servicing Study indicates that fire flows exceeding 217 L/s (13,020 L/min) are available from the proposed watermain network in the Fernbank community at all locations along the trunk watermain. The Abbott Street watermain is part of the proposed trunk system. Fire flows adequate to meet the FUS and the mechanical engineering calculations (based on Ontario Building Code requirements) are therefore available.

2.5 Check of High Pressures

Section 8.4 of the Master Servicing Study indicates that service areas within the Fernbank Community with ground elevations below 105.7 m will be susceptible to daily pressures exceeding 80 psi (550 kPa). The pavilion elevation is 104.8 m, and therefore pressures exceeding 80 psi can be anticipated, although the length of the private watermain will create some pressure loss. To allow for this condition, a pressure reducing valve is suggested for the mechanical design of the pavilion.

2.6 Water Quality Analysis

A water quality analysis was performed to demonstrate that the age of the water in the water service does not exceed 5 days. Based on the analysis using EPANET and the average day flows to the school and the new pavilion building, the age of the water is much less than the 5 day period. Results of this analysis can be found in Appendix D.

2.7 Phasing Constraints

No phasing constraints exist for the pavilion and dome project.

2.8 Reliability Requirements

The water distribution network for the community will be a looped system, allowing for flow to the site via multiple directions.

2.9 Need for Pressure Zone Boundary Modification

The School Board is not required to implement any modification in pressure zone boundaries.

2.10 Capability of Major Infrastructure to Supply Sufficient Water

The Master Servicing Study was developed assuming the development of a school on the subject site. The construction of the proposed dome and pavilion to not significantly alter the anticipated water demand. As acknowledged by Novatech Engineering, the existing water distribution system is adequate for the site and will not be adversely impacted by the addition of the pavilion and dome.

2.11 Description of Proposed Water Distribution Network

The existing 150mm diameter private watermain serving the school will be replaced and extended using a 200mm diameter private watermain to service the pavilion, the dome and a new private hydrant located southeast of the pavilion.

The single private hydrant proposed meets Ontario Building Code requirements for offset and proximity to the building.

2.12 Off-site Requirements

No off-site improvements to watermains, feedermains, pumping stations, or other water infrastructure are required to service the project. The Master Servicing Study outlines off-site requirements for the future development of the Fernbank Community.

2.13 Calculation of Water Demands

Water demand calculations are provided in Sections 2.3 and 2.4 above.

2.14 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 that sanitary sewers be designed using a sanitary flow allowance of 50,000 L/ha/day for institutional uses, with a peaking factor of 1.5. The area of the building site is 6 ha. The peak flow allowed for the site calculated using the guidelines is therefore 5.21 L/s. The extraneous flow allowance is 0.28 L/s/ha, raising the peak estimated allowable flow to 6.89 L/s.

The Ottawa Sewer Design Guidelines also provide estimates of sewage flows based on per capita unit rates. The anticipated average flow based on the estimated ultimate population of the school of 1307 persons (at an average rate of 90 L/person/day) is 1.36 L/s. The per capita value is based on a day school containing a gymnasium with showers, and a cafeteria. Applying the peaking factor of 1.5, and adding the extraneous flow, the estimated ultimate peak flow is 3.72 L/s based on the building population. This value is lower than the sewer network design value of 6.89 L/s.

The mechanical engineer has estimated a total of 140 fixture units for the pavilion, which equates to a peak flow of approximately 4.4 L/s using plumbing calculations. As the peak flows for the school and pavilion will not be concurrent, the overall demand is anticipated to be within the parameters allowed for the site.

3.2 Consistency with Master Servicing Study

The use of the subject property for a secondary school was included in the Fernbank Community Design Plan Master Servicing Study. Sanitary sewage from the site will be conveyed to the existing Stittsville Trunk Sewer, which travels easterly along the Abbott Road corridor, and discharges to the Hazeldean Pumping Station, located east of Terry Fox Drive. A new trunk sewer (Fernbank CDP Trunk) will be constructed in the same corridor to augment the capacity of the Stittsville Trunk Sewer, with the new sewer being the outlet for virtually all of the Fernbank area. Some upgrades will be required at the Hazeldean Pumping Station to support the full development of the area.

The MSS indicates that the Stittsville Trunk Sewer is at capacity west of Iber Road, but has some residual capacity in the lower reaches. The minor amount of flow introduced by the school will not have a significant impact on the capacity of this existing 750 mm diameter pipe. The MSS does not specifically indicate that the school lands at 5315 Abbott Street will discharge into the Stittsville Trunk, as this decision was made at a later date by Novatech Engineering Consultants to permit servicing of the school in advance of the construction of the Fernbank CDP Trunk Sewer.

The existing sanitary service from the site is a 250 mm diameter sewer at a slope of 2%. This size and slope of sewer provides a capacity of 87.7 L/s. The sanitary service from the pavilion will be added to this existing outlet. No new connections are proposed to the 750mm diameter trunk sewer.

3.3 Review of Soil Conditions

Soil conditions have been reviewed by exp Services Inc. The geotechnical report indicated that the soil type on site is predominantly wet silty clay, which is susceptible to consolidation and settlement, especially in the event of seismic activity. A flexible pipe joint will be used at the building service connection in order to ensure the sanitary service pipe does not break in the event of settlement.

Bedding and backfill will be provided as recommended, conventional sewer materials will be utilized, and dewatering will be undertaken as necessary in accordance with the geotechnical recommendations and conditions encountered. The lowest top of pipe level for the proposed 100mm forcemain from the pavilion is 101.28m. The geotechnical report indicates that groundwater table was observed to be between 100.8 and 101.8 m. It is therefore expected that the groundwater impact on the forcemain and sewer construction will be minimal. No groundwater control issues were encountered during the construction of the existing development.

3.4 Description of Existing Sanitary Sewer

The outlet sanitary sewer is a 250mm diameter PVC sewer located off of Abbott Street.

3.5 Verification of Available Capacity in Downstream Sewer

The 250mm diameter sanitary sewer provided to the site by the developer has a capacity of 87.7 L/s, which is well in excess of the anticipated peak flows calculated by the methods outlined above. As noted in earlier sections, the receiving Stittsville Trunk Sewer has been deemed to have some additional capacity within this reach, and the flow from this site will have an insignificant impact on this sewer.

3.6 Calculations for Sanitary Sewers

The peak sanitary flow from the pavilion is estimated as 4.4 L/s. The required pumping station is included within the building as part of the mechanical design. A 75mm diameter forcemain, and 150mm diameter gravity sewer, with the latter having slopes of 0.9% and 1.55% in its two segments, will have adequate capacity to convey the pumped flow to the 250mm sanitary outlet sewer.

3.7 Description of Proposed Sewer Network

In addition to the piping described in Section 3.6, two additional 1200mm diameter sanitary manholes, designated as SANMH17-1 and 17-2, will be provided on the new 150mm gravity sewer. A new inlet will be required to existing SANMH2 at the southeast corner of the site.

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, other than as part of the plumbing system for the proposed pavilion. As noted in the Master Servicing Study, some upgrades will be required to the Hazeldean Pumping Station at later stages of development of the Fernbank community.

3.10 Force-Mains

A 75mm diameter force-main is proposed between the new pavilion and new SANMH17-2. This forcemain will generally follow a vertical alignment based on maintaining 2 metres of cover. No changes in existing downstream forcemains are required specifically for the proposed additional development on this site.

3.11 Emergency Overflows from Sanitary Pumping Stations

The small sanitary pumping facility proposed for the pavilion building will be a duplex system, with backup power. In the event of failure of the pump station and/or the primary and backup power systems, the facility will be shut down until repairs can be made. No provision is therefore necessary for emergency overflows.

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. Clay dykes in service trenches, flexible joints at structures, and specific bedding requirements related to soil conditions have been addressed in the design notes and plan on Drawing C003.

4. STORMWATER SERVICING

4.1 Description of Drainage Outlets and Downstream Constraints

The existing piped drainage outlet from the site is a 975 mm diameter storm sewer on the east boundary of the site. The sewer discharges to an interim channel and treatment facility off-site. In the future, it is anticipated that the sewer may discharge to a municipal sewer on Robert Grant Avenue.

The allowable flow release from the site has been set at 850 L/s, and remains unchanged from the existing condition.

Flows exceeding 850 L/s up to the 100 year storm have to be temporarily detained on site and released at a rate not exceeding 850 L/s.

The geotechnical investigation determined that the water table on the site ranged in elevation between 100.8 m and 101.8 m. All proposed subdrains are above 101.8 m. The entire storm sewer network is higher than 100.8 m, but several segments have pipe inverts below 101.8 m. There is the possibility of some groundwater infiltration into the storm sewer network. Standard sewer design methods use an infiltration allowance of 0.28 L/s/ha, which when used for this 6 ha site, would result in a design allowance of 1.68 L/s. As noted on the storm sewer design table in Appendix A, the 5 year design flow for the sewer network is 837.7 L/s. Infiltration is therefore not anticipated to be a significant proportion of the allowable release rate of 850 L/s.

4.2 Analysis of Available Capacity in Existing Public Infrastructure

As the allowable release rate from the site will be unchanged, and was determined in conjunction with the design of the public infrastructure, there are no concerns related to the adequacy and available capacity of the downstream network.

4.3 Drainage Drawing

Drawing C002 provides proposed grading and drainage, and includes existing grading information. Drawing C003 illustrates sections of the existing site storm sewer network that are being removed. Drawing C004 indicates the proposed new sections of the storm sewer network. A drainage sub-area plan is provided on Drawing C001, with a breakdown of subarea information, based on the proposed site plan changes. Sub-area information is also provided on the storm sewer design sheet attached to this report in Appendix A.

4.4 Water Quantity Control Objective

The water quantity objective for the entire site is to limit the flow release to 850 L/s.

Stormwater storage calculations are shown in Section 4.10 of this report. Detention stormwater storage is presently provided on the school roof, and is not being changed in this present site plan amendment. No new additional roof storage is proposed. Ground surface storage areas provided in the original design have been modified to accommodate the increased flow rate generated by the new impervious surfaces.

No quantity control is required on the 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 from the site at the northeast corner, consistent with existing conditions.

4.5 Water Quality Control Objective

As established in the original design, stormwater quality control treatment is required for the site based on correspondence provided by MVCA. The required quality treatment is provided in an off-site facility provided by the developer.

4.6 Description of Stormwater Management Concept

The drainage system for the site consists of a series of catch basins, manholes, catch basin manholes, and storm sewers leading to the 975 mm outlet sewer.

The existing school roof is provided with 30 controlled flow roof drains, generating a flow of 1.9 L/s per drain at the maximum storage depth of 150 mm on the roof. The release rate of 1.9 L/s is a design characteristic of the type of roof drain specified. The estimated storage calculations are provided in Section 4.10.

Ground level surface ponding will be provided in 6 ground surface areas, controlled by a flow regulator at the outlet sewer. The maximum depth of surface ponding will be 300 mm. The ponding limits, areas, depths and volumes are noted on Drawing C002.

Calculations for the storage requirements and the outlet flow regulator are provided in subsequent sections.

4.7 Setback from Sewage Disposal Systems, Water Courses, and Hazard Lands

There are no required setbacks from sewage disposal systems, water courses or hazard lands that apply to works on the site.

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

As no changes will be made to off-site flow rates or infrastructure that requires MVCA approval, no pre-consultation has been initiated with the Mississippi Valley Conservation Authority for the proposed site plan amendment. A copy of the response provided by the MVCA for the original school development is provided in Appendix C to this report.

No pre-consultation with the Ottawa District office of the Ontario Ministry of the Environment and Climate Change has been initiated. The original site development did not require an Environmental Compliance Approval (ECA), and the changes being proposed also do not require an ECA.

4.9 Consistency with Higher Level Studies

The stormwater management design for the site is consistent with the requirements established at the time of the original site development. Quality control is provided off-site in the interim in a temporary pond servicing the school site, and at a later date in a communal pond downstream. The quality control design provided by the developer will be required to adhere to all applicable policies and guidelines of the Mississippi Valley Conservation Authority, the City of Ottawa, MOCC and other approvals agencies, as stated in Section 6.1.1 of the Fernbank Community Design Plan Master Servicing Study.

Community quantity control requirements are also listed in Section 6.1.1 of the MSS. The site specific requirements were provided in e-mail correspondence dated November 15, 2013 from

Novatech Engineering Consultants to GENIVAR (now WSP). That correspondence indicated an allowable stormwater release rate of 850 L/s, with storage required up to the 100 year event.

4.10 Storage Requirements and Conveyance Capacity

Detention stormwater storage is required on the site so that the discharge generated by the 6.0176 ha area up to the 100 year event does not exceed the allowable release rate of 850 L/s calculated for the site.

The ultimate development (including the future bus loop, portable classrooms and school additions, plus the proposed dome and pavilion) includes the following areas:

Paved surfaces, pathways and roof areas 3.4808 ha.

Landscaped surfaces 2.4552 ha.

Gravel surface 0.0816 ha for pathways. Interim gravel surfaced fire route will be removed in the ultimate development.

Assigning a runoff coefficient of 0.9 to the impervious areas, a coefficient of 0.7 for the gravel surfaces, and a coefficient of 0.25 to the landscaped area yields a weighted average runoff coefficient of 0.632 for the entire site. Under 100 year conditions, the coefficient for the pervious and gravel areas is increased by 25%, and the impervious area coefficient is increased to 1.0, yielding a weighted average runoff coefficient of 0.718.

The required volume of storage is calculated using the modified Rational Method as indicated in the following tables calculated for the drainage area of 6.0176 ha.

Flows are calculated using the Rational Method with 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

Required storage is calculated by determining the difference between actual and allowable flow rates for the site, and multiplying by the associated duration.

TABLE 4.1 100 YEAR STORAGE REQUIREMENTS

For 100 year storm event (C = 0.721 and area = 6.0176 ha)

Duration Minutes	Intensity mm/hr	Q L/s	Q Allowable L/s	Difference L/s	Storage m^3
5	242.6	2914	850	2064	619
10	179.0	2150	850	1300	780
15	146.8	1763	850	913	822
20	119.95	1441	850	591	709
25	103.85	1247	850	397	596
30	91.90	1104	850	254	457
35	82.58	992	850	142	298

A required volume of 822 m^3 is indicated for the 100 year event.

TABLE 4.2 5 YEAR STORAGE REQUIREMENTS

For 5 year storm event (C = 0.632 and area = 6.0176 ha)

Duration Minutes	Intensity mm/hr	Q L/s	Q allowed L/s	Difference L/s	Storage m^3
5	140.20	1482	850	632	190
10	104.40	1104	850	254	152
15	85.60	905	850	55	50

A required storage volume of 190 m^3 is indicated for the 5 year event.

Detention stormwater storage will be provided on-site using roof top and ground surface storage.

Storage on the school roof was calculated only for the initial phase of development. Storage on future roof expansion areas was not assumed. The roof is provided with 30 flow controlled roof drains, each delivering a maximum flow of 1.9 L/s at the maximum ponding depth of 150 mm.

The roof can be divided into 12 separate segments, with each segment being an independent surface. The locations of the roof areas are shown in Figure 1 in Appendix B, copied from the original site servicing report. The areas and number of drains associated with each of these segments is provided in the following table.

TABLE 4.3 ROOF STORAGE – AREA, DEPTH AND PHYSICAL VOLUME

Roof Segment	No. of Drains	Ponding Area (m^2)	Ponding Depth (m)	Theoretical Storage Volume (m^3)
R1	1	25	0.15	0.3
R2	2	349	0.15	8.7
R3	3	615	0.15	29.2
R4	2	249	0.15	11.8
R5	4	1402	0.15	66.6
R6	2	188	0.15	8.9
R7	2	873	0.15	41.5
R8	6	1240	0.15	52.9
R9	2	779	0.15	37.0
R10	2	806	0.15	38.3
R11	2	165	0.11	5.7
R12	2	160	0.14	7.1

The theoretical storage volume provided in the table above is based on the physical dimensions of the roof, and is calculated using the formula for an inverted pyramid (Volume = area x depth / 3). The areas shown above were further reduced by 5% to allow for roof top equipment displacing available storage, and for Segment R8, the area of the skylights (126 m^2) was also deleted. The 5% reduction is an arbitrary number, but was selected so that the volume of ponding available is not overestimated. The size of roof top equipment will vary, but not to the extent that it would exceed 5% of the roof space available. As will be noted below the physical storage available on the roof in almost all cases does not govern the storage available, so the 5% value is not of significance in estimating the storage available.

In determining the actual amount of storage that can be achieved, it is necessary to also complete a flow balance analysis, comparing incoming rainfall to the discharge rate from the drains. In some cases, the amount of runoff generated is less than the physical capacity of the storage volume available. The rainfall balance analysis is also completed using the Modified Rational Method, with the incoming flow calculated using the Rational Method, and the outgoing flow determined by the number of drains multiplied by a flow rate of 1.9 L/s per drain. The rainfall balance for each of the roof segments is provided in the tables below for the 100 year condition.

TABLE 4.4 ROOF SEGMENT R1 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	1.7	1.9	0	0
10	179.0	1.2	1.9	0	0

TABLE 4.5 ROOF SEGMENT R2 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	23.5	3.8	19.7	5.9
10	179.0	17.4	3.8	13.6	8.2
15	146.8	14.2	3.8	10.4	9.4
20	119.95	11.6	3.8	7.8	9.4
25	103.85	10.1	3.8	6.3	9.4
30	91.90	8.9	3.8	5.1	9.2
35	82.58	8.0	3.8	4.2	8.8

TABLE 4.6 ROOF SEGMENT R3 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	41.5	5.7	35.8	10.7
10	179.0	30.6	5.7	24.9	14.9
15	146.8	25.1	5.7	19.4	17.5
20	119.95	20.5	5.7	14.8	17.8
25	103.85	17.8	5.7	12.1	18.2
30	91.90	15.7	5.7	10.0	18.0
35	82.58	14.1	5.7	8.4	17.6

TABLE 4.7 ROOF SEGMENT R4 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	16.8	3.8	13.0	3.9
10	179.0	12.4	3.8	8.6	5.2
15	146.8	10.2	3.8	6.4	5.8
20	119.95	8.3	3.8	4.5	5.4
25	103.85	7.2	3.8	3.4	5.1
30	91.90	6.4	3.8	2.6	4.7
35	82.58	5.7	3.8	1.9	4.0

TABLE 4.8 ROOF SEGMENT R5 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	94.6	7.6	87.0	26.1
10	179.0	69.8	7.6	62.2	37.3
15	146.8	57.2	7.6	49.6	44.6
20	119.95	46.8	7.6	39.2	47.0
25	103.85	40.6	7.6	33.0	49.5
30	91.90	36.0	7.6	28.4	51.1
35	82.58	32.3	7.6	24.7	51.9
40	75.15	29.4	7.6	21.8	52.3
45	69.05	27.0	7.6	19.4	52.4
50	63.95	25.0	7.6	17.4	52.2
55	59.62	23.3	7.6	15.7	51.8
60	53.20	20.7	7.6	13.1	47.2

TABLE 4.9 ROOF SEGMENT R6 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	12.7	3.8	8.9	2.7
10	179.0	9.4	3.8	5.6	3.4
15	146.8	7.7	3.8	3.9	3.5
20	119.95	6.3	3.8	2.5	3.0
25	103.85	5.4	3.8	1.6	2.4
30	91.90	4.8	3.8	1.0	1.8
35	82.58	4.3	3.8	0.5	1.2

TABLE 4.10 ROOF SEGMENT R7 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	58.8	3.8	55.0	16.5
10	179.0	43.4	3.8	39.6	23.8
15	146.8	35.6	3.8	31.8	28.6
20	119.95	29.1	3.8	25.3	30.4
25	103.85	25.2	3.8	21.4	32.1
30	91.90	22.3	3.8	18.5	33.3
35	82.58	20.0	3.8	16.2	34.0
40	75.15	18.2	3.8	14.4	34.6
45	69.05	16.8	3.8	13.0	35.1
50	63.95	15.5	3.8	11.7	35.1
55	59.62	14.5	3.8	10.7	35.3
60	53.20	12.9	3.8	9.1	32.8

TABLE 4.11 ROOF SEGMENT R8 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	83.6	11.4	72.2	21.7
10	179.0	61.7	11.4	50.3	30.2
15	146.8	50.6	11.4	39.2	35.3
20	119.95	41.3	11.4	29.9	35.9
25	103.85	35.8	11.4	24.4	36.6
30	91.90	31.7	11.4	20.3	36.5
35	82.58	28.5	11.4	17.1	35.9

TABLE 4.12 ROOF SEGMENT R9 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	52.5	3.8	48.7	14.6
10	179.0	38.8	3.8	35.0	21.0
15	146.8	31.8	3.8	28.0	25.2
20	119.95	26.0	3.8	22.2	26.6
25	103.85	22.5	3.8	18.7	28.1
30	91.90	19.9	3.8	16.1	29.0
35	82.58	17.9	3.8	14.1	29.6
40	75.15	16.3	3.8	12.5	30.0
45	69.05	15.0	3.8	11.2	30.2
50	63.95	13.8	3.8	10.0	30.0
55	59.62	12.9	3.8	9.1	30.0
60	53.20	11.5	3.8	7.7	27.7

TABLE 4.13 ROOF SEGMENT R10 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	54.4	3.8	50.6	15.2
10	179.0	40.1	3.8	36.3	21.8
15	146.8	32.9	3.8	29.1	26.2
20	119.95	26.9	3.8	23.1	27.7
25	103.85	23.3	3.8	19.5	29.3
30	91.90	20.6	3.8	16.8	30.2
35	82.58	18.5	3.8	14.7	30.9
40	75.15	16.8	3.8	13.0	31.2
45	69.05	15.5	3.8	11.7	31.6
50	63.95	14.3	3.8	10.5	31.5
55	59.62	13.4	3.8	9.6	31.5
60	53.20	11.9	3.8	8.1	29.2

TABLE 4.14 ROOF SEGMENT R11 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	11.1	3.8	7.3	2.2
10	179.0	8.2	3.8	4.4	2.6
15	146.8	6.7	3.8	2.9	2.6
20	119.95	5.5	3.8	1.7	2.0
25	103.85	4.8	3.8	1.0	1.5
30	91.90	4.2	3.8	0.4	0.7
35	82.58	3.8	3.8	0	0

TABLE 4.15 ROOF SEGMENT R12 – STORAGE CALCULATED BY FLOW BALANCE

Duration Minutes	Intensity mm/hr	Q in L/s	Q out L/s	Difference L/s	Storage m^3
5	242.6	10.8	3.8	7.0	2.1
10	179.0	8.0	3.8	4.2	2.5
15	146.8	6.5	3.8	2.7	2.4
20	119.95	5.3	3.8	1.5	1.8
25	103.85	4.6	3.8	0.8	1.2
30	91.90	4.1	3.8	0.3	0.5
35	82.58	3.7	3.8	0	0

For all roof segments other than R2, the rainfall balance calculation yields the storage volume that can be used. For R2, the available rainfall volume exceeds the physical storage available, and therefore the physical storage based on roof geometry governs the amount of storage. The total roof storage available is the sum of the storage in the 12 segments.

Roof storage = 0 +8.7 +18.2 +5.8 +52.4 +3.5 +35.3 +36.6 +30.2 +31.6 +2.6 +2.5 = 227.4 m^3.

Ground level storage is available under interim conditions is noted on Drawing C002, and again summarized in Appendix A. Storage available was calculated using the formula for an inverted pyramid, based on the maximum water surface level of 103.46 m, and a depth of 0.3 m. A summary of the surface stormwater storage is provided in Appendix A. The total for all surface stormwater storage is 890.9 m^3.

The combination of roof and surface level storage = 227.4 m^3 + 890.9 m^3 = 1118 m^3. This value exceeds the required 100 year storage requirement of 829 m^3. There will be adjustments made to the surface ponding volumes if and when the future bus loop is added, but any changes made will still result in sufficient ponding volume being available.

Flow regulation is provided at manholes STMH1 and CBMH2. Flow at STMH1, which is the outlet manhole from the site, is limited to 850 L/s, which is the release rate allowed from the site as stipulated by Novatech Engineering Consultants Ltd. This release rate was determined by Novatech for this site as part of their stormwater design for the Fernbank Community. As noted earlier in this report, this release rate was provided by Novatech to GENIVAR in November 2013.

An orifice plate is used to regulate this flow as shown on Drawing C003. The maximum head of water is dictated by the overland overflow elevation of 103.46 m. The invert of the outlet pipe is 100.91 m. The orifice plate was sized using the orifice equation:

$Q = 0.61 \times A \times (2 \times g \times H)^{0.5}$, where Q = discharge rate in m^3/s,

Orifice coefficient = 0.61

A = area of orifice in m^2

g = gravitational constant = 9.81 m/s^2

H = head of water (m) above the centre of the orifice = (103.46 – 100.91) – (0.5 x orifice diameter)

An orifice diameter of 514 mm provides the required flow. No change is required at STMH1.

$Q = .61 \times A \times (2 \times g \times H)^{0.5} = 0.61 \times (\Pi \times (0.514/2)^2) \times (2 \times 9.81 \times (2.55 - (0.5 \times 0.514)))^{0.5} = .849\ m^3/s$

The flow limit at CBMH2 is calculated as the difference between the flow limit of 850 L/s and the flow entering STMH1 from the building roof and drainage sub-areas to the south, consisting of sub-areas 5, 9, 11, 12, 13 and 14. It is preferred that the flow from the south be separated from the flow entering the surface ponding areas in order to provide a less restricted flow path. The controlled flow from the roof of 0.7160 ha is limited to 57 L/s based on the use of 30 flow controlled roof drains, with a flow of 1.9 L/s per drain.

The 100 year flow from the remaining south areas is generated from 0.3296 ha of impervious surfaces, 0.0118 ha of gravel surfaces, and 0.2987 ha of landscaped surfaces. This area totals to 0.6401 ha and has a runoff coefficient of 0.593. For 100 year conditions, the runoff coefficient is increased to 0.677. The 100 year uncontrolled runoff from this area can be estimated using the Rational Method, assuming a time of concentration of 10 minutes, and a corresponding rainfall intensity of 179 mm/hour.

$Q = 2.78 \times C \times I \times A = 2.78 \times 0.677 \times 179 \times 0.6401 = 215.6$ L/s.

The desired controlled rate of flow leaving CBMH2 is therefore 850 – (57 + 215.6) = 577.4 L/s.

At CBMH2, the maximum head of water is again dictated by the overland overflow elevation of 103.46 m. The invert of the outlet pipe is 101.069 m. The orifice plate was sized using the orifice equation:

H = head of water (m) above the centre of the orifice = (103.46 – 101.069) – (0.5 x orifice diameter)

An orifice diameter of 429 mm provides the required flow. A new orifice plate meeting this requirement will replace the existing orifice plate.

$Q = .61 \times A \times (2xgxH)^{0.5} = 0.61 \times (\Pi x(0.429/2)^2) \times (2 \times 9.81 \times (2.391 - (0.5 \times 0.429))^{0.5} = 0.5762$ m^3/s, which is slightly below the maximum release rate of 0.5774 m^3/s.

The storage required upstream of CBMH2 to restrict the flow to 576.2 L/s can be calculated using the Modified Rational Method. The contributing sub-areas are 1 to 4, 6 to 8, 10, and 15 to 33, with a total area of 4.5039 ha. This area is comprised of 2.4352 ha of impervious area, 0.0698 ha of gravel surface, and 1.9989 ha of landscaped area. The weighted average runoff coefficient is 0.608, which is increased to 0.693 for 100 year conditions.

TABLE 4.16 100 YEAR STORAGE REQUIREMENTS UPSTREAM OF CBMH2

For 100 year storm event (C = 0.693 and area = 4.5039 ha)

Duration Minutes	Intensity mm/hr	Q L/s	Q Allowable L/s	Difference L/s	Storage m^3
5	242.6	2105	576.2	1529	458.7
10	179.0	1554	576.2	977.8	586.7
15	146.8	1274	576.2	697.8	628.0
20	119.95	1041	576.2	464.8	557.8
25	103.85	901	576.2	324.8	487.2
30	91.90	797	576.2	220.8	397.4
35	82.58	717	576.2	140.8	295.7

A required volume of 628 m^3 is indicated for the 100 year event. This volume is easily provided by the upstream surface detention ponding which has an available volume of 890.9 m^3.

4.11 Watercourses

No alterations to watercourses are required as a result of this proposed site plan amendment.

4.12 Pre and Post Development Peak Flow Rates

The existing site has an allowable release rate of 850 L/s for all storm events up to 100 years. No modification to this rate is proposed for the site plan amendment.

As noted on the storm sewer design sheets for this report and the original servicing report, the 5 year design flow rate from the storm sewer system is increasing from 727.9 L/s to 819.4 L/s without taking into account any flow controls. As the controlled outflow rate from the site is 850 L/s, there will be no surface ponding at 5 year and lesser storm events. Under the proposed conditions for the site plan amendment, the five year flow rate at CBMH2 will be restricted to 576.2 L/s as compared to the original site design 5 year rate at this location of 500.9 L/s.

Post-development peak flow rates under 5 and 100 year conditions are provided in Section 4.10 above for several different return periods as part of the storage calculations.

The drainage area plan on Drawing C001, and the storm sewer design sheet describe the post-development drainage areas and extent of imperviousness.

As noted, peak flows up to the 100 year event will be attenuated to not exceed the allowable release rate of 850 L/s.

4.13 Diversion of Drainage Catchment Areas

There will be no major diversion of on-site drainage catchment areas arising from the proposed work described in this report. Drainage from the site itself is diverted to the off-site treatment facility prior to rejoining with the existing outlet channel. Off-site drainage has been diverted around the west and north perimeter of the site at the time of original site development.

4.14 Minor and Major Systems

Proposed minor and major systems are shown on Drawings C004 and C005, and have been described in previous sections of the report. The minor site storm sewer system is described on the attached storm sewer calculation sheets. The 10 minute minimum inlet time is the standard utilized by the City of Ottawa for the design of municipal storm sewer systems as per clause 5.1.4 of the Sewer Design Guidelines, Second Edition, Document SDG002, October 2012.

The proposed stormwater management facility includes roof top and ground level storage, and flow regulation at the outlet manholes. Quality treatment will be provided off-site by others. Stormwater will back up into the storage areas when incoming flows exceed the allowable system release rate, and will be released over an extended period as incoming flows diminish and cease.

4.15 Downstream Capacity Where Quantity Control Is Not Proposed

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

4.16 Impacts to Receiving Watercourses

The impact to the receiving watercourse has been mitigated through conformance with regulatory requirements for quantity and quality control.

4.17 Municipal Drains and Related Approvals

No municipal drains are located on 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 overflow from the site will be to the north-east of the property in the direction of the existing overland flow.

4.20 Hydraulic Analysis

Hydraulic calculations for the site storm sewers are provided in the storm sewer design sheet. The maximum hydraulic grade line is defined by the maximum stormwater overflow elevation of 103.46 m.

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 C001 includes requirements for the Contractor to implement Best Management Practices to minimize erosion and sediment release during construction activities. Specific measures are dictated including SiltSack from Terrafix or approved alternative filter at catch basins and catch basin manholes, and a temporary silt control fence installed as per OPSD 219.110.

Erosion control measures are also listed on Drawing C001, including the need to minimize areas of disturbed soil, prevent runoff from flowing across disturbed areas, reinstatement or protection of disturbed surfaces as soon as possible, and temporary protection of disturbed areas and stockpiles that have to be in place for extended periods of time.

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 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.

4.23 Fill Constraints

There are no specific fill constraints applicable to this site. The proposed grade raise and finished floor elevation have been considered in the geotechnical engineering report recommendations.

5. APPROVAL AND PERMIT REQUIREMENTS

The proposed development is subject to site plan amendment 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

The MVCA comments from the original site development are included in Appendix C of this report. No other review agency comments have been submitted, with the exception of engineering review comments from the City of Ottawa.

6.3 Signature and Professional Stamp

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APPENDIX A

STORM SEWER DESIGN SHEET

SURFACE STORMWATER STORAGE CALCULATIONS

Storm Sewer Calculation Sheet (Rational Method)   Manning's Coefficient = 0.013   Return Period = 5 years.   Minimum Inlet Time = 10 minutes.

Local Drainage Area	LOCATION		AREA (Ha)				FLOW												
	From Node	To Node	R = 0.9	R = 0.7	R = 0.25	Total Area	Indiv. 2.78 AC	Accum. 2.78 AC	Time of Conc.	Rainfall Intensity	Peak Flow Q (L/s)	Dia. (mm)	Type	Slope (%)	Length (M)	Capacity (L/s)	Velocity (m/s)	Time of Flow (min.)	Ratio Q/Q full
14	CBMH 18	CBMH 13	.0423		.0437	.0860	.1362	.1362	10	104.4	14.2	250	PVC	0.432	32.2	40.8	0.8	0.67	0.35
13	CBMH 13	CBMH 12	.0011		.0339	.0350	.0263	.1625	10.67	100.8	16.4	300	PVC	0.34	89.9	58.4	0.8	1.87	0.28
11	CBMH 12	CBMH 10	.0767		.0471	.1238	.2246	.3871	12.54	92.5	35.8	450	CON	0.195	49.6	131.4	0.8	1.03	0.27
5	DCB 16	STMH 8	.1258		.0594	.1852	.3560	.3560	10	104.4	37.2	300	PVC	0.34	42.5	58.4	0.8	0.89	0.64
12	ROOF	STMH 8	.7160			.7160	1.7914	1.7914	10	104.4	187.0*	450	CON	2.75	22.5	493.2	3.0	0.12	0.38
-	STMH 8	STMH 9						2.1474	10.89	99.7	214.1	525	CON	0.3	33.0	245.7	1.1	0.50	0.87
-	STMH 9	CBMH 10						2.1474	11.39	97.4	209.2	600	CON	0.19	30.6	279.2	0.96	0.53	0.75
9	CBMH 10	STMH 1	.0837	.0118	.1146	.2101	.3120	2.8465	13.57	88.5	251.9	675	CON	0.15	61.0	339.6	0.92	1.10	0.74
1	CBMH 17	CBMH 16	.0095		.1087	.1182	.0993	.0993	10	104.4	10.4	250	PVC	0.432	47.4	40.8	0.8	0.99	0.25
2	CBMH 16	CBMH 15	.2229		.0496	.2725	.5922	.6915	10.99	99.2	68.6	375	PVC	0.40	51.3	115.7	1.01	0.85	0.59
15, 26	LCB17-29	CBMH17-4	.2058	.0151	.0880	.3089	.6055	.6055	10	104.4	63.2	300	HDPE	0.5	74.0	71.3	0.98	1.26	0.89
16	CBMH17-4	CBMH17-2	.1310		.0539	.1849	.3652	.9707	11.26	98.0	95.1	375	PVC	0.5	26.5	129.3	1.13	0.39	0.74
3, 4	CBMH15	CBMH17-2	.7059		.2135	.9194	1.9145	2.6060	11.84	95.4	248.6	600	CON	0.21	61.5	293.5	1.01	1.01	0.85
	CBMH17-2	CBMH17-3						3.5767	12.85	91.2	326.2	600	CON	0.26	22.0	326.6	1.12	0.33	1.00
17	PAVILION	MAIN	.0581			.0581	.1454	.1454	10	104.4	15.2	200	PVC	1.0	8.0	34.2	1.06	0.13	0.44
33	CBMH17-3	STMH17-1	.0184		.0139	.0323	.0557	3.7778	13.97	87.0	328.7	600	CON	0.29	53.0	345.1	1.18	0.75	0.95
10	CB 14	STMH17-1	.0380		.0863	.1243	.1551	.1551	10	104.4	16.2	300	PVC	0.34	45.8	58.4	0.8	0.95	0.28
6, 18, 19, 21,25	STMH17-1	STMH3	.2467	.0040	.0946	.3453	.6908	4.6237	15.15	83.1	384.2	675	CON	0.22	33.0	411.3	1.11	0.50	0.93
27	LCB17-1	LCB17-4	.1912	.0116	.0586	.2614	.5417	.5417	10	104.4	56.6	300	HDPE	0.5	60.2	71.3	0.98	1.02	0.79
28	LCB17-4	CBMH6	.1091	.0095	.0454	.1640	.3230	.8647	11.02	99.1	85.7	375	HDPE	0.5	86.0	129.3	1.13	1.27	0.66
31	LCB17-14	CBMH6	.0069	.0028	.0607	.0704	.0649	.0649	10	104.4	6.8	250	HDPE	0.5	40.0	43.9	0.87	0.77	0.15
29	CBMH6	STMH17-4	.1430	.0087	.2602	.4119	.5556	1.4852	12.29	93.5	138.9	450	CON	0.222	88.0	139.5	0.85	1.73	1.00
30	LCB17-12	STMH17-4	.1117	.0072	.1682	.2871	.4104	.4104	10	104.4	42.8	250	HDPE	0.5	6.0	43.9	0.87	0.03	0.97
	STMH17-4	STMH5						1.8956	14.02	86.9	164.7	525	CON	0.22	6.5	210.4	0.94	0.12	0.78
24, 20	STMH5	STMH3	.1396	.0047	.0168	.1611	.3701	2.2657	14.14	86.4	195.8	600	CON	0.124	48.0	225.5	0.78	1.03	0.87
7, 22	STMH3	CBMH2	.0492		.0890	.1382	.1850	7.0744	15.65	81.5	576.7	825	CON	0.168	69.7	612.3	1.11	1.05	0.94
23, 32	LCB17-15	CBMH2	.0326		.5053	.5379	.4327	.4327	10	104.4	45.2	300	HDPE /PVC	0.5	113.5	71.3	0.98	1.93	0.63
8	CBMH2	STMH1	.0156	.0062	.0862	.1080	.1110	7.6181	16.70	78.4	597.3**	825	CON	0.36	3.9	898.1	1.63	0.04	0.67
	STMH1	OUTLET						10.4646	16.74	78.3	819.4***	975	CON	0.2	6.0	1046	1.36	0.07	0.78

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

Notes:
*Flow from controlled flow roof drains is limited to 57 L/s.
**Flow from CBMH2 restricted to 576.2 L/s.
*** Flow restricted to 850 L/s.

PROJECT:
École Secondaire Catholique Paul-Desmarais Dome
LOCATION: 5315 Abbott Street, Ottawa, ON
File Ref.: 17M-02044-00   Revision Date: 2018-03-06   Sheet No. 1 of 1

École Secondaire Catholique Paul-Desmarais

5315 Abbott Street, Ottawa, ON.

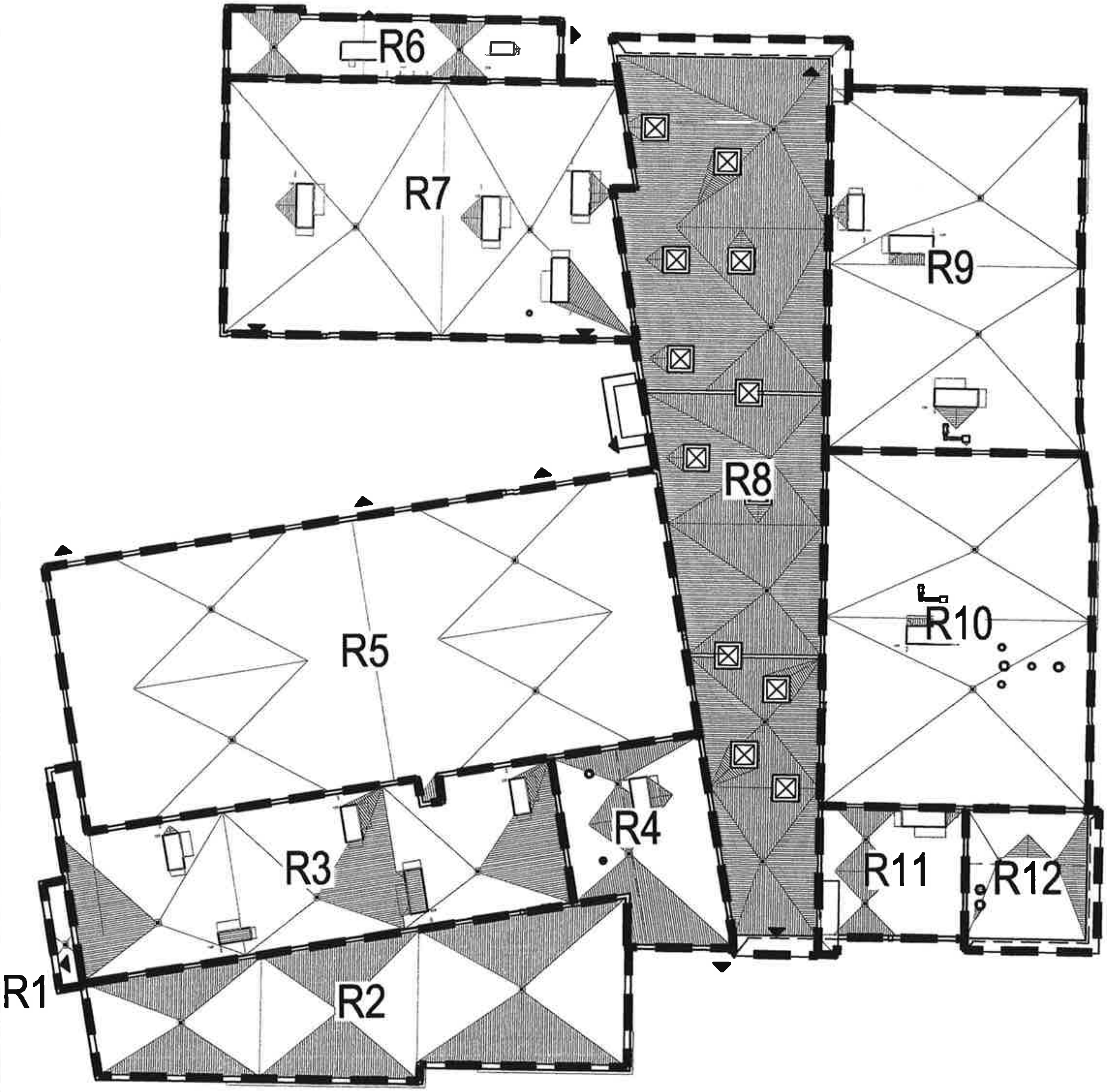
SURFACE STORMWATER STORAGE – EXISTING AND PROPOSED CONDITIONS – JULY 20, 2018

Ponding Location	Existing Ponding Area (m^2)	Existing Ponding Depth (m)	Existing Ponding Volume (m^3)	Proposed Ponding Area (m^2)	Proposed Ponding Depth (m)	Proposed Ponding Volume (m^3)
CB1	1244	0.23	95.4			
CB3	184	0.31	19.0			
CB4	347	0.27	31.2			
CB5	143	0.25	11.9			
CB6	130	0.27	11.7			
CB7	121	0.22	8.9			
CB8	79	0.25	6.6			
CB11				439	0.2	29.3
CB13	1110	0.27	99.9	399	0.27	35.9
CB15	307	0.24	24.6			
CBMH7	1110	0.27	99.9			
CB17-3				362	0.2	24.1
CB17-4				309	0.2	20.6
CB17-6				1010	0.3	101.0
New sports field				6804		680
		Total Volume	409.1		Total Volume	890.9

Pond volumes were estimated using the inverted pyramid formula: Volume = area x depth / 3.

APPENDIX B

ROOF DRAINAGE AREA LOCATIONS (FIG. 1)



WSP

2611 QUEENSVIEW DRIVE, SUITE 300
OTTAWA (ONTARIO) CANADA K2B 8K2
TELEPHONE: (613) 829-2800
FAX: (613) 829-8299

PROJECT: ÉCOLE SECONDAIRE CATHOLIQUE COMMUNAUTÉ DE FERNBANK	DRAWING NAME: ROOF AREAS		DESIGNED BY: J.J DRAWN BY: B.N.
5315 ABBOTT STREET OTTAWA, ONTARIO.	DATE: MAR 07, 2014 SCALE: N.T.S.	REVIEWED BY: J.J	SHEET: FIG.1
PROJECT NO.: 121-26221-00			

APPENDIX C

CORRESPONDENCE FROM MISSISSIPPI VALLEY CONSERVATION AUTHORITY – ORIGINAL SITE

Johnston, James

From: Craig Cunningham <ccunningham@mvc.on.ca>
Sent: Monday, January 07, 2013 4:00 PM
To: James Johnston
Subject: RE: Rép. : RE: Pre-Consultation: New High School in Kanata (CECCE)

Hi Jim,

I forwarded this on to our engineering staff for review, but I'm not certain anyone got back to you directly with preliminary comments. If that is the case, here are some initial comments related to the proposal for your consideration:

- Until the trunk sewer is built, must match post development flows to pre development flows. After it is built, can match post development flows to the design.
- Assume it (swale/watercourse) is fish habitat (because there is fish habitat within 1 km of the site), therefore will need 70% quality treatment.
- An offline pond will be required since it (swale/watercourse) has been assumed to be fish habitat.
- Development setbacks from watercourse as identified by City OP, Carp River Subwatershed Study, should be discussed.

Excuse the delay, but hope this helps. Let me know if you wish to discuss further.

Regards,

Craig

Craig Cunningham
Environmental Planner
Mississippi Valley Conservation
Tel: (613) 259-2421 x229
Fax: (613) 259-3468

ccunningham@mvc.on.ca

From: James Johnston [mailto:James.Johnston@genivar.com]
Sent: December-07-12 8:47 AM
To: ccunningham@mvc.on.ca
Subject: FW: Rép. : RE: Pre-Consultation: New High School in Kanata (CECCE)

Craig,

We are starting work on a new high school project at the east end of Stittsville. The proposed site is highlighted on the attached plans. It appears that drainage will be directed to a future off-site SWM pond located north of the site. There is an existing wet pond on the west side of Iber Road not far west of the site, and the outlet ditch from this pond currently crosses the high school site as it drains to the east and north. We would be interested in receiving any preliminary comments from MVC pertaining to this development and SWM requirements.

*Please take note of our new address / contact info *

GENIVAR

James (Jim) Johnston, P.Eng., LEED® AP
GENIVAR INC.
2611 Queensview Drive, Suite 300, Ottawa, Ontario K2B 8K2
T 613.829-2800 x19349 | F 613.829-8299 | C 613.298-5960 | www.genivar.com

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APPENDIX D

WATER SYSTEM BOUNDARY CONDITIONS

WATER AGE ANALYSIS

FUS FIRE DEMAND CALCULATION

EPANET WATERMAIN ANALYSIS

Johnston, James

From: Whittaker, Damien <Damien.Whittaker@ottawa.ca>
Sent: Wednesday, March 12, 2014 8:35 AM
To: Johnston, James
Subject: RE: City File D07-12-13-0234 & D02-02-13-0127 5315 Abbott - Boundary Conditions

Jim,

Please find boundary conditions below:

- PKHR = 155.6m
- Max HGL = 161.3m
- MXDY + Fire = 155.6m

Please feel free to ask for clarification, or further information, on any of the comments above.

Thank you,

Damien Whittaker, P.Eng Project Manager Development Review, Suburban West Sub-unit
City of Ottawa 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
613-580-2424 x16968 damien.whittaker@ottawa.ca 01-14

From: Johnston, James [mailto:James.Johnston@wspgroup.com]
Sent: Wednesday, February 26, 2014 4:48 PM
To: Whittaker, Damien
Subject: FW: City File D07-12-13-0234 & D02-02-13-0127 5315 Abbott - Boundary Conditions

Damien,
Please find attached the requested water demand information to enable establishment of boundary conditions.

Please note that we may be making some changes to the sanitary piping shown on the location sketch, which may result in a minor shift in the site watermain feeding the private hydrant. We do not anticipate any change in the location of the hydrant, the water service entry to the building, or the stub from the street main.

WSP
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www.wspgroup.com
We were GENIVAR. We are now WSP.

From: Whittaker, Damien [mailto:Damien.Whittaker@ottawa.ca]
Sent: Friday, February 21, 2014 3:09 PM
To: Johnston, James
Subject: RE: City File D07-12-13-0234 & D02-02-13-0127 5315 Abbott - Boundary Conditions

Jim,

Please provide a location plan, a discussion of the proposed water connection (only for this special application), and the following data

- Avg. Day = X l/s

- Max. Day = Y l/s
- Peak Hour = Z l/s
- Fire Flow = A l/s

Please note that the fire flow should be calculated as per FUS guidelines and calculations are required with the application.

Please feel free to ask for clarification, or further information, on any of the comments above.

Regards,

Damien Whittaker, P.Eng **Project Manager** **Development Review, Suburban West Sub-unit**
City of Ottawa 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
☎ 613-580-2424 x16968 damien.whittaker@ottawa.ca 01-14

From: Johnston, James [mailto:James.Johnston@wspgroup.com]
Sent: Thursday, February 20, 2014 5:38 PM
To: Whittaker, Damien
Subject: City File D07-12-13-0234 & D02-02-13-0127 5315 Abbott - Boundary Conditions

Damien,
We are in the process of responding to the City comments on the initial site plan application. It was noted that we should request boundary conditions for water servicing. I would appreciate if you could assist us in obtaining this information.

WSP
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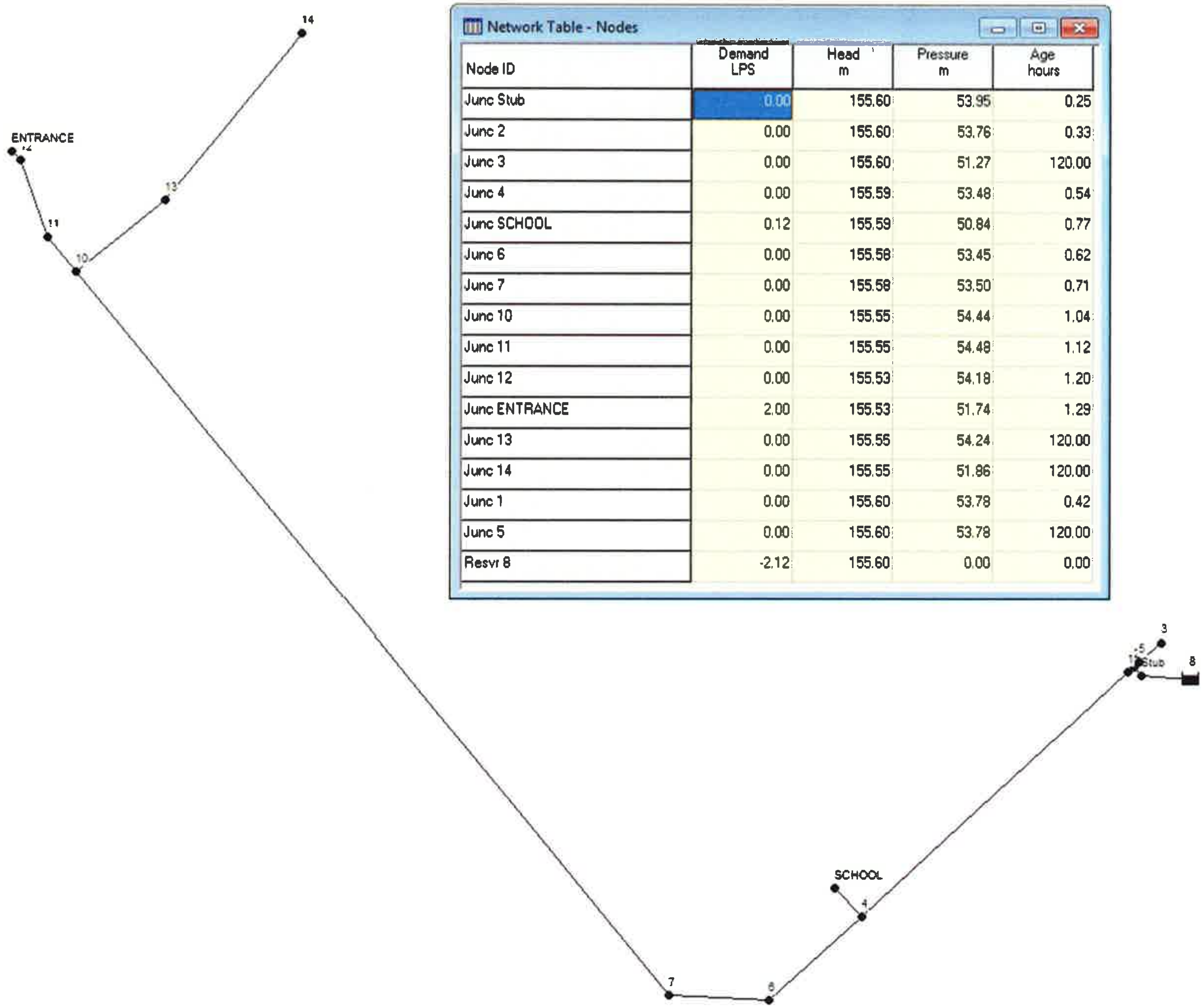
www.wspgroup.com
We were GENIVAR. We are now WSP.

précité) ou par courriel, puis supprimer sans délai la version originale de la communication ainsi que toutes ses copies. Je vous remercie de votre collaboration.

Network Table - Nodes

Node ID	Demand LPS	Head m	Pressure m	Age hours
Junc Stub	0.00	155.60	53.95	0.25
Junc 2	0.00	155.60	53.76	0.33
Junc 3	0.00	155.60	51.27	120.00
Junc 4	0.00	155.59	53.48	0.54
Junc SCHOOL	0.12	155.59	50.84	0.77
Junc 6	0.00	155.58	53.45	0.62
Junc 7	0.00	155.58	53.50	0.71
Junc 10	0.00	155.55	54.44	1.04
Junc 11	0.00	155.55	54.48	1.12
Junc 12	0.00	155.53	54.18	1.20
Junc ENTRANCE	2.00	155.53	51.74	1.29
Junc 13	0.00	155.55	54.24	120.00
Junc 14	0.00	155.55	51.86	120.00
Junc 1	0.00	155.60	53.78	0.42
Junc 5	0.00	155.60	53.78	120.00
Resvr 8	-2.12	155.60	0.00	0.00



WATER DISTRIBUTION - PROPOSED FIRE FLOW DEMANDS

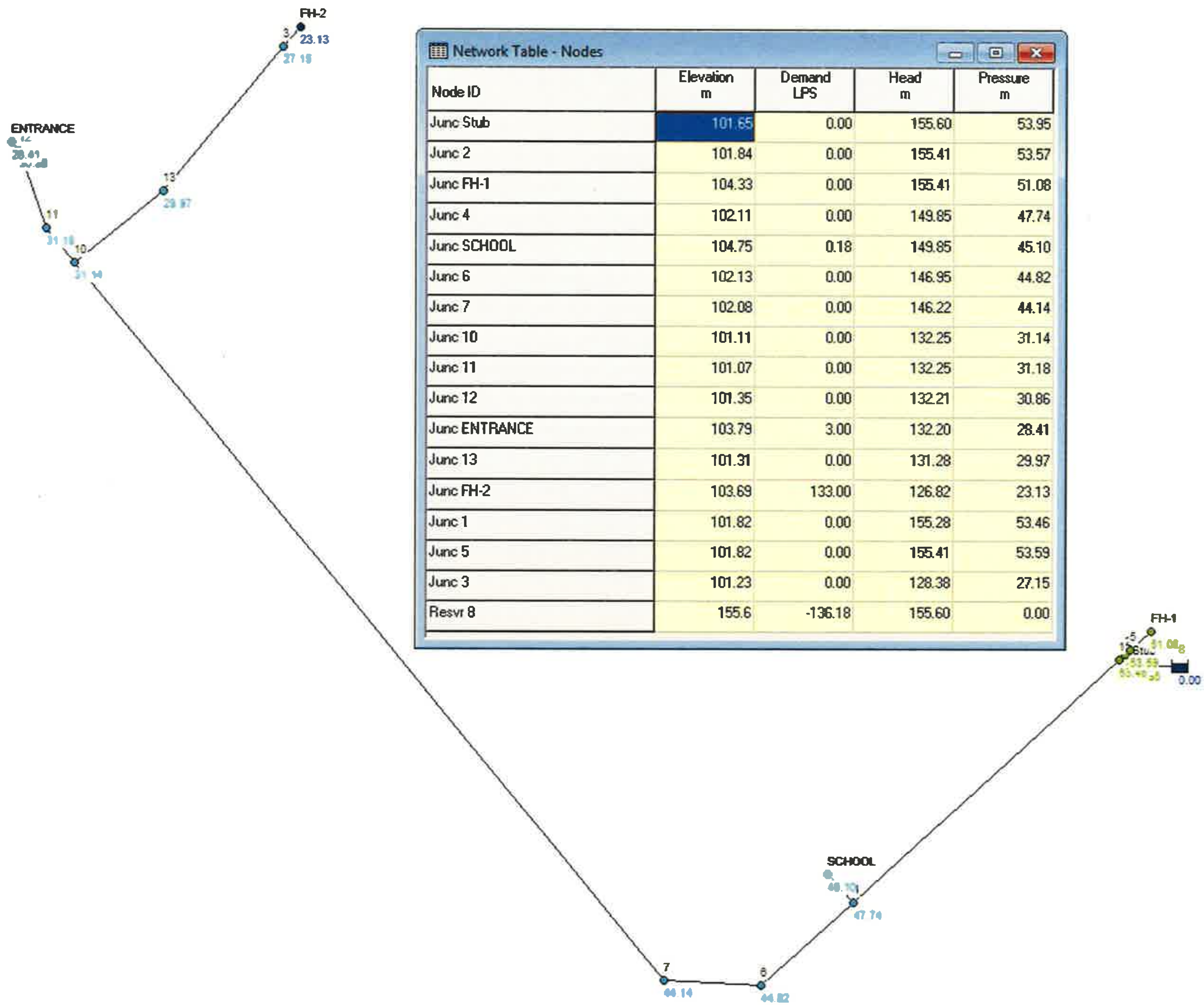
$F = 220 C \sqrt{A}$

Type of Construction Coefficient:		Comments
Wood Frame	1.5	(all structurally combustible)
Ordinary	1.0	(brick, masonry wall, combustible floor and interior)
Non-Combustible	0.8	(unprotected metal structural component, masonry or metal walls)
Fire Resistive	0.6	(fully protected frame, floors and roof)

Combustibility:		
Non-Combustible (Low Hazard Occupancy)	-25%	
Limited Combustible	-15%	
Combustible	0%	
Free Burning	15%	
Rapid Burning	25%	

Sprinkler Protection:		
Complete Sprinkler System	-50%	(max.)
NFPA 13 Conformed	-30%	(max.)
If Water Supply Standard for Both System and Fire Lines	-10%	additional (max.)
Fully Supervised System	-10%	additional (max.)
None	0%	

	New Pavilion at Sports Dome - Ecole secondaire Paul-Desmarais		
Type of Construction Coefficient	Wood Frame		
	1.5		
Gross Floor Area (m^2)	581	m2	
Fire Flow, F (L/min)	7,954	L/min	
	8,000	L/min	
Combustibility	Limited Combustible		
	-15%		
F	6,800	L/min	
Sprinkler Protection	None		
	0%		
Additional Credit	None		
	0%		
Exposure Distances			
North	6	m	20%
South	46	m	0%
East	>45	m	0%
West	>45	m	0%
	Total	=	**20%**
Total Required Fire Flow, F	8,000	L/min	
F	**133**	**L/s**	



Network Table - Nodes

Node ID	Elevation m	Demand LPS	Head m	Pressure m
Junc Stub	101.65	0.00	155.60	53.95
Junc 2	101.84	0.00	155.41	53.57
Junc FH-1	104.33	0.00	155.41	51.08
Junc 4	102.11	0.00	149.85	47.74
Junc SCHOOL	104.75	0.18	149.85	45.10
Junc 6	102.13	0.00	146.95	44.82
Junc 7	102.08	0.00	146.22	44.14
Junc 10	101.11	0.00	132.25	31.14
Junc 11	101.07	0.00	132.25	31.18
Junc 12	101.35	0.00	132.21	30.86
Junc ENTRANCE	103.79	3.00	132.20	28.41
Junc 13	101.31	0.00	131.28	29.97
Junc FH-2	103.69	133.00	126.82	23.13
Junc 1	101.82	0.00	155.28	53.46
Junc 5	101.82	0.00	155.41	53.59
Junc 3	101.23	0.00	128.38	27.15
Resvr 8	155.6	-136.18	155.60	0.00