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39 Years

Stinson Lands
4386 Rideau Valley Drive

Conceptual Site Servicing & Stormwater Management Report

Engineering excellence. Planning progress. Liveable landscapes.

STINSON LANDS
(4386 RIDEAU VALLEY DRIVE)

CONCEPTUAL SITE SERVICING AND STORMWATER MANAGEMENT REPORT

Prepared for:

Uniform Urban Developments Ltd.
Suite 300, 117 Centrepointe Drive
Ottawa, Ontario
K2G 5X3

Prepared By:

NOVATECH
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario
K2M 1P6

January 24, 2023

Novatech File: 121153

NOVATECH
Engineers, Planners & Landscape Architects

January 24, 2023

City of Ottawa
Planning, Real Estate, and Economic Development Department
Development Review - Rural Branch
110 Laurier Avenue West, 4th Floor
Ottawa, ON K1P 1J1

Attention: **Mr. Jeff Ostafichuk, Planner**
Planner III

Mr. Brian R. Morgan, CET
Project Manager

Reference: **Stinson Lands**
Preliminary Site Servicing and Stormwater Management Brief
Novatech File No.: 121153
City Planning File No.: TBD

Please find enclosed the Preliminary Site Servicing and Stormwater Management Brief for the Stinson Lands, located at 4386 Rideau Valley Dive in Manotick.

The report has been prepared to confirm that the proposed draft plan can be serviced with the existing sewers, watermain, drainage outlet and utilities fronting the site. The analysis within this report are based the pre-consultation meeting and recent discussions with the City of Ottawa (Appendix E).

If you have any questions or comments, please do not hesitate to contact us.

Yours truly,

NOVATECH

Bassam Bahia, M.Eng., P. Eng.
Senior Project Manager | Land Development

cc: Ryan McDougall/Annibale Ferro, Uniform Urban Developments

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1.0 INTRODUCTION

1.1 Background

This report will assess the adequacy of services for the proposed Stinson Lands (Subject Site) development located at the intersection of Rideau Valley Drive and Bankfield Road as shown on **Figure 1.1** – Key Plan**.** The site is located at the northwest corner of Rideau Valley Drive and Bankfield Road. The site is bounded on the west by the Wilson-Cowan Drain, the north by Mud Creek and the Oxbow ditch, the east by Rideau Valley Drive, and the south by Bankfield Road. The draft plan also includes a parcel east of Rideau Valley Drive and bounded to the west by the Rideau River.

The existing land use consists of a single residential building and three barns and is generally agriculture with a vegetated area near the intersection of Rideau Valley Drive and Bankfield Road as shown on **Figure 1.2** – Existing Conditions Plan. The grade of the development property generally slopes from southeast to northwest to east towards the Rideau River with a grade difference of 7.5m from the southeast corner to the northwest corner of the site.

1.2 Development Intent

The proposed subdivision of the Subject Site will comprise of residential dwellings, public right-of-ways (ROW), open space blocks, two park block and servicing/road widening blocks, as shown in **Table 1.1**. The proposed development concept is shown on **Figure 1.3** – Site Plan.

Table 1.1: Land Use, Development Potential, and Yield

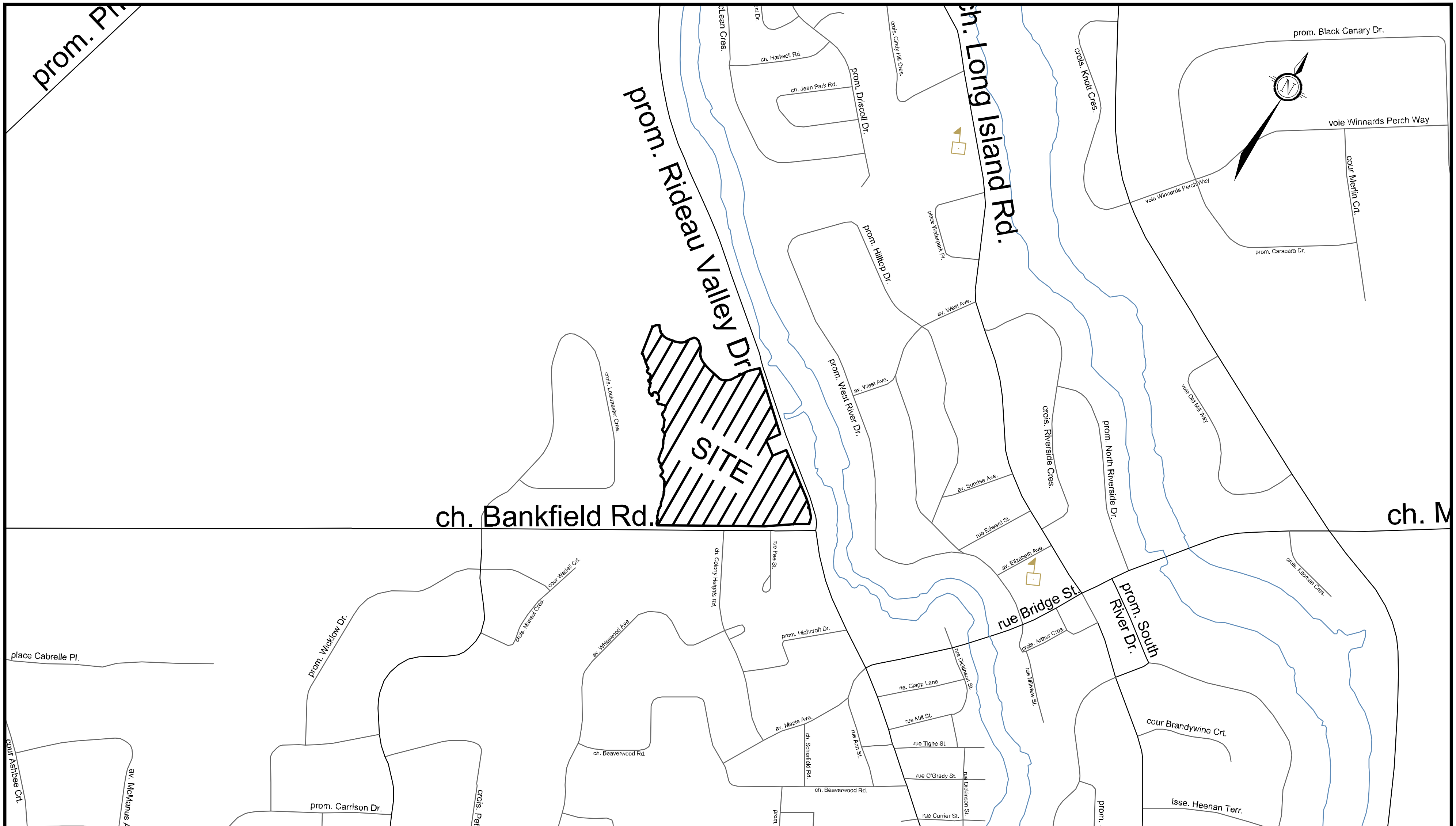
Unit Type	Number of Units	Area
Singles	62	3.05
Semis	16	0.41
Townhomes	69	1.57
Open Space & Park Blocks	-	2.98
Local Roads	-	2.05
Servicing and Road Widening	-	0.22
TOTAL	**147**	**10.28 ha**

The Subject Site is inherently located within the public service area in the Official Plan of the City of Ottawa and the Secondary Plan of the Village of Manotick; therefore, the site has been designed with municipal water and sanitary sewage collection. The development will contain City of Ottawa municipal road allowances of 14.75 - 18.0 meters wide.

1.3 Report Objective

This report assesses the adequacy of existing and proposed services to support the proposed development. This report will be provided to the various agencies for draft plan approval.

The City of Ottawa Applicant Study and Plan Identification List along with proof of a pre-consultation meeting is provided in **Appendix A**. The City of Ottawa Servicing Study Guidelines for Development Applications checklist has been completed and is provided in **Appendix B**.



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CITY OF OTTAWA
STINSON LANDS
4386 RIVER ROAD

KEY PLAN

SCALE	NOT TO SCALE
DATE	AUG 2022
JOB	121153
FIGURE	1.1

M:\2021\121153\CAD\Design\121153-BP.dwg, Fig 1.1 - KP, Oct 07, 2022 - 11:58am, brundle

M:\2021\121153\CAD\Design\121153-EX.dwg, Fig 1.2, Nov 22, 2022 - 4:21pm, brundle

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LEGEND

- DEVELOPMENT LIMIT
- BOUNDARY
- WATERCOURSE

CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

EXISTING CONDITIONS

SCALE NOT TO SCALE

DATE	JOB	FIGURE
AUG 2022	121153	1.2

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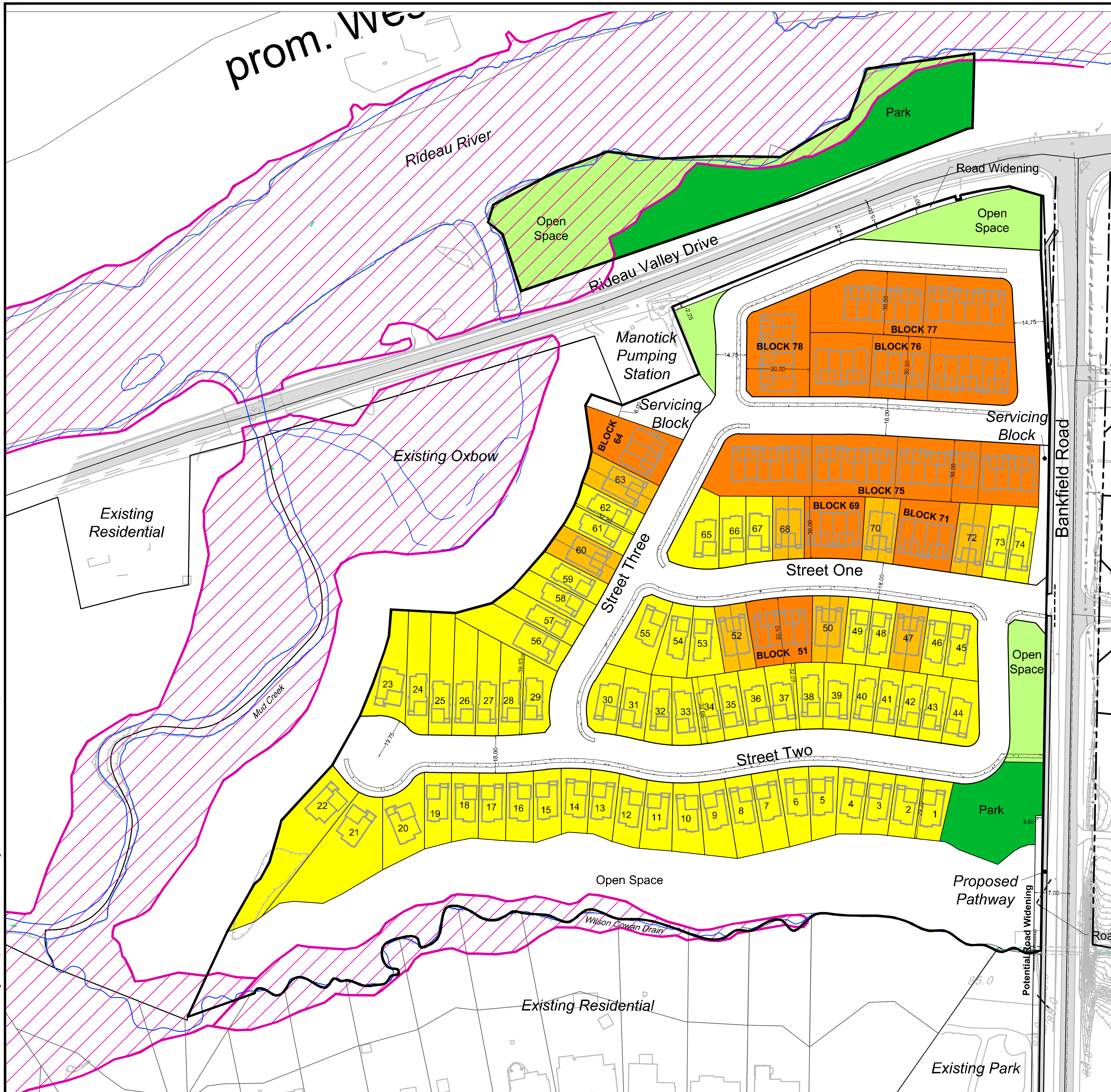

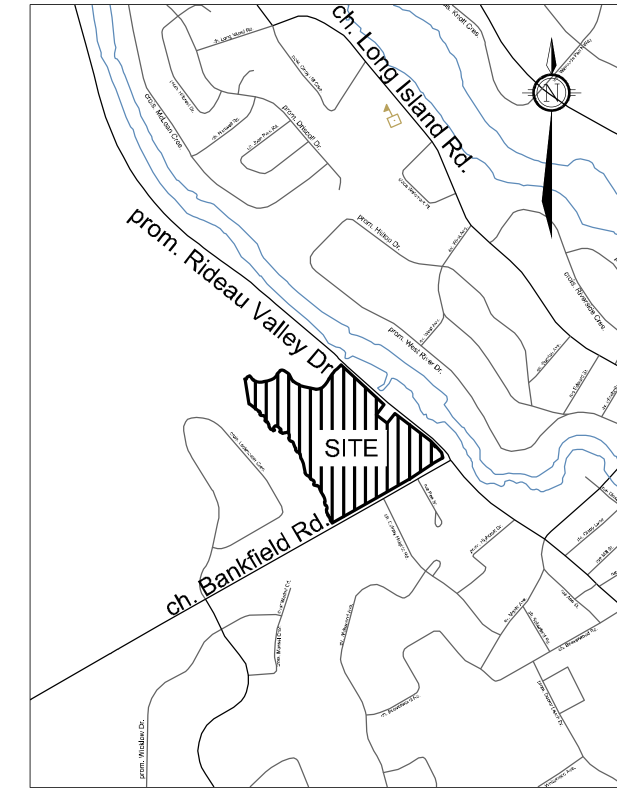

LEGEND

- Single Detached
- Semi Detached
- Townhome
- Park
- Open Space
- Floodplain

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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

SITE PLAN

SCALE		
1:2000		
DATE AUG 2022	JOB 121153	FIGURE 1.3

M:\2021\121153\CAD\Planning\Concept Plans\121153-DCP.dwg, FIG 1.3 Site Plan, Nov 16, 2022 - 2:33pm, brundle

2.0 REFERENCES AND SUPPORTING DOCUMENTS

2.1 Guidelines and Supporting Studies

The following guidelines and supporting documents were utilized in the preparation of this report:

- **City of Ottawa Official Plan** (OP)
  City of Ottawa, adopted by Council 2003.
- **City of Ottawa Infrastructure Master Plan** (IMP)
  City of Ottawa, November 2013.
- **Village of Manotick Secondary Plan** (SP)
  City of Ottawa [Amendment #162, March 3, 2016]
- **Village of Manotick Servicing Master Plan and Trunk Services** (Manotick MSP)
  J. L. Richards and Associates, May 2003.
- **Village of Manotick Municipal Servicing – Main Sanitary Sewage Pump Station** (Manotick PS Report)
  IBI Group, September 2008.
- **City of Ottawa Water Distribution Guidelines** (OWDG)
  City of Ottawa, October 2012.
- **Revisions to OWDG** (ISTBs-2010-01, 2014-02, 2018-02, 2018-04, & 2021-03)
  City of Ottawa, December 2010, May 2014, March 2018, June 2018, and August 2021.
- **City of Ottawa Sewer Design Guidelines** (OSDG)
  City of Ottawa, October 2012.
- **Revisions to OSDG** (ISTBs-2016-01, 2018-01, & 2018-03)
  City of Ottawa, September 2016 and March 2018.
- **Design Guidelines for Sewage Works and Drinking Water System** (MECP Guidelines)
  Ontario's Ministry of the Environment, 2008.
- **Stormwater Management Planning and Design Manual** (MECP SWM Guidelines)
  Ontario's Ministry of the Environment, 2003.
- **Mud Creek Sub Watershed Study**
  City of Ottawa, October 2015.
- **Engineer's Report on the Wilson Cowan Municipal Drain** (WCMD).
  A.J. Robinson & Associates Inc., July 1983.
- **Engineer's Report for Mud Creek Municipal Drain** (MCMD).
  A.J. Robinson & Associates Inc., December 1984.
- **Mud Creek Flood Risk Mapping from Prince of Wales Drive to Rideau River** (MCFR Mapping).
  Rideau Valley Conservation Authority, July 9, 2019.
- **4386 Rideau Valley Drive N – Stinson Lands SWM Strategy Outline** (Stinson Lands SWM Memo).
  Novatech, June 8, 2022.

2.2 Geotechnical Investigation and Fluvial Geomorphology Assessment

Paterson Group (Paterson) conducted a geotechnical investigation (**Appendix F in the digital version of this report**) in support of the proposed residential development:

Geotechnical Investigation – Proposed Residential Development 4386 Rideau Valley Drive, Ottawa, Ontario; Report No. PG5828-1, June 16, 2021, Revised October 14, 2022.

Based on the geotechnical study, it is not anticipated that there will be any significant geotechnical concerns with respect to servicing and developing the site. The borehole locations are provided as **Figure 2.1**. A summary of the geotechnical report findings is provided in **Table 2.1** below.

Table 2.1: Summary of Geotechnical Servicing and Grading Considerations

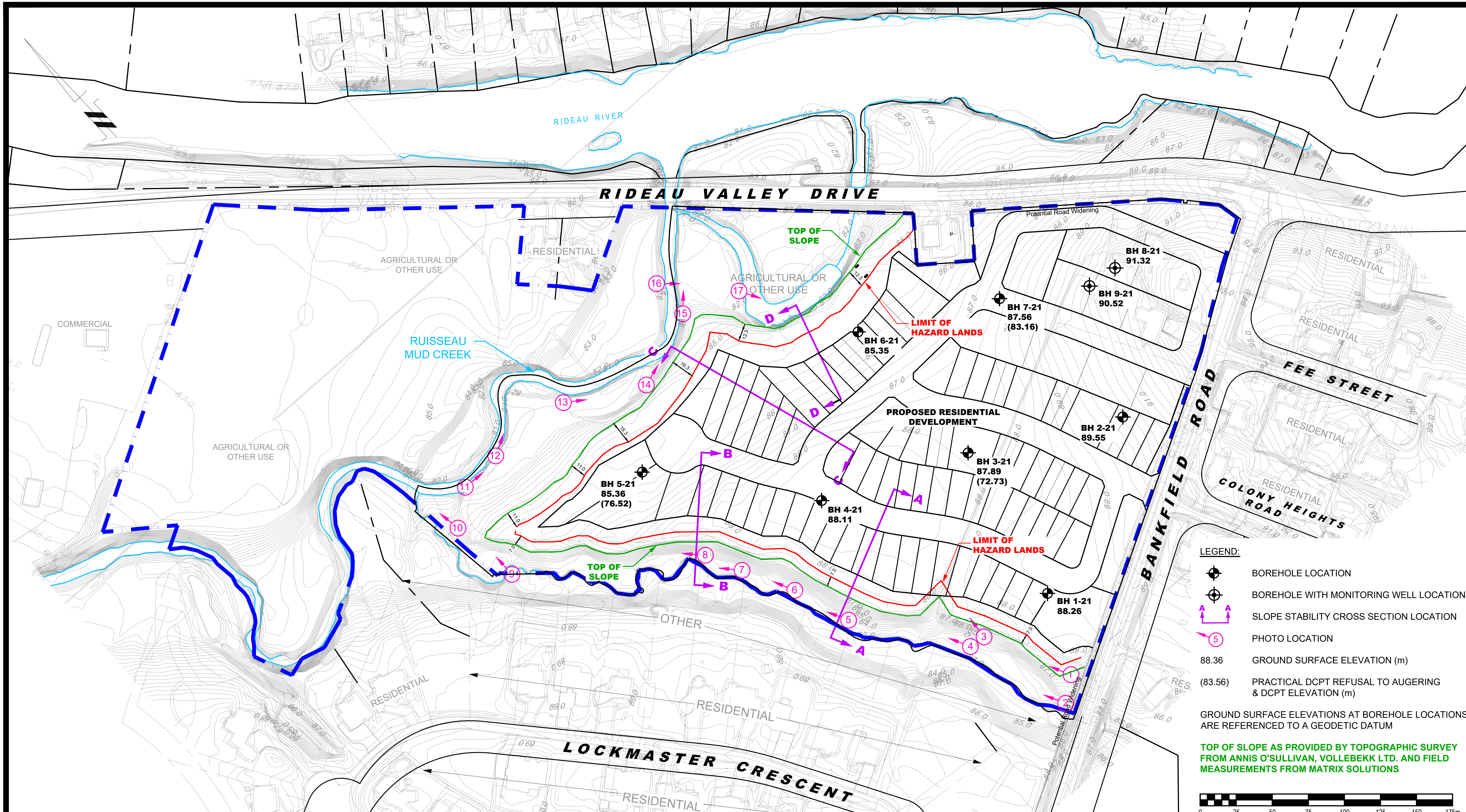
Parameter	Summary
Sub-Soil Conditions	Topsoil underlain by a deposit of silty clay (hard to stiff weathered crust) and glacial till
Grade Raise Restriction	2.0m within the assessment area. Alternate methods of increasing the permissible grade raise could include use of lightweight fill or preloading/surcharging the areas where required.
OHSA Soil Type	Type 2 or 3 for trench excavation side slopes
Groundwater Considerations	Low to Moderate groundwater flow
Pipe Bedding / Backfill	Pipe Bedding 150 mm Granular A Pipe Cover 300 mm Granular A Backfill Native Material 1.5m clay seals
Pavement Structure	40mm Wear Course (SuperPave 12.5) 50mm Binder Course (SuperPave 19.0) 150mm Base (Granular A) 450mm Subbase (Granular B Type II)
Landscape Consideration	Medium Plasticity Soils (PI of 17 to 37%) Large Tree (mature height > 14m) Setback = full mature height of tree Medium Tree (7.5m mature height > 14m) Setback = 4.5m* Large Tree (mature height > 7.5 m) Setback = 4.5m* *Note: Six conditions per City of Ottawa Tree Planting in Sensitive Marine Clay (2017) must be met.

In addition to the above, a slope stability assessment was performed by Paterson as part of the above report and a supplemental slope stability analysis for the blocks adjacent to the Rideau River.

Furthermore, a fluvial geomorphic and erosion hazard assessment has been performed by Matrix Solutions (Matrix) to address potential erosion and hazard potential along the Wilson Cowan Municipal Drian, Mud Creek, and the Oxbow ditch. The report is titled:

Fluvial Geomorphic and Erosion Hazard Assessment Stinson Lands (FGEHA). Report No. 35268-504, November, 2022.

The above report findings and recommendations have been considered in establishing the development limits of the draft plan and to address erosion potential because of increased stormwater flows, as a result of development.



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PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	CONCEPTUAL PLAN UPDATED	14/10/2022	MS
1	TOP OF SLOPE REVISED AS PER SITE SPECIFIC TOPOGRAPHIC SURVEY FROM NOVATECH	08/23/2022	MS

UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500
Date: 06/2021
Drawn by: JM
Report No.: PG5828-1
Checked by: MS
Dwg No.: PG5828-1
Approved by: DJG
Revision No.: 2

3.0 SERVICING AND GRADING

3.1 Bankfield Road and Rideau Valley Drive

Modifications will be required to Bankfield Road to provide access to the proposed subdivision. Additionally, to service the Subject Site, the sanitary and water will need to tie into existing services running along Rideau Valley Drive.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services for the off-site extensions.

3.2 General Servicing

The Subject Site will be serviced using municipal local storm and sanitary sewers, and watermains. As per the above, to service the Subject Site the sanitary and water will need to tie into existing services running along Rideau Valley Drive. Storm sewers would outfall into the Oxbow and shall be conveyed to Mud Creek just upstream of the Rideau Valley Drive Bridge.

The storm / stormwater management, sanitary, and water servicing strategy is discussed in further detail in the following sections.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services for the on-site servicing of the Subject Site.

3.3 General Grading

The grading will direct emergency overland flows from the local road towards the existing Oxbow tributary of Mud Creek, which will ultimately outlet to the Rideau River.

The lots will be graded from front to back to direct surface drainage to the rear yard areas.

Refer to **Figures 3.3 and 3.4** – Conceptual Grading, Erosion and Sediment Control Plan for reference to the Subject Site.

SEE DRAWING No. 121153-GP2

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

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APPROVED BHB

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LICENSED PROFESSIONAL ENGINEER
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100164647
Jan 16, 2023
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Website www.novatech-eng.com

LOCATION
CITY OF OTTAWA
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES

PROJECT No. 121153-00
REV REV # 1
DRAWING No. 121153-GP1

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STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES

PROJECT No. 121153-00
REV # 1
DRAWING No. 121153-GP2

EROSION AND SEDIMENT CONTROL NOTES:

1. ALL EROSION AND SEDIMENT CONTROLS ARE TO BE INSTALLED TO THE SATISFACTION OF THE CITY OF OTTAWA AND MINISTRY OF ENVIRONMENT, CONSERVATION AND PARKS (MOECP), APPROPRIATE TO THE SITE CONDITIONS, PRIOR TO UNDERTAKING ANY SITE ALTERATIONS (FILLING, GRADING, REMOVAL OF VEGETATION, ETC.) AND DURING ALL PHASES OF SITE PREPARATION AND CONSTRUCTION IN ACCORDANCE WITH THE CURRENT BEST MANAGEMENT PRACTICES FOR EROSION AND SEDIMENT CONTROL SUCH AS BUT NOT LIMITED TO INSTALLING SILTSACKS ACROSS MANHOLE/CATCHBASIN LIDS TO PREVENT SEDIMENTS FROM ENTERING STRUCTURES AND INSTALL AND MAINTAIN A LIGHT DUTY SILT FENCE BARRIER AS REQUIRED.
2. TO PREVENT SURFACE EROSION FROM ENTERING THE STORM SYSTEM DURING CONSTRUCTION, SILTSACKS WILL BE PLACED UNDER ALL PROPOSED AND SURROUNDING CATCHBASINS AND MANHOLES. THE SILTSACKS WILL REMAIN IN PLACE UNTIL VEGETATION HAS BEEN ESTABLISHED AND CONSTRUCTION COMPLETE.
3. CONTRACTOR IS TO INSTALL LIGHT DUTY SILT FENCE AS PER OPSD 219.110. CONTRACTOR SHALL MAINTAIN SILT FENCE FOR THE DURATION OF THE CONTRACT.
4. CONTRACTOR IS TO INSTALL STRAW BALES AS PER OPSD 219.180 AS INDICATED AND DIRECTED BY THE ENGINEER PRIOR TO THE COMMENCEMENT OF CONSTRUCTION.
5. ALL AREAS DISTURBED BY CONSTRUCTION ARE TO BE TREATED WITH IMPORTED TOPSOIL, SEED AND MULCH.
6. THE CONTRACTOR ACKNOWLEDGES THAT FAILURE TO IMPLEMENT EROSION AND SEDIMENT CONTROL MEASURES MAY BE SUBJECT TO PENALTIES IMPOSED BY ANY APPLICABLE REGULATORY AGENCY.
7. ALL STREETS ARE TO BE SWEPT ONCE ROADWAYS ARE PAVED AND TO CONTINUE FOR THE DURATION OF CONSTRUCTION ACTIVITY. STREETS ARE TO SWEPT REGULARLY AS INDICATED BY THE ENGINEER.
8. THE CONTRACTOR SHALL KEEP MATERIAL FOR ADDITIONAL EROSION AND SEDIMENT CONTROLS ONSITE AT ALL TIMES. THESE MATERIALS INCLUDE BUT ARE NOT LIMITED TO: SILT FENCES, STRAW BALES, SEDIMENT BAGS AND CLEAR STONE. A CONTINGENCY PLAN TO INCLUDE THE PROVISION OF ADDITIONAL LABOUR, EQUIPMENT OR MATERIALS TO INSTALL ADDITIONAL CONTROL MEASURES, AS WELL AS PROVIDE AN EMERGENCY RESPONSE PLAN IN CASE OF AN ACCIDENTAL EVENT. AS SUCH, THE CONTRACTORS SHALL HAVE ADDITIONAL CONTROL MEASURES ON SITE ALL TIMES WHICH ARE EASILY ACCESSIBLE AND MAY BE IMPLEMENTED AT A MOMENT'S NOTICE.
9. MUD MATS ARE TO BE INSTALLED AND MAINTAINED AT CONSTRUCTION ACCESS POINTS TO MINIMIZE SEDIMENT TRANSFER TO EXISTING ROADWAYS (SEE MUD MAT DETAIL).
10. COORDINATE ALL WORK WITH THE OWNER AND CONTRACT ADMINISTRATOR

LEGEND

- 89.85 / 87.72 — PROPOSED ELEVATION / EXISTING ELEVATION
- EMERGENCY OVERLAND FLOW
- LIGHT DUTY SILT FENCE BARRIER AS PER OPSD 219.110
- TEMPORARY LIGHT-DUTY STRAW BALE BARRIER AS PER OPSD 219.180
- FLOODPLAIN
- Top of Slope (AOV, Matrix)
- New Top of Slope (Novatech)
- Existing Gravel Shoulder
- Existing Paved Shoulder
- Existing Ditch

NORTH

KEY PLAN
N.T.S.

SEE DRAWING No. 121153-GR2

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NOTE:
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	APPROVED	BHB

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LOCATION
CITY OF OTTAWA
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GRADING, EROSION, & SEDIMENT CONTROL PLAN

PROJECT No. 121153-00
REV
REV # 1
DRAWING No.
121153-GR1

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CONCEPTUAL GRADING, EROSION, & SEDIMENT CONTROL PLAN

PROJECT No. 121153-00
REV REV # 1
DRAWING No. 121153-GR2

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4.0 STORM SERVICING AND STORMWATER MANAGEMENT

The proposed storm servicing and stormwater management strategy for the Subject Site has been conceptually designed to adhere to the criteria established in the OSDG and associated technical bulletins.

4.1 Existing Drainage Conditions

Under existing conditions, storm runoff from the proposed development is split between the Wilson-Cowan Drain, Mud Creek, and Oxbow Ditch that outlets to Mud Creek immediately upstream of the confluence with the Rideau River, and the roadside ditch on the southwest side of Rideau valley Drive. Refer to **Figure 1.2** – Existing Conditions.

4.2 Previous Studies

The following supporting documents were utilized in the preparation of this report:

- WCMD
- MCMD
- MCFR Mapping
- Stinson Lands SWM Memo

4.3 Stormwater Management Criteria

As per previous discussions with the Rideau Valley Conservation Authority (RVCA) and the City of Ottawa (the City), there is no water quantity control proposed for the Subject Site as it discharges to the Oxbow Ditch that ultimately discharge within 35m to the Rideau River. An *"Enhanced"* level of water quality control corresponding to 80% long-term Total Suspended Solids (TSS) removal is required. Refer to meeting minutes from June 22, 2022 and June 29, 2022 included in **Appendix A**.

4.3.1 Minor System (Storm Sewers)

- Storm sewers are to be designed using the Rational Method and sized for the 2-year storm event (local streets),
- Inlet control devices (ICDs) are to be installed in road and rearyard catchbasins to control inflows to the storm sewers,
- Ensure that the 100-year hydraulic grade line in the storm sewer is at least 0.3 m below the underside of footing (USF) elevations for the proposed development.

4.3.2 Major System (Overland Flow)

- Overland flows are to be confined within the right-of-way and/or defined drainage easements for all storms up to and including the 1:100 year event,
- Maximum depth of flow (static + dynamic) on local and collector streets shall not exceed 0.35 m during the 100-year event. The depth of flow may extend adjacent to the right-of-way provided that the water level must not touch any part of the building envelope and must remain below the lowest building opening during the stress test event,
- Runoff that exceeds the available storage in the right-of-way will be conveyed overland along defined major system flow routes towards the proposed major system outlet to the Oxbow Ditch. There must be at least 15cm of vertical clearance between the spill elevation on the street and the ground elevation at the front of the building envelope that is in the proximity of the flow route or ponding area.

- The product of the 100-year flow depth (m) and flow velocity (m/s) within the right-of-way shall not exceed 0.60,
- Furthermore, 30cm of vertical clearance between the spill elevation and the ground elevation at the rear of the building envelope.

4.3.3 Water Quality & Quantity Control

- Provide an *'Enhanced'* (80% long-term total suspended solids removal) level of quality control to be provided by a Water Quality Treatment Unit (WQT) upstream of the storm sewer outlet,
- Implement lot level and conveyance Best Management Practices to promote infiltration and treatment of storm runoff.

4.4 Proposed Storm Drainage System

Existing drainage patterns will be altered somewhat under post development conditions, however runoff from the site will still be tributary to the same ultimate receiving watercourse (the Rideau River). The proposed changes to the drainage patterns have been generally agreed upon by the RVCA and the City.

Storm servicing for the proposed subdivision will be provided using a dual drainage system: Runoff from frequent storm events will be conveyed by storm sewers (minor system), while flows from larger storm events which exceed the capacity of the storm sewers will be conveyed overland along defined overland flow routes (major system) to the Oxbow Ditch and ultimately the Rideau River. There will be some uncontrolled runoff from rear yards and open space/ parks to the Wilson Cowan Drain, Oxbow Ditch, and Rideau Valley Drive roadside ditch with no quantity or quality control. Interior lot rear yards will flow into rear yard catch basin systems that will convey into the storm sewers (minor system).

4.4.1 Storm Sewers (Minor System)

The storm sewers comprising the minor system have been designed in accordance with Ottawa Sewer Design Guidelines (October 2012) and Technical Bulletins PIEDTB-2016-01 (September 2016), ISTB-2018-01 (March 2018), and ISTB-2018-04 (June 2018). The criteria used to design the storm sewers are summarized in **Table 4.1**. **Storm Sewer Design Parameters.**

Table 4.1: Storm Sewer Design Parameters

Parameter	Design Criteria
Local Roads	2 Year Return Period
Storm Sewer Design	Rational Method / PCSWMM
IDF Rainfall Data	Ottawa Sewer Design Guidelines
Initial Time of Concentration (T_c)	10 min
Minimum Velocity	0.8 m/s
Maximum Velocity	3.0 m/s
Minimum Diameter	250 mm
Minimum Pipe Cover	2.0 m (Unless frost protection provided)

Inlet Control Devices

Inlet control devices (ICDs) are to be installed in all catchbasins to limit inflows to the minor system capacity (2-year storm event). Exact ICD sizes and catchbasin locations will be determined during the detailed design stage.

4.4.2 Major System Design

The major system design will conform to the design standards outlined in the Ottawa Sewer Design Guidelines (October 2012) and Technical Bulletins PIEDTB-2016-01 (September 2016), ISTB-2018-01 (March 2018), and ISTB-2018-04 (June 2018). The proposed works will require approximately 1075 m of pipe ranging from 250 mm to 1200 mm diameter. During detailed design, the right-of-way will be graded to contain the major system runoff from storm events exceeding the minor system capacity for all storms up to and including the 100-year design event. The site will be graded to provide an engineered overland flow route for large, infrequent storms, or in the event that the storm sewer system becomes obstructed, with the majority of major system flows routed to the Oxbow Ditch.

Major System Flow Depths

For events exceeding the minor system design storm and up to the 100-year design storm flow depths in the right of way are to be limited to a maximum of 0.35m at the edge of pavement.

Infiltration Best Management Practices

Infiltration of surface runoff will be accomplished using lot level and conveyance controls. The most suitable practices for groundwater infiltration include:

- Infiltration of runoff captured by rear yard catchbasins;
- Direct roof leaders to rear yard areas;
- Infiltration trenches underlying drainage swales in park areas;
- The use of fine sandy loam topsoil in parks and on residential lawns.

By implementing infiltration Best Management Practices as part of the storm drainage design for the Subject Site, the impacts of development on the hydrologic cycle can be considerably reduced. Infiltration of clean runoff will also have additional benefits for stormwater management; by reducing the volume of "clean" water conveyed to the proposed WQT unit, the performance of WQT unit will be increased.

4.4.3 Water Quality Control

Water quality treatment will be provided using a prefabricated Water Quality Treatment Unit (WQT) installed upstream of the storm outlet to the Oxbow Ditch. The proposed WQT unit is an offline Vortechs model PC1421 (or approved equivalent) and would provide an *'Enhanced'* level of water quality treatment (80% long-term TSS removal) with a means of capturing oil and floatables upstream of Mud Creek and the Rideau River. Supporting correspondence and documentation for the Vortechs unit sizing are provided in **Appendix C**.

The Vortechs model PC1421 will have an internal orifice and internal weir, and the specifications of which were provided by the manufacturer (Contech). A bypass weir will be installed upstream in STM MH-169 to redirect high flows during storm events greater than a 25mm event. The invert of the bypass weir has been set based on the 25mm 6-hour Chicago storm HGL in STM MH-169. The length of the bypass weir is equivalent to the internal length of STM MH-169.

The WQT unit has been located within a grassed area and would be accessible from the right-of-way for inspection and maintenance. The layout of the WQT Unit, storm sewers, by-pass maintenance hole, and accessibility shall be refined during the detailed design stage of the Subject Site. For further details on the WQT unit refer to **Appendix C**.

4.4.4 Impact of the Municipal Drains and the Drainage Act

The proposed development will have no adverse impacts on the Wilson Cowan and Mud Creek Municipal Drains. The drainage areas and peak flows to these watercourses will be less than existing conditions, so there should be no requirement to revise the Engineer's Reports for these Municipal Drains at this time.

At the pre-consultation meeting with the City, a request was made to facilitate a pathway along the north side of Bankfield Road that will connect Rideau Valley Drive up to Millar's Point Park. An extension of the Wilson Cowan Drain culvert at Bankfield Road will be required to facilitate the pathway within the Bankfield Road ROW.

Notwithstanding the above, the Macro Servicing Plan indicates the proposed lot development limit and top of slope for the existing drains which demonstrates that access for future maintenance will be protected. Access to the Municipal Drains will be provided via the open space block through the setback between the development limits and the top of slope which remain relatively flat.

Robinson Consultants Inc. (RCI) have already appointed as the Drainage Engineer to the Wilson-Cowan Drain to address a change in land use as a result of upstream development. Additional communication and correspondence will be undertaken with Drainage Superintendent – Municipal Drainage and RCI to determine the impact and legislative requirements for both the Wilson-Cowan Drain and Mud Creek as a result of this development and land use change.

4.4.5 Alterations to Watercourses

The proposed development will require some alterations to the watercourses in order to fill an existing ditch and the construction of new outlet. The alterations are summarized below:

- An extension of the Bankfield Road culvert will be required to facilitate a pathway along the north side of Bankfield Road.
- Filling in an existing ditch between Lots 4-6.
- A new storm outlet to the Oxbow Ditch will be required. This storm outlet will be the primary outlet for the proposed development's minor and major flows.

4.5 Preliminary SWM Modeling

The *City of Ottawa Sewer Design Guidelines* (October 2012) require hydrologic modeling for all dual drainage systems. The performance of the proposed storm drainage system for the Subject Site was evaluated using the PCSWMM hydrologic/hydraulic model.

A pre-development model of the existing site was completed as a part of the previously submitted memorandum: *4386 Rideau Valley Drive N – Stinson Lands SWM Strategy Outline* (Novatech, June 8, 2022).

A post-development model of the proposed subdivision storm sewers and outlet to the Oxbow Ditch was developed using PCSWMM. The PCSWMM model represents both the minor and major system flows from the development. The results of the analysis were used to:

- Simulate major and minor system runoff from the site,
- Determine the storm sewer hydraulic grade line for the 100-year storm event,
- Ensure the WQT unit is sufficiently sized to treat storm runoff from the proposed development at an *'Enhanced'* level (80% TSS removal).

Model parameters and schematics for both pre- and post-development models have been provided in **Appendix C**.

4.5.1 Design Storms

The hydrologic analysis was completed using the following synthetic design storms and historical storms. The IDF parameters used to generate the Chicago and SCS Type II design storms were taken from the *Ottawa Design Guidelines - Sewer* (November 2004).

6 Hour Chicago Distribution:
- 25mm Event (Water Quality)
- 2-year Event
- 5-year Event
- 100-year Event
- 100-year Event +20%

12 Hour SCS Type II Distribution:
- 2-year Event
- 5-year Event
- 100-year Event

The 6-hour Chicago distribution generated the highest peak flows on a per-subcatchment basis, as well as the highest HGL elevations. Thus, the Chicago storm event was used in the design of the storm sewer system.

4.5.2 Downstream Boundary Conditions

Under existing conditions, a portion of the site (approx., 2.3 ha) drains to Mud Creek. However, under post-development conditions the majority of the site (6.16 ha) will drain to the existing Oxbow Ditch which outlets to Mud Creek just upstream of the confluence with the Rideau River.

The Mud Creek Flood Risk Mapping from Prince of Wales Drive to Rideau River (RVCA, July 9, 2019) report provides details of the HEC-RAS model prepared to analyze the water levels and peak flows within Mud creek for various storm events. The Oxbow Ditch is within the floodplain of Mud Creek and the Rideau River, so the water levels from this model would be similar to those expected in the Ditch during the storm events. Water levels from Table 10 and peak flows from Table 8 in the RVCA report are outlined in **Table 4.2**. Both *Cross Section 17595* and *Node J4* are approximately where Mud Creek meets the Rideau River:

Table 4.2: Downstream Boundary Conditions

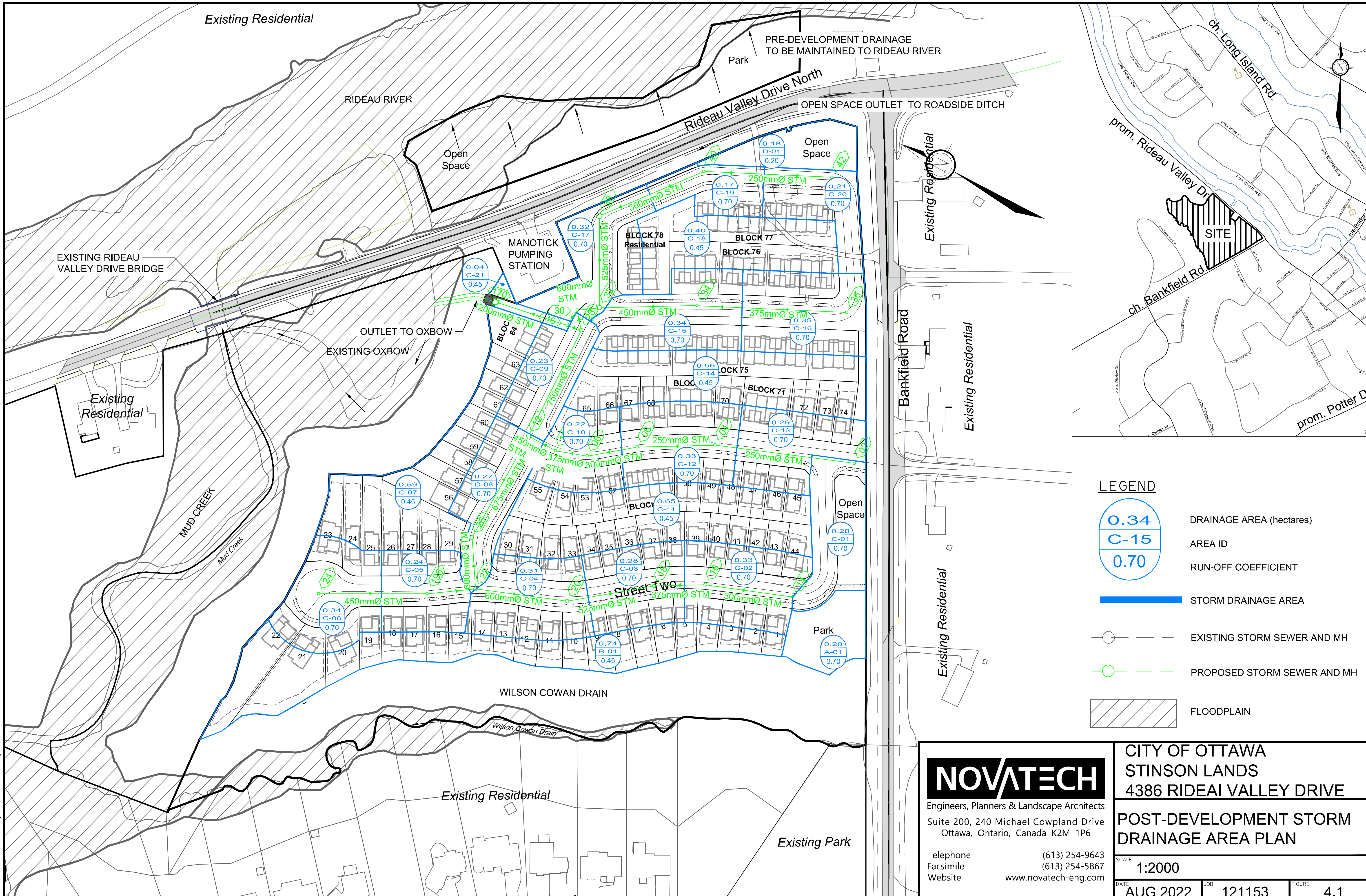
Storm Event	Water Level (m) Cross Section 17595	Peak Flow (cms) Node J4
2-year	82.20	12.13
5-year	82.56	23.06
100-year	83.22	59.70

A basic survey of the Oxbow Ditch has been completed which was used to include the ditch within the PCSWMM model from where the site will outlet to the Ditch and where the Ditch outlets to Mud Creek. At the outlet to Mud Creek the water levels from the Flood Risk Mapping report (RVCA) where Mud Creek meets the Rideau River were applied to give an idea of the impact (if any) the flows from the subdivision would have on the outlet water levels. Refer to **Appendix C** for details of the cross sections used in the PCSWMM model.

Due to the approximately 1.8 m drop from where the subdivision outlets to the ditch to the Oxbow Ditch, it is not expected that the water levels in Mud Creek will have an impact on the HGL of the storm sewers.

4.5.3 Storm Drainage Areas

The site has been divided into subcatchments based on the proposed land use and roadway design. The catchment areas shown on the Storm Drainage Area Plan **121153-STM** (**Figure 4.1)** correspond to the areas used in the Storm Sewer Design Sheet (**Appendix C**).



LEGEND

0.34	DRAINAGE AREA (hectares)
C-15	AREA ID
0.70	RUN-OFF COEFFICIENT
	STORM DRAINAGE AREA
	EXISTING STORM SEWER AND MH
	PROPOSED STORM SEWER AND MH
	FLOODPLAIN

NOVATECH

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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAI VALLEY DRIVE

POST-DEVELOPMENT STORM DRAINAGE AREA PLAN

SCALE 1:2000

DATE AUG 2022

JOB 121153

FIGURE 4.1

M:\2021\121153\CAD\Design\121153-STM.dwg, FIG 4.1, Nov 17, 2022 - 4:47pm, brundle

4.5.4 Model Parameters

The pre-development model developed for the 4386 Rideau Valley Drive N – Stinson Lands SWM Strategy Outline (Novatech, June 8, 2022) has not been changed since submission, and details are included in **Appendix C** for reference.

For the post-development model, the hydrologic parameters for each subcatchment were developed based on **Figure 1.3** – Site Plan and the Storm Drainage Area Plan (**112153-STM**). An overview of the modeling parameters is provided in **Table 4.3**.

Table 4.3: PCSWMM Subcatchment Area Parameters

Area ID	Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.200	0.70	71%	0%	38.18	52.38	1.0%
B-01	0.740	0.45	36%	100%	21.26	348.14	1.0%
C-01	0.280	0.70	71%	45%	23.78	117.77	1.0%
C-02	0.330	0.70	71%	45%	24.82	132.96	1.0%
C-03	0.280	0.70	71%	45%	23.62	118.54	1.0%
C-04	0.310	0.70	71%	45%	22.83	135.79	1.0%
C-05	0.240	0.70	71%	45%	23.31	102.96	1.0%
C-06	0.340	0.70	71%	45%	34.45	98.70	1.0%
C-07	0.590	0.45	36%	100%	39.87	148.00	1.0%
C-08	0.270	0.70	71%	45%	22.14	121.97	1.0%
C-09	0.230	0.70	71%	45%	22.42	102.59	1.0%
C-10	0.220	0.70	71%	45%	22.45	98.00	1.0%
C-11	0.650	0.45	36%	100%	19.05	341.19	1.0%
C-12	0.330	0.70	71%	45%	19.77	166.94	1.0%
C-13	0.290	0.70	71%	45%	23.66	122.59	1.0%
C-14	0.560	0.45	36%	100%	14.10	397.05	1.0%
C-15	0.340	0.70	71%	45%	22.21	153.06	1.0%
C-16	0.350	0.70	71%	45%	21.78	160.71	1.0%
C-17	0.320	0.70	71%	45%	22.95	139.43	1.0%
C-18	0.400	0.45	36%	100%	13.81	289.75	1.0%
C-19	0.170	0.70	71%	45%	19.18	88.63	1.0%
C-20	0.210	0.70	71%	45%	18.84	111.49	1.0%
C-21	0.040	0.45	36%	100%	45.28	8.83	1.0%
D-01	0.180	0.20	0%	0%	20.51	87.76	1.0%
TOTAL:	**7.87**						

Runoff Coefficient/ Impervious Values

Impervious (%IMP) values for each subcatchment area were calculated based on the Runoff Coefficients (see **Table 4.1**) noted on the Storm Drainage Area Plan (**121153-STM**) using the equation:

$$\%IMP = \frac{(C - 0.2)}{0.7}$$

Depression Storage

The default values for depression storage in the City of Ottawa were used for all catchments.

- Depression Storage (pervious areas): 4.67 mm
- Depression Storage (impervious areas): 1.57 mm

Residential rooftops are assumed to provide no depression storage and all rainfall is converted to runoff. The percentage of rooftop area to total impervious area is represented by the 'No Depression' column in **Table 4.3**.

Curve Number

The Carp River Watershed PCSWMM model uses an SCS Curve Number of 80.5. Thus, all subcatchments within the Kizell Lands have been given a curve number value of 80.5 to remain consistent with the Carp River Watershed model.

Equivalent Width

'Equivalent Width' refers to the width of the sub-catchment flow path. This parameter is calculated as described in the *Sewer Design Guidelines, October 2012, Section 5.4.5.6*

Major System

Since the major system has not yet been designed, the subcatchment areas are not based on a detailed grading plan. A very preliminary major system is represented in the PCSWMM model using a standard local roadway cross section with an inlet (catchbasin pair represented by a single junction) to the minor system for each subcatchment area. The top-of-grate elevation for each catchbasin pair has been based off the macro grading plan. Based on the macro grading, all catchbasins are currently on-grade with no low-point storage. The major system connections to the minor system have been given outlet rating curves based on a pair of City standard sized inlet control devices (ICDs) and sized based on the 2-year approach flow.

As the project is only at the Draft Plan stage, the detailed lot-level grading information is not yet available.

Modeling Files / Schematic

The PCSWMM model schematics are provided in **Appendix B**. Digital copies of the modeling files and model output for all storm events are provided with the digital report submission.

4.5.5 Model Results

The results of the PCSWMM model are summarized in the following sections.

Peak Flows

Under post-development conditions, the drainage areas and peak flows to Mud Creek, the Wilson Cowan Drain, and the Roadside ditch on Rideau Valley Drive will be less than existing conditions. Storm runoff from the perimeter of the site will continue to flow to these outlets, but most of the drainage will be routed to a proposed outlet to the Oxbow Ditch.

The Oxbow Ditch outlets to Mud Creek immediately upstream of the confluence with the Rideau River on the upstream side of the bridge under Rideau Valley Drive. Due to the proximity of the site to the Rideau River, no quantity control storage is proposed. The peak flows from the site will reach the Rideau River in advance of the peak flow from Mud Creek, so there should be no adverse impact to Mud Creek or the Wilson Cowan Drain resulting from the proposed development. A comparison of pre- vs. post-development peak flows is provided in **Table 4.4**.

Table 4.4: Pre vs. Post-Development Peak Flows (L/s)

Storm Distribution->		6hr Chicago					12hr SCS		
Return Period->		25mm	2yr	5yr	100yr	100yr +20%	2yr	5yr	100yr
Mud Creek	*Pre*	23	60	109	263	342	59	94	195
	Post	-	-	-	-	-	-	-	-
Oxbow	*Pre*	48	126	228	549	714	124	197	407
	Post	530	850	1,245	2,395	2,977	476	701	1,271
Wilson Cowan Drain	*Pre*	56	140	245	588	767	150	242	506
	Post	59	108	178	383	482	60	96	186
Rideau Valley Drive (culvert)	*Pre*	26	65	118	287	376	64	102	216
	Post	0	5	15	50	68	5	12	30

Hydraulic Grade Line

The PCSWMM model was used to evaluate the 100-year hydraulic grade line (HGL) elevations within the proposed storm sewers. As the design is only at the draft plan stage, the underside of footing (USF) elevations have not yet been determined. The HGL analysis will be revised at the detailed design stage to reflect the controlled inflows at each inlet to the storm sewers.

The model indicates that there will be some minor surcharging of the sewers during the 100-year event, as outlined in the following table.

Table 4.5: 100-year HGL Elevations

Manhole ID	MH Invert Elevation (m)	T/G Elevation (m)	Outlet pipe invert (m)	Outlet Pipe Diameter (m)	Outlet Pipe Obvert (m)	HGL Elevation (m)	WL Above Obvert (m)
135_(STM)	84.54	87.82	84.84	0.53	85.37	85.41	0.04
136_(STM)	86.44	89.77	86.74	0.30	87.04	86.90	-0.14
137_(STM)	89.89	93.22	90.19	0.25	90.44	90.29	-0.15
142_(STM)	84.33	87.92	84.63	0.60	85.23	85.24	0.01
144_(STM)	86.49	90.32	86.79	0.45	87.24	86.95	-0.29
145_(STM)	89.65	93.28	89.95	0.38	90.33	90.06	-0.27
146_(STM)	84.38	87.82	84.68	0.75	85.43	85.68	0.25
148_(STM)	85.03	87.85	85.33	0.38	85.71	85.83	0.13
149_(STM)	85.87	88.62	86.17	0.30	86.47	86.45	-0.02
150_(STM)	87.62	90.70	87.92	0.25	88.17	88.08	-0.09
151_(STM)	84.65	87.96	84.95	0.68	85.63	85.84	0.22
152_(STM)	84.81	88.20	85.11	0.60	85.71	85.97	0.26
153_(STM)	85.23	88.52	85.53	0.45	85.98	86.11	0.13
154_(STM)	85.74	89.18	86.04	0.38	86.42	86.33	-0.09
156_(STM)	86.08	89.56	86.38	0.30	86.68	86.57	-0.11
159_(STM)	85.27	88.31	85.57	0.45	86.02	86.02	0.00
169_(STM)	84.04	87.65	84.48	0.75	85.23	85.01	-0.22
170_(STM)	83.50	86.83	83.80	1.20	85.00	83.91	-1.09
186_(STM)	83.67	88.05	83.97	1.20	85.17	84.76	-0.41
187_(STM)	83.90	86.96	83.90	1.20	85.10	84.53	-0.57

Manhole ID	MH Invert Elevation (m)	T/G Elevation (m)	Outlet pipe invert (m)	Outlet Pipe Diameter (m)	Outlet Pipe Obvert (m)	HGL Elevation (m)	WL Above Obvert (m)
189_(STM)	85.51	88.96	85.81	0.45	86.26	86.19	-0.07
191_(STM)	85.40	88.13	85.70	0.30	86.00	86.06	0.06
193_(STM)	86.51	89.34	86.81	0.25	87.06	86.97	-0.09

As shown in the above table, the 100-year HGL elevations are at or below 0.30m above the pipe obvert. During the detailed design stage, pipe sizes and building elevations may be refined to ensure the 100-year HGL will be at least 0.30m below the design USF elevations.

Outlet Water Levels & Impact

As discussed in **Section 4.5.2**, the Oxbow Ditch outlets to Mud Creek just upstream of its confluence with the Rideau River. As such, it is directly affected by the water levels in both the Rideau River and Mud Creek. There is also the concern that the additional flows from the Subject Site to the Oxbow Ditch and ultimately Mud Creek would have a negative impact on the creek and possibly result in an increase in erosion.

As the design is only in the Draft Plan stage, a very preliminary analysis was done to determine if there would be much impact to the receiving watercourses. For each storm event, the outlet water level was compared to the outlet boundary condition to determine if the flows from the subdivision are high enough to have an impact on the water level. Results are outlined in the following table.

Table 4.6: Boundary Conditions vs. PCSWMM Model Output

Storm Event	Water Level (m) Cross Section 17595	Water Level at Outlet	Peak Flow (cms) Node J4	Peak Flow (cms) Model Outlet
2-year	82.20	82.20	12.13	0.929
5-year	82.56	82.56	23.06	1.411
100-year	83.22	83.22	59.70	2.578

As shown in **Table 4.6** there are no changes to the outlet water level and the peak flows from the site are much lower than those in Mud Creek. As such, it is expected that there will be little to no impact to the existing conditions due to an increase in flows from the site to the Oxbow Ditch and Mud Creek.

Matrix is currently completing a Fluvial Geomorphology Assessment and Meander Belt Width Analysis for the site and surrounding watercourses to determine the existing site conditions and to determine the meander belt width and 100-year erosion limit for reaches of the Wilson-Cowan Drain and Mud Creek.

At the detailed design stage and after the completion of the assessment by Matrix, further analysis will be completed to ensure there will be no negative impacts to the Oxbow Ditch, Mud Creek, or the Rideau River due to the increase in peak flows from the proposed development.

Outlet Velocities

Matrix has completed a Fluvial Geomorphic and Erosion Hazard Assessment for the proposed outlet to the Oxbow Ditch. Overall, the analysis showed that a critical velocity of 0.91m/s should not be exceeded in the Ditch to ensure that outlet flows do not cause erosion.

Outlet velocities from the site have been analyzed using the PCSWMM model to determine what they would be with no mitigation. Results are outlined in the following table, with reference to the figure below:

Table 4.7: Outlet Velocities (m/s)

	Minor System Outlet	Major System Outlet	Outlet Channel	Max. in Oxbow Ditch
25mm	1.55	0.27	1.80	0.83
2-year	1.78	0.37	1.00	0.24
5-year	1.96	0.63	0.75	0.21
100-year	2.04	1.80	1.09	0.21

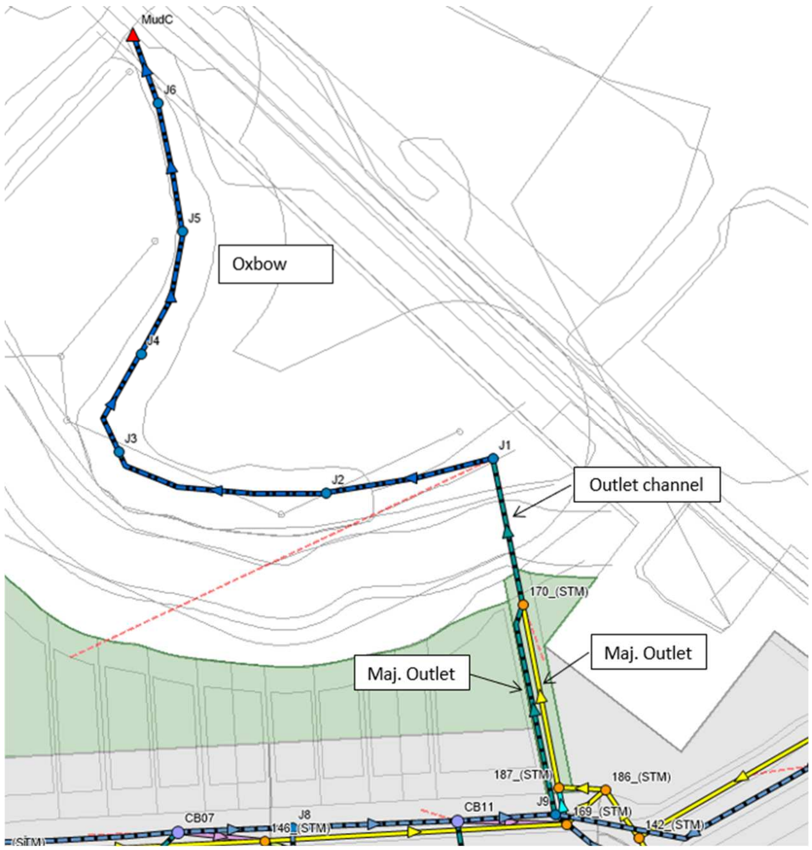
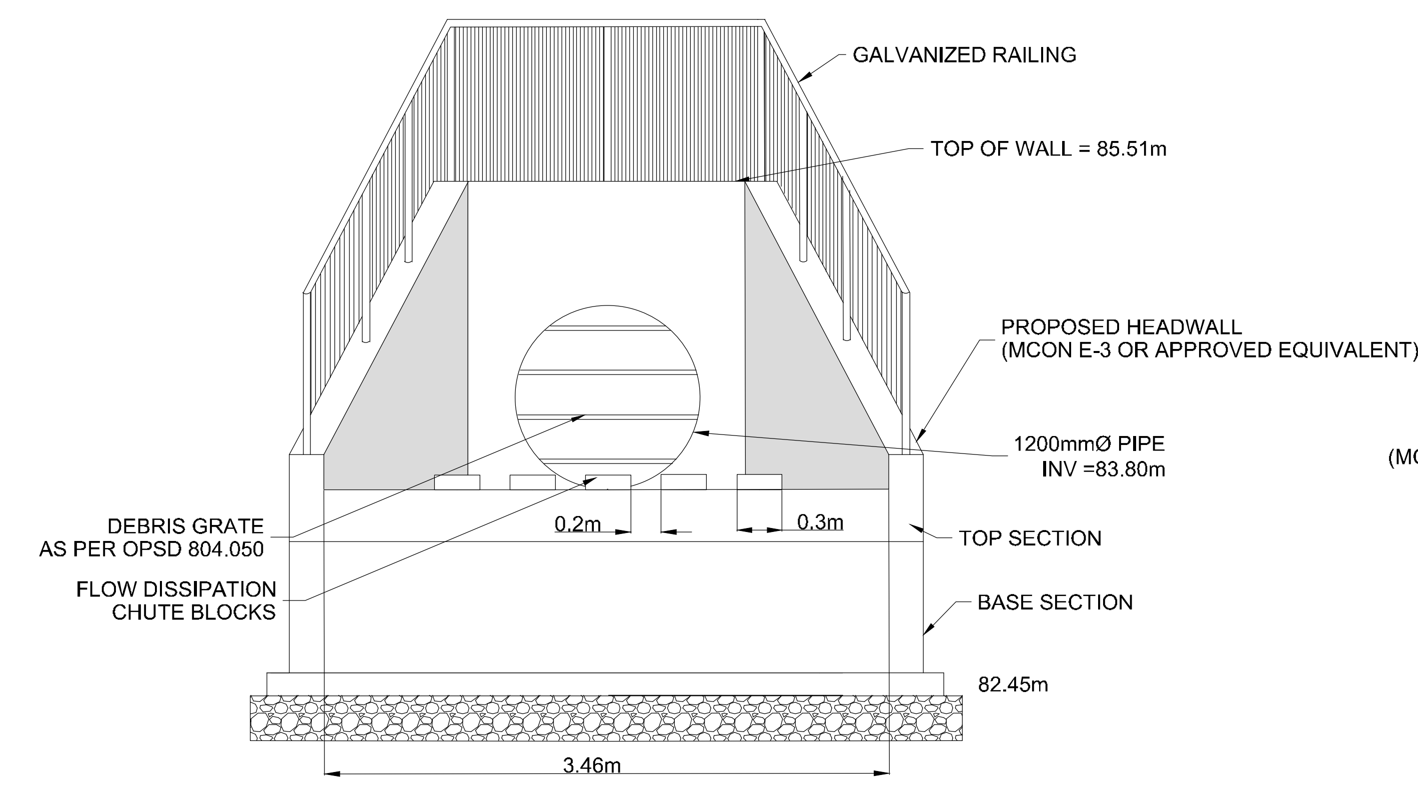


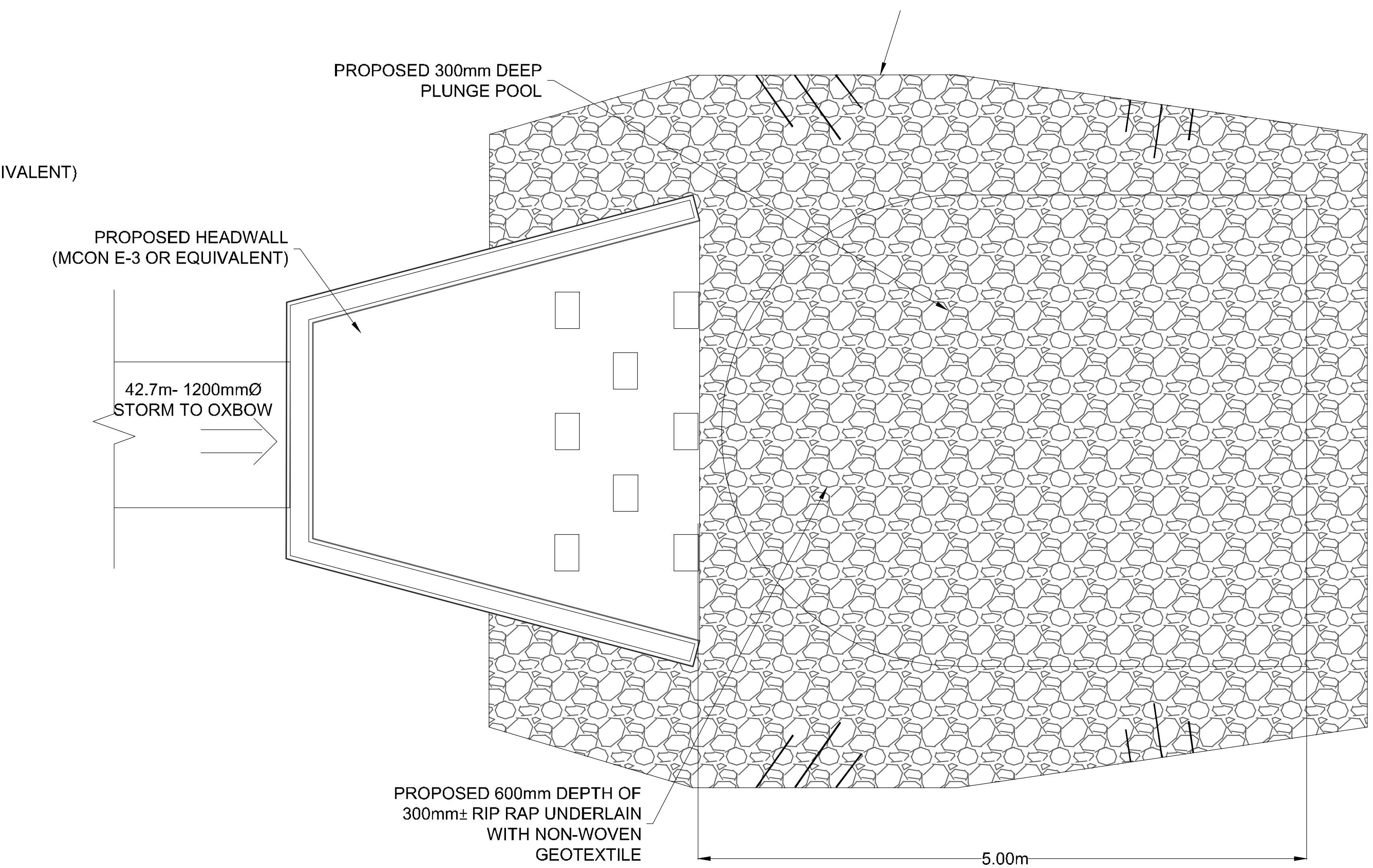
Figure 4.2: Outlet Velocity Locations

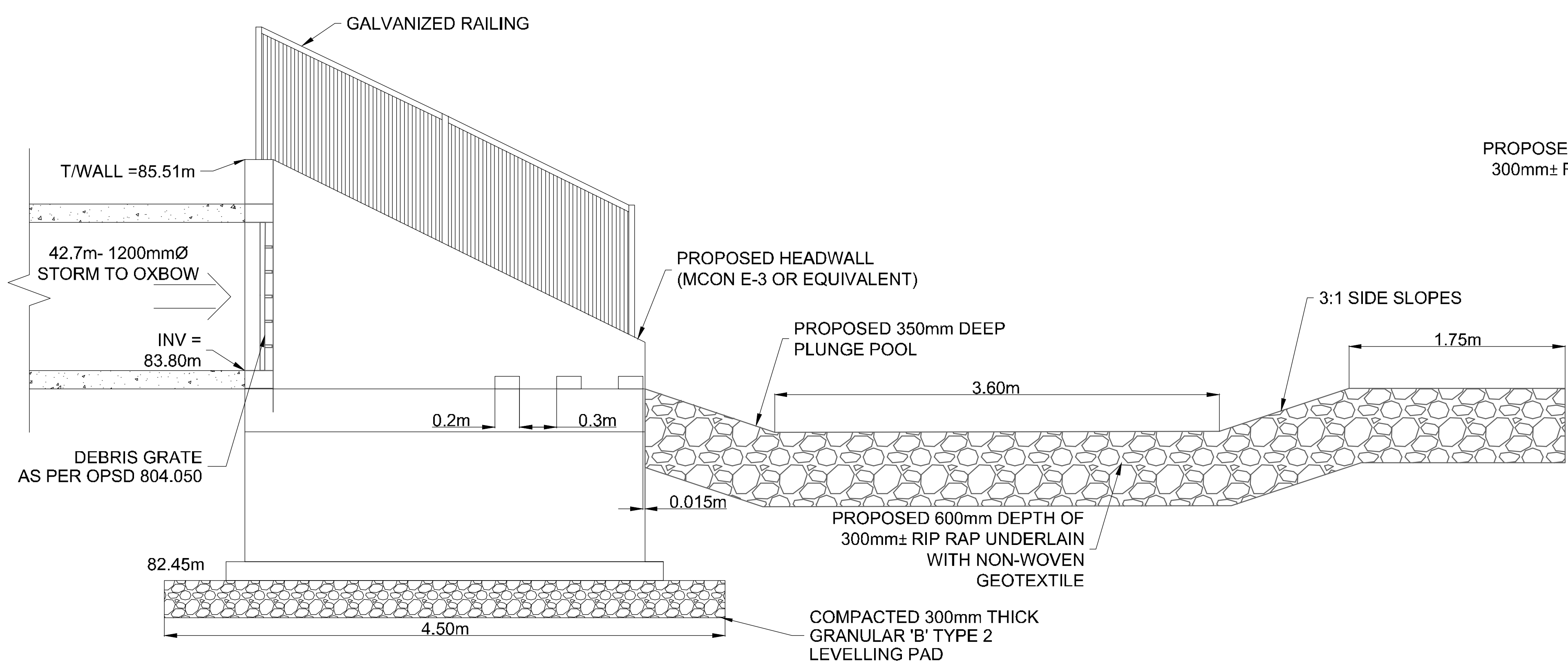
As shown in the table, outlet velocities in the Outlet Channel, downstream from the major and minor system outlets, will exceed 0.91m/s. While velocities do slow down in the Oxbow, this is due to the backwater from Mud Creek and the Rideau River.

To ensure that outlet velocities are reduced to an acceptable level and there is no risk of erosion to the Oxbow, a plunge pool will be installed where the major and minor system outlet meet. Sizing of the plunge pool was done to ensure velocities for all storm events up to the 100-year would be reduced to at or below 0.91m/s. Refer to Appendix C for sizing calculations, and **Figure 4.3 - Proposed Outlet to Oxbow with Plunge Pool** for the proposed plunge pool design. The outlet channel downstream from the plunge pool will also be lined with rip-rap to further decrease outlet velocities before runoff enters the Oxbow Ditch.

During detailed design stage, additional assessment to address erosion mitigation measures will be completed to ensure there will be no negative impacts to the Oxbow Ditch, Mud Creek, or the Rideau River due to the increase in peak flows from the proposed development.







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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

PROPOSED OUTLET TO OXBOW WITH PLUNGE POOL

SCALE 1:50		
DATE SEP 2022	JOB 121153	FIGURE 121153-OUTLET

SHT11X17.DWG - 279mmX432mm

5.0 SANITARY SEWER SYSTEM

5.1 Existing Sanitary Sewers

The sanitary outlet for the Subject Site is an existing 600 mm trunk sanitary sewer located within Rideau Valley Drive ROW, approximately 15 m northeast of the Subject Site. It will connect to existing maintenance hole MHSA58925, and from there will flow through the existing trunk sewer to the Manotick Pumping Station located 85m away at 4344 Rideau Valley Drive.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services for an illustration of the proposed sanitary connection and layout details.

5.2 Existing Manotick Sanitary Pumping Station

The existing Manotick Pump Station currently has a firm capacity of 56 L/s (1 operational pump and 1-305mm forcemain), however, based on the pre-consultation minutes, City Staff have indicated that the Manotick Pumping Station located is planned to be upgraded to have an interim capacity of 170 L/s by Q4 2024.

Based on the existing and projected demands of the serviced lands tribiutary to the Manotick Pumping Station, a sanitary design sheet has been prepared to calculate the combined peaked sanitary flows from the Core, Hillside Gardens, Minto Mahogany Lands, Riverwalk, and various servicing connections between the said areas. Furthermore, the Subject Site has been added as a proposed flow to the station. Refer to **Figure 5.1** – Manotick PS Servicing Areas for reference to the areas studied and the design sheet within **Appendix D**. The combined peak flow of the existing and projected areas is 157 L/s; therefore, the interim 170L/s upgrade would allow the Subject Site to be serviced by the municipal wastewater collection system.

Additional discussions can be held with the City (Wastewater Collection and Development Review) to determine if Manotick Pump Station can be operated with the larger forcemain during wet weather flows to provide an increased residual flow, in advance of the interim upgrade.

5.3 Proposed Sanitary Infrastructure

Off-site works

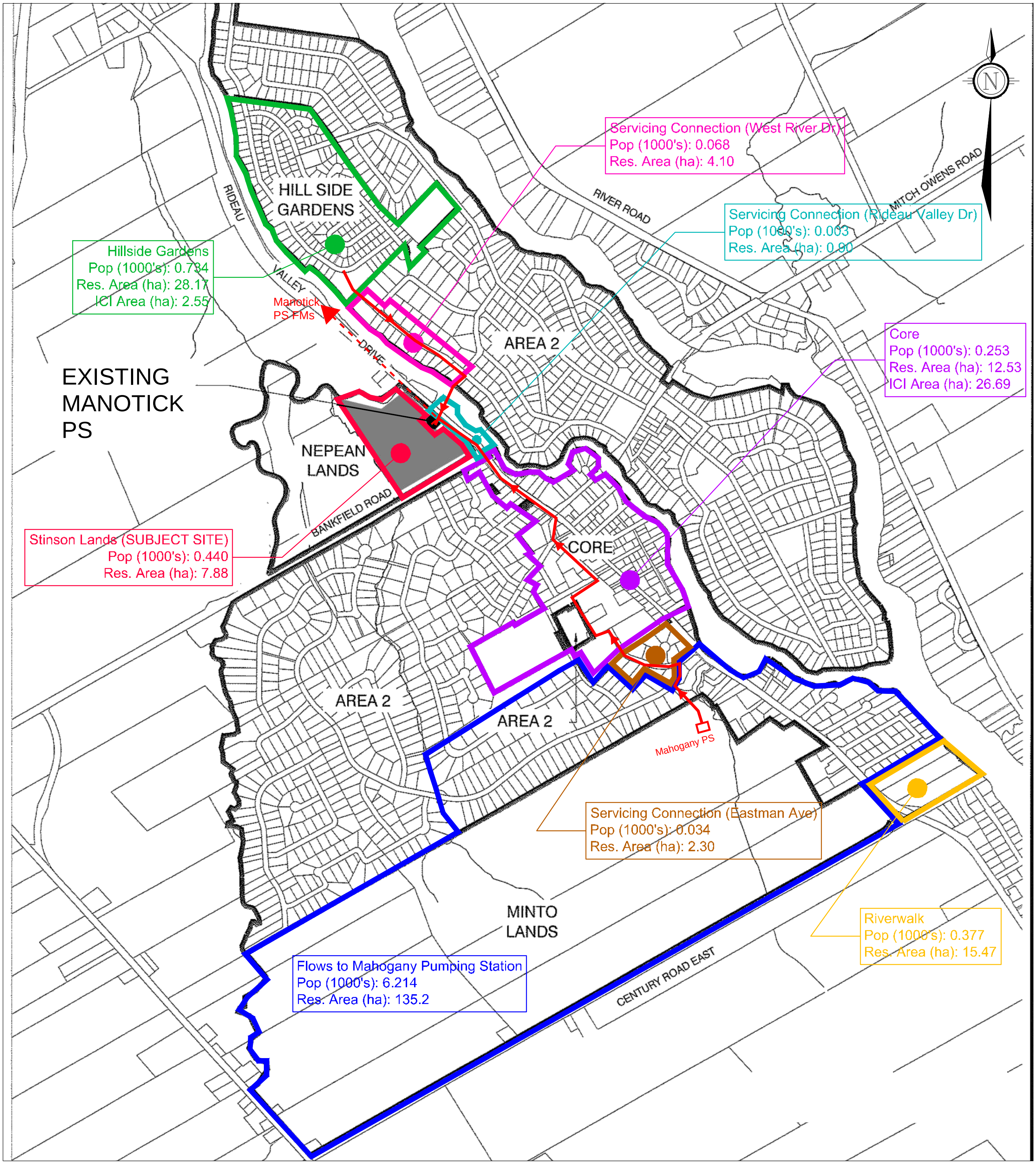
The proposed off-site works will require connecting a 25 m long, 250 mm diameter pipe to an off-site trunk sanitary sewer within the Rideau Valley Drive ROW at existing maintenance hole MHSA58925. The extension will require reinstatement of the existing road to match existing conditions or better.

On-site works

The proposed on-site works will require approximately ~1000 m of 200 mm and 250 mm diameter on-site sanitary sewer to collect and direct wastewater flows to the outlet pipe located in the north-east corner of the Subject Site, which shall connect to the Off-site works described above.

5.4 Sanitary Demand and Design Parameters

The peak design flow parameters in **Table 5.1** have been used in the sewer capacity analysis. Unit and population densities and all other design parameters are specified in the OSDG.



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Engineers, Planners & Landscape Architects
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Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

MANOTICK PS SERVICING AREAS

STINSON LANDS SUBDIVISION

DATE	JOB	FIGURE
JAN 2023	121153	FIG 5.1

SHT8X11.DWG - 216mmx279mm

Table 5.1: Sanitary Sewer Design Parameters

Design Component	Design Parameter
Unit Population: Single Detached Home Semi-Detached /Townhomes 2-BR Apartments	 3.4 people/unit 2.7 people/unit 2.1 people/unit
Residential Flow Rate, Average Daily	280 L/cap/day
Residential Peaking Factor	Harmon Equation (min=2.0, max=4.0) Harmon Correction Factor, k = 0.8
Minimum Pipe Size	200mm (Res)
Minimum Velocity[1]	0.6 m/s
Maximum Velocity	3.0 m/s
Minimum Pipe Cover	2.5 m (Unless frost protection provided)

[1]A minimum gradient of 0.65% is required for any initial sewer run with less than 10 residential connections.

The sanitary sewer design sheet, located in **Appendix D** confirms the peaked sanitary flows from the Subject Site will be 7.45 L/s. Refer to **Figure 5.2** – Post-Development Sanitary Drainage Area Plan for reference to the Subject Site.

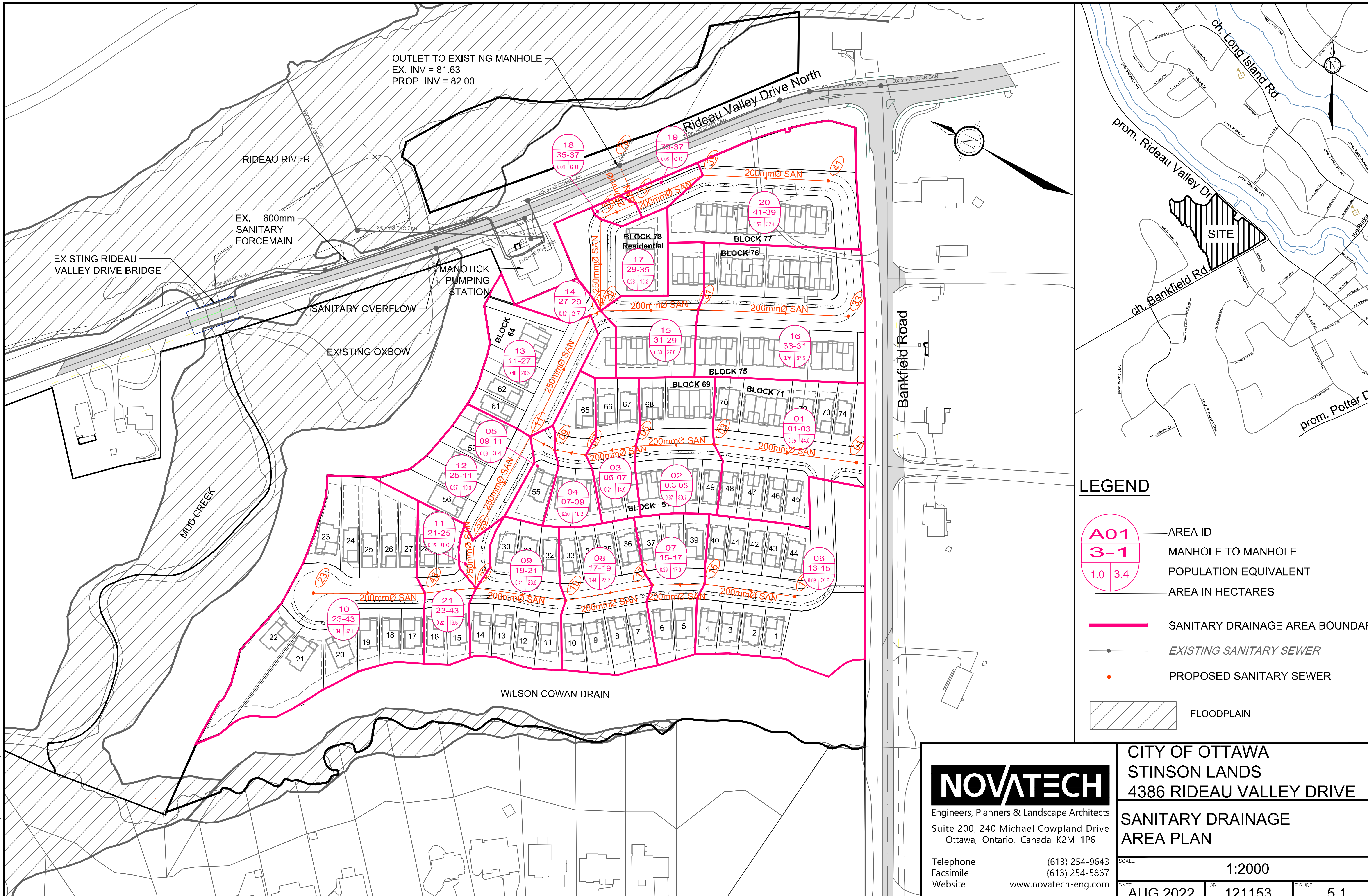
5.5 Hydraulic Grade Line (HGL)

The emergency overflow elevation at the Manotick Pumping Station is located at the by-pass maintenance hole (MHSA58901) within the station's compound which is directed to the Oxbow Ditch. The elevation of the overflow is 83.57, based on GeoOttawa Mapping, which is set above the 100-year water level of Mud Creek. The Manotick PS Report includes plans and profiles of the sanitary HGL during an emergency overflow condition. The HGL at the node 267, where the Subject Site's sanitary sewer will connect is approximately 84.00m. The HGL within the Subject Site may increase in the magnitude of 0.35m to account for minor losses within the local sanitary system of the Subject Site; therefore, the HGL within the Subject Site shall be assumed to be in the magnitude of 84.35m. This HGL elevation will be utilized to compare the basement elevations of the Subject Sites to ensure that sewer backups do not impact the units.

The lowest centreline of road elevation within the Subject Site is 87.70. The lowest underside of footing (USF) is conservatively set at 2.35 m below the centreline of road which would yield a USF elevation of 85.35 m.

The available freeboard between the on-site HGL and the lowest USF is 1.00 m. This exceed the OSDG requirements of 0.3m.

Although the foregoing is a high-level comparison to determine the available freeboard, an additional analysis can be completed during the detailed design stage of the Subject Site to ensure that the wastewater collection system meets the OSDG requirements.



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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

SANITARY DRAINAGE AREA PLAN

SCALE		
1:2000		
DATE AUG 2022	JOB 121153	FIGURE 5.1

M:\2021\121153\CAD\Design\121153-SAN.dwg, FIG 5.1, Nov 17, 2022 - 4:46pm, brundle

6.0 WATER SUPPLY SYSTEM

6.1 Existing Water Infrastructure and City Planned Construction

The City has a 400 mm diameter trunk watermain along Rideau Valley Drive fronting the Subject Site. The watermain connections for the Subject Site will both be along the northeast side of the project along this trunk watermain (Connections 1 & 2).

The City has provided boundary conditions with respect to existing and future conditions. The City has a cited concern with a lack of redundancy for the Village of Manotick. To improve the redundancy for the area, the Phase 2 of the Manotick Feedermain project will need to be completed. Based on the pre-consultation minutes, City Staff have indicated that Manotick Feedermain will be completed by Q4 2024.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services for an illustration of the proposed water supply system connections and layout details.

6.2 Proposed Water Infrastructure

Off-site works

There will be two connections made to the 400mm watermain: Connection 1 will be near the sanitary pipe that will be connecting to an existing manhole, and Connection 2 will be approximately 35m further south on the same section of street. Additional valving or a revised water connection configuration between the two connections and the on-site watermain may need to be considered during the detailed design stage.

Depending on the timing of the Subject Site servicing and the Manotick Feedermain status, connection details and methods can be determined with the City in due course.

On-site works

The proposed on-site works will require approximately 1130m of 200mm and 250mm on-site watermain. Proposed hydrant locations have been provided. An additional fire hydrant has been provided along Street Two’s dead-end portion to ensure the required fire flow for the furthest lot (lot 22). These locations will be confirmed during detailed design.

6.3 Watermain Design Parameters

Boundary conditions were provided by the City based on the OWDG water demand criteria for both existing and future conditions. For the purpose of this report both the existing and future conditions were analysed and the results provided. The boundary conditions are included in **Appendix E**.

The domestic demand design parameters, fire fighting demand design scenarios, and system pressure criteria design parameters are outlined in **Table 6.1** below. The system pressure design criteria used to determine the size of the watermains, required within the Subject Site, and are based on a conservative approach that considers three possible scenarios.

Table 6.1: Watermain Design Parameters and Criteria

Domestic Demand Design Parameters	Design Parameters
Population: Single Detached Home Semi-Detached /Townhomes 2-BR Apartments	 3.4 people/unit 2.7 people/unit 2.1 people/unit
Average Day Residential Demand (AVDY)	280 L/c/d
Maximum Day Demand (MXDY)	2.5 x Average Day
Peak Hour Demand (PKHR)	2.2 x Maximum Day
Fire Demand Design	**Design Flows**
Conventional single/town units, unless otherwise noted. Hydrant spacing and coding	10,000 L/min per FUS / OWDG TB-2014 90 to 120 m spacing per OWDG
System Pressure Criteria Design Parameters	**Criteria**
Maximum Pressure (AVDY) Condition	< 80 psi occupied areas < 100 psi unoccupied areas
Minimum Pressure (PKHR) Condition	> 40 psi
Minimum Pressure (MXDY+FF) Condition	> 20 psi

The firefighting water demands for the Subject Site have been estimated per OWDG which refers to the Fire Underwriters Survey (CGI, 2020) document, abbreviated as FUS.

In accordance with the FUS and based on the proposed zoning, there is potential for less than 3m of separation between the single family, semi-detached, and row townhome wood-framed buildings, which would require the fire area in the FUS estimate for multiple buildings to be treated as a contiguous block area. This results in a high fire flow demand which is difficult to attain from the existing system; moreover, it would trigger larger diameter watermain size within the Subject Site creating system vulnerabilities such as water age issues. As per the ISTB-2014-02, fire flows may be capped at 167 L/s (10,000 L/min) for single family, semi-detached, and row townhome provided certain site criteria are met.

The criteria are:

- For singles: a min separation of 10m between the backs of adjacent units.
- Traditional side-by-side semi-detached or row townhomes:
    a. firewalls with a min two-hour rating to separate the block into fire areas of no more than the lesser of 7 dwelling units, or 600 m^2 of building area; and
    b. Min separation of 10 m between the backs of adjacent units.

The proposed layout of the Subject Site will meet the minimum separation of 10 meters between the backs of adjacent units. As such, the proposed layout shall meet the foregoing criteria allowing the capped fire flow of 167 L/s to be used for these unit types of residential units. Detailed FUS calculations can be found attached in **Appendix E**.

6.4 System Pressure Modeling and Results

System pressures for the Subject Site were estimated using the EPANET engine within PCSWMM.

The PCSWMM model layout is demonstrated in **Figure 6.1** – Proposed Watermain Sizing, Layout and Junction IDs and **Figure 6.2** – Ground Elevations (m).

Domestic Demand

The water demand summary for the complete build out of the Subject Site for the average daily and peak hour demands has been provided in **Table**Error! Reference source not found. **6.2** below. For detailed results refer to the tables provided in **Appendix E.** The detailed results are also demonstrated in **Figure 6.3** – Maximum Pressures During AVDY Condition and **Figure 6.4** – Minimum Pressures During PKHR Condition. Figures under existing conditions have been provided in **Appendix E**.

Table 6.2: System Pressure (EPANET)

Condition	Demand (L/s)	Allowable Pressure (psi)	Max/Min Pressure (psi)
Existing Conditions			
AVDY	1.43	80 (Max)	101
PKHR	7.71	40 (Min)	66
Future Conditions			
AVDY	1.43	80 (Max)	89
PKHR	7.71	40 (Min)	66

The hydraulic analysis demonstrates that the proposed watermain sizing meets the design criteria for both conditions. It is noted that the system pressures during the Maximum Pressure (AVDY) in both conditions exceeds the maximum allowable service pressure. As such, pressure reducing valves (PRVs) will be required. PRV locations will be confirmed during detailed design.

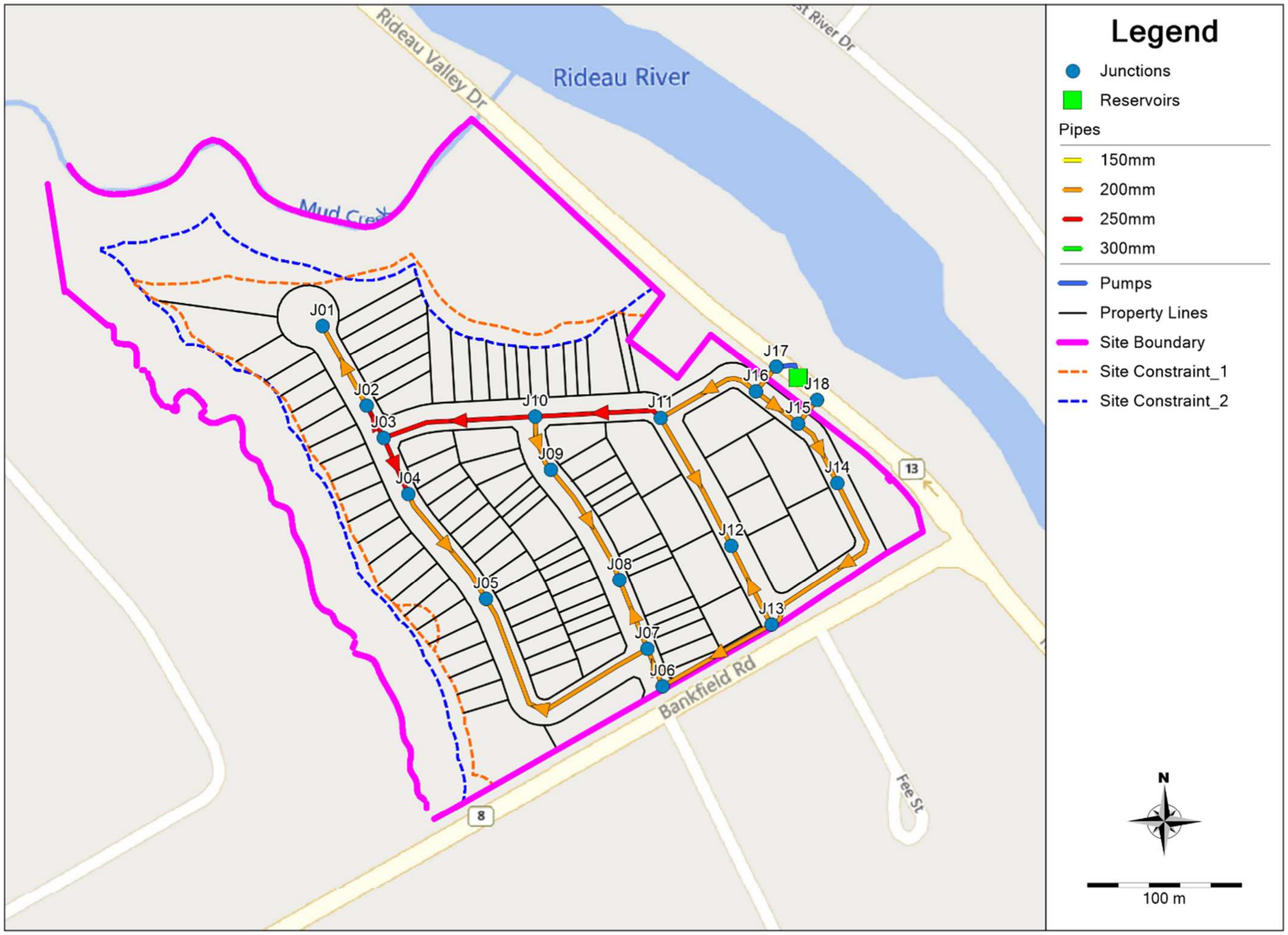
Fire Demand

An analysis was carried out to determine the available fire flow under maximum day demand while maintaining a residual pressure of 20psi. This was completed using the EPANET fire flow analysis feature within PCSWMM.

For detailed results refer to the tables provided in **Appendix E.** The detailed results are also demonstrated in **Figure 6.5** – Available Flow at 20psi During MXDY+FF Condition. Figures under existing conditions have been provided in **Appendix E**.

To achieve the required fire flow and optimize watermain sizes, the OWDG and its subsequent revisions (specifically ISTB-2018-02) allow for multiple hydrants to be drawn from, as opposed to drawing from a single hydrant to meet the required demand. Upon review of the results from the hydraulic analysis the required fire flows can be achieved for the proposed structures by utilizing multiple hydrants. An excerpt from ISTB-2018-02 of Appendix I: Guideline on Coordination of Hydrant Placement with Required Fire Flow has been included in **Appendix E**, for reference on the maximum flow that can be considered from a given hydrant. Hydrant locations will be reviewed and confirmed during detailed design.

Stinson Lands (121153)
PCSWMM Model Schematic



<u>Proposed Watermain Sizing, Layout and Junction IDs</u>

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Stinson Lands (121153)
PCSWMM Model Schematic

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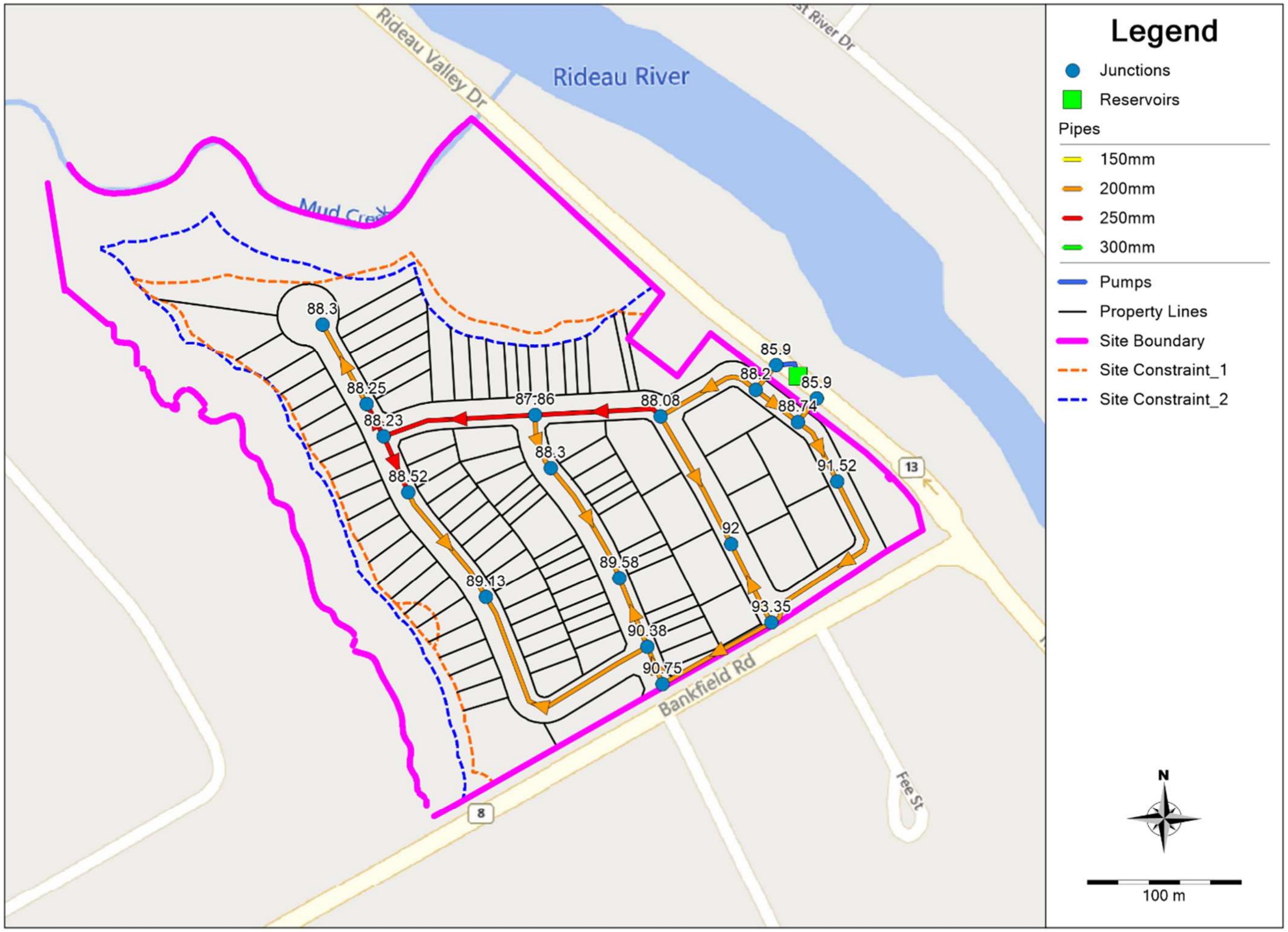

Ground Elevations (m)

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Stinson Lands (121153)
PCSWMM Model Schematic

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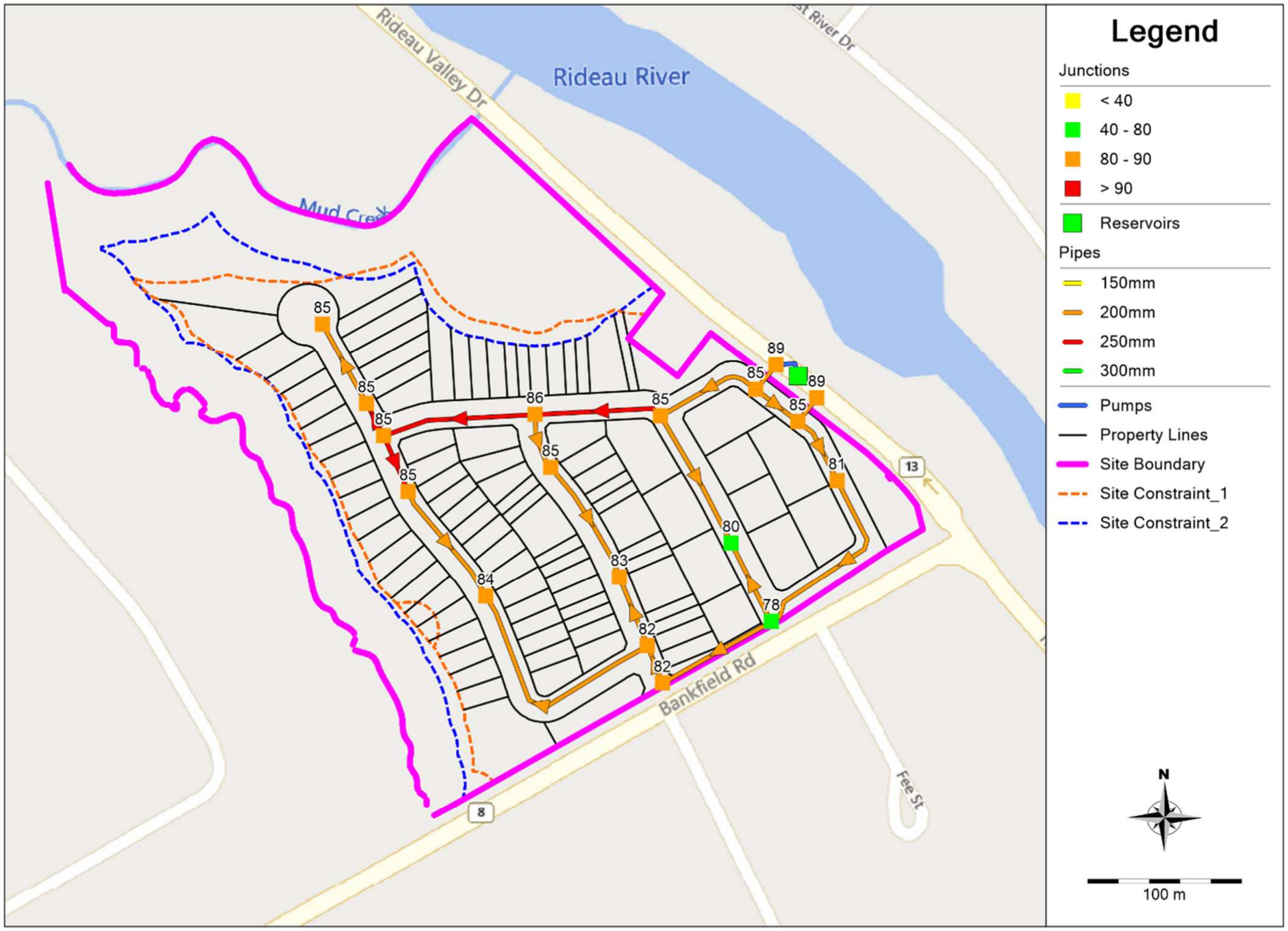

<u>Maximum Pressure During AVDY Conditions – Future</u>

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Stinson Lands (121153)
PCSWMM Model Schematic

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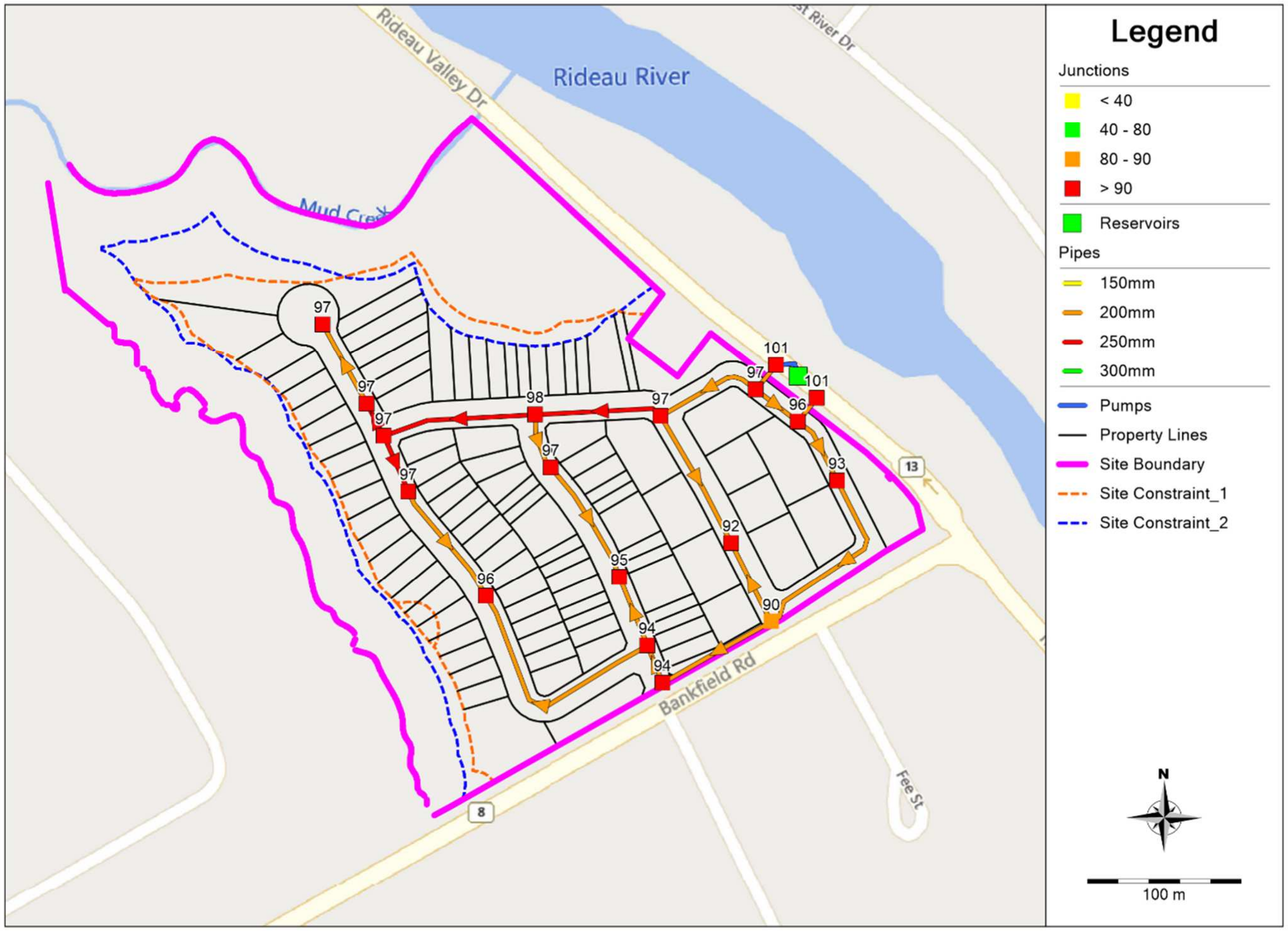

Maximum Pressure During AVDY Conditions – Existing

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Stinson Lands (121153)
PCSWMM Model Schematic

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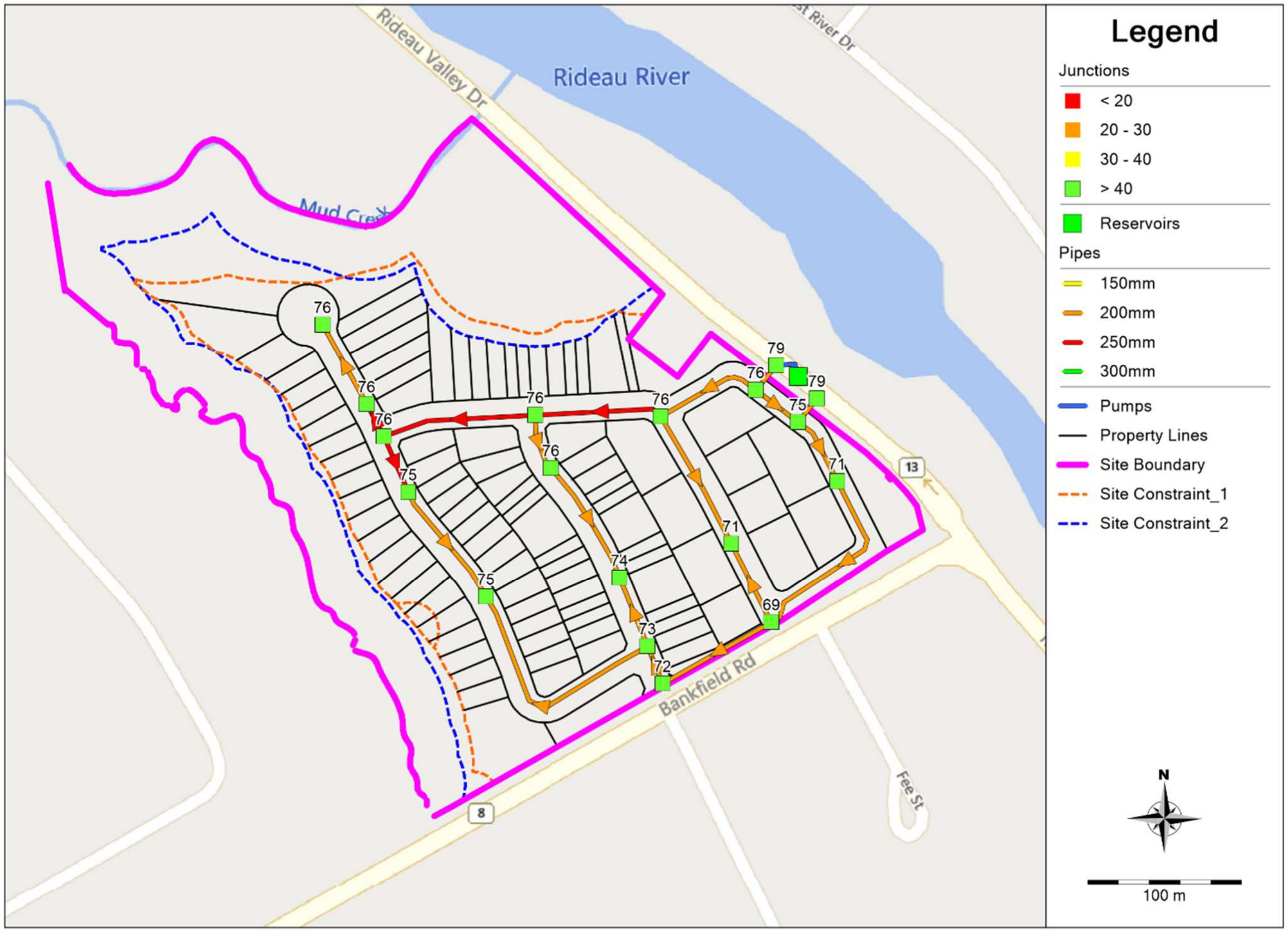

Minimum Pressure During PKHR Conditions – Future

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Stinson Lands (121153)
PCSWMM Model Schematic

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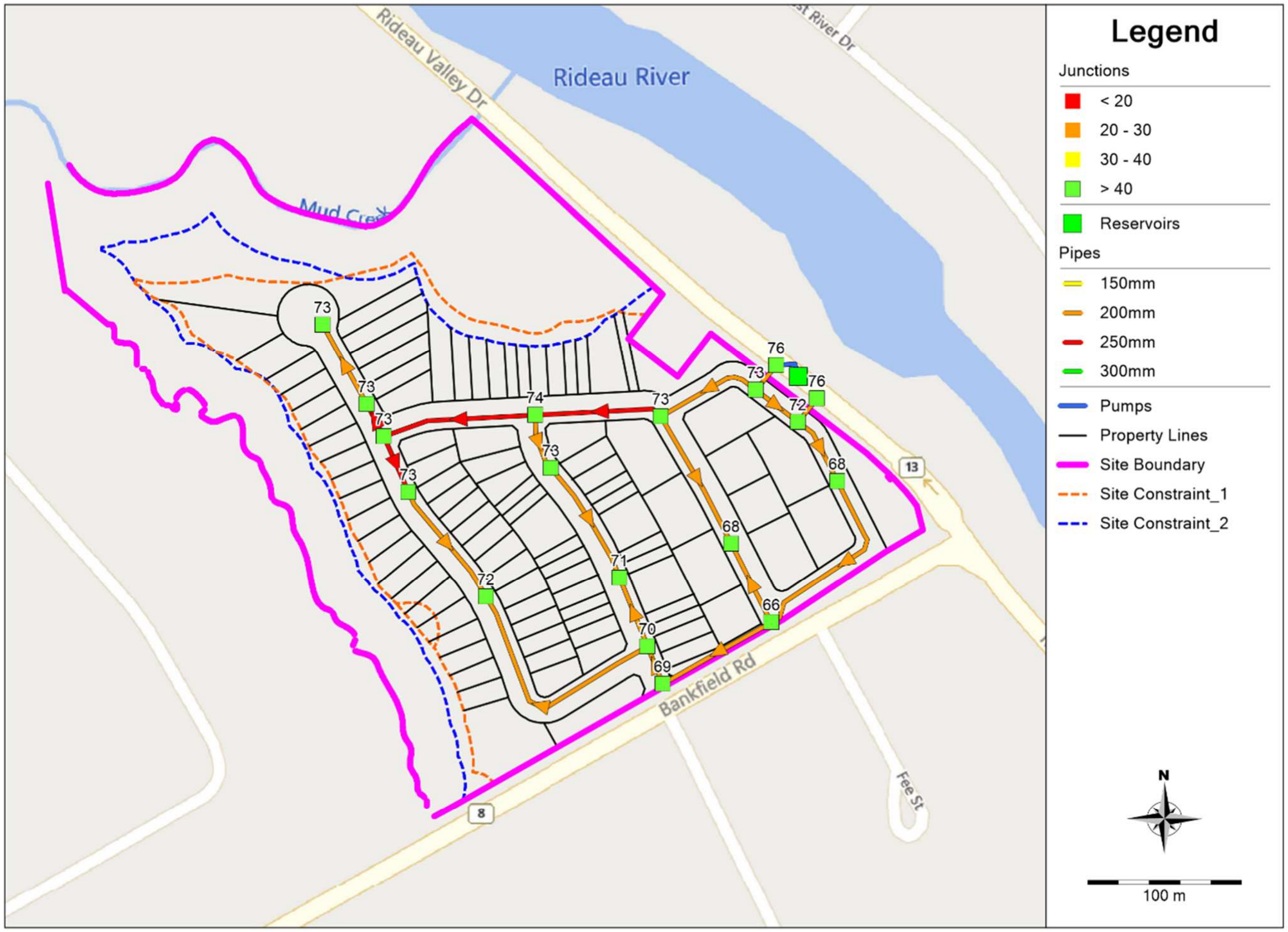

<u>Minimum Pressure During PKHR Conditions – Existing</u>

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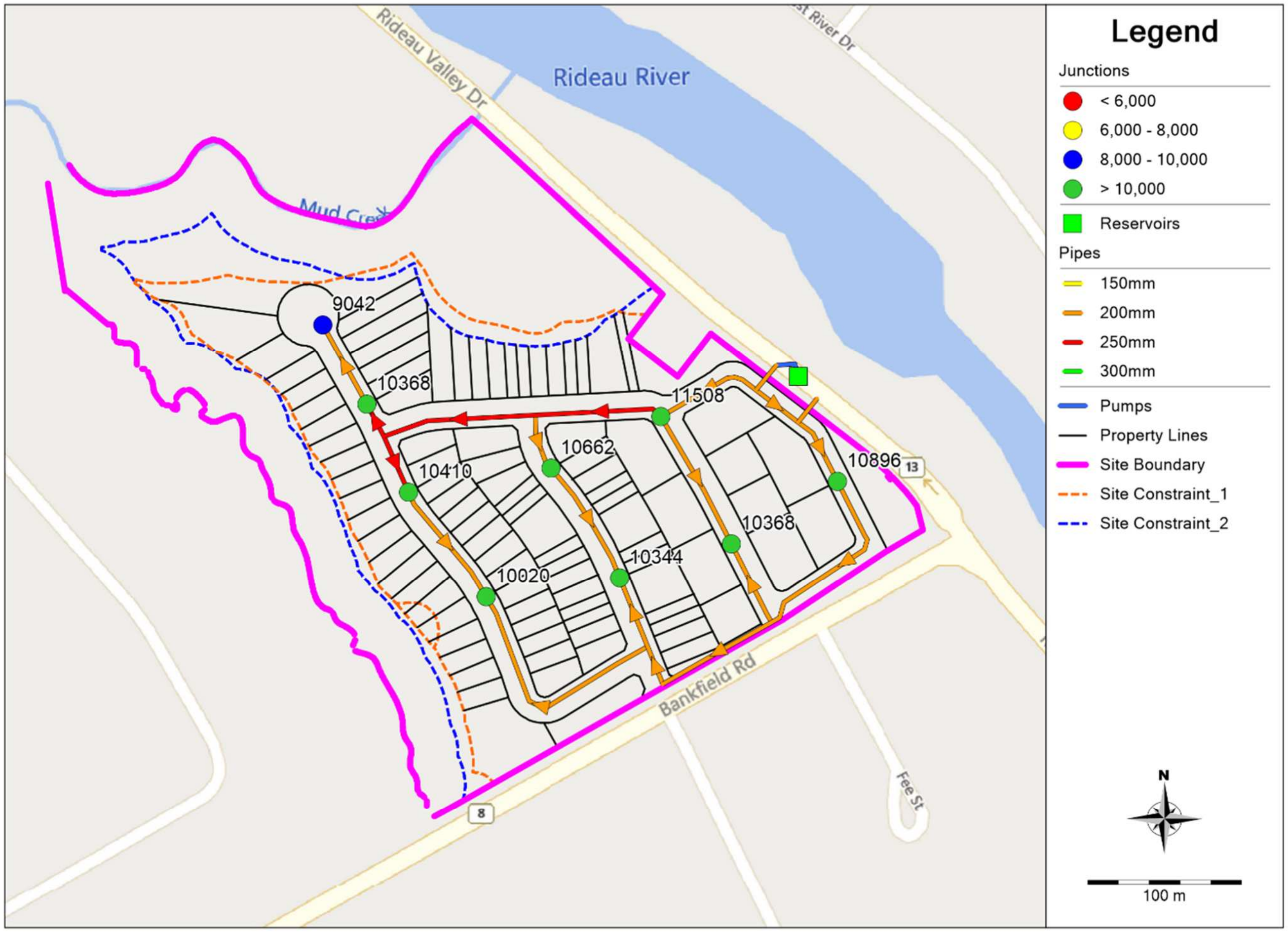
Stinson Lands (121153)
PCSWMM Model Schematic



Available Flow at 20psi During MXDY+FF Conditions – Future

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Stinson Lands (121153)
PCSWMM Model Schematic

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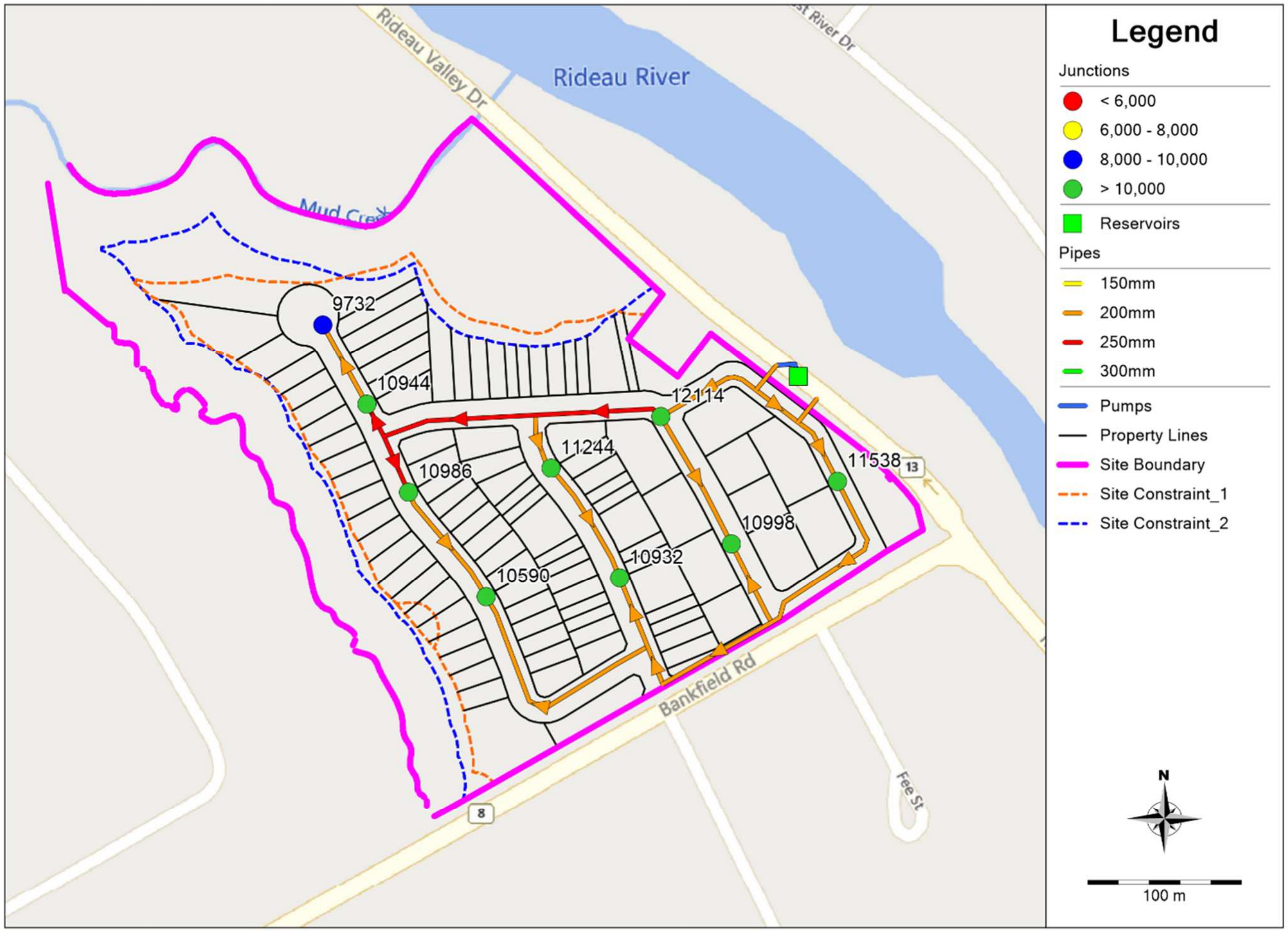

__**Available Flow at 20psi During MXDY+FF Conditions – Existing**__

Date: 2022/07/20
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7.0 UTILITIES, ROADWAYS, AND STREETSCAPE

The development will be serviced by Hydro Ottawa, Bell Canada, Rogers Communications, and Enbridge Gas Distribution Inc. Furthermore, streetlighting will be provided within the proposed road allowances, and will be designed in accordance with the City's lighting policy (2016). The works will be coordinated with local utility companies during detailed design. The cross-section of the utility layout and the connection to the existing services will also be confirmed during detailed design.

A potential 6.0m wide paved emergency pathway will be considered between Rideau Valley Drive and the nearby local street (Street 3). It will be constructed with heavy vehicle road structure, a ditch culvert crossing, and a P-gate or breakdown bollard per City of Ottawa F10 or F11.

Refer to **Appendix G** for the pre-vetted roadway cross-sections that considers roadway width, sidewalk, utilities, and streetscape.

8.0 EROSION AND SEDIMENT CONTROL AND DEWATERING MEASURES

Temporary erosion and sediment control measures will be implemented during construction in accordance with the “Guidelines on Erosion and Sediment Control for Urban Construction Sites” (Government of Ontario, May 1987). Details will be provided on an Erosion and Sediment Control Plan, prepared during detailed design. Erosion and sediment control measures may include:

- Placement of filter fabric under all catch basin and maintenance hatches
- Tree protection fence around the trees to be maintained
- Silt fence around the area under construction placed as per OPSS 577 / OPSD 219.110
- Light duty straw bale check dam per OPSD 219.180

The erosion and sediment control measures will need to be installed to the satisfaction of the engineer, the City, the Ontario Ministry of Environment, Conservation and Parks (MECP), and the Rideau Valley Conservation Authority (RVCA), prior to construction and will remain in place during construction until vegetation is established. The erosion and sediment control measure will also be subject to regular inspection to ensure that measures are operational.

Refer to **Figures 3.3 and 3.4** – Conceptual Grading, Erosion and Sediment Control Plan.

Furthermore, due to the dewatering activities required during construction of the proposed infrastructure a Permit-To-Take-Water (PTTW) application or activity registry will be submitted to the MECP. The permit will outline the water taking quantity and location / quality of the discharge.

9.0 NEXT STEPS, COORDINATION, AND APPROVALS

The proposed municipal infrastructure may be subject, but not limited, to the following next steps, coordination, and approvals:

- MECP PTTW. Submitted to: MECP. Proponent: Developer
- RVCA Approval and Development, Interference with Wetlands and Alterations to Shorelines and Watercourses” (Ont. Reg. 174/06). Submitted to: RVCA. Proponent: Developer
- MECP Environmental Certificate of Approval (ECA) for the storm / sanitary sewers through the “Transfer of Review” program. Submitted to: City of Ottawa/ MECP and approved by MECP. Proponent: Developer
- MECP Pre-authorized watermain alteration and extension program granted as part of City of Ottawa’s Drinking Water Works Permit (F-1 Form). Submitted to: City of Ottawa. Proponent: Developer
- Tree Cutting Permit. Submitted to City of Ottawa. Proponent: Developer, or its contractor/agent
- City of Ottawa Commence Work Notice. Submitted to City of Ottawa. Proponent: Developer, or its contractor/agent
- Road Closure Permit (if required). Submitted to City of Ottawa. Proponent: Developer, or its contractor/agent
- Road Cut Permit. Submitted to City of Ottawa. Proponent: Developer, or its contractor/agent

10.0 SUMMARY AND CONCLUSIONS

This report demonstrates that the proposed development can be adequately serviced with storm and sanitary sewers and watermain. The report is summarized below:

Stormwater Management:

- The Subject Site will be serviced with approximately 1075 m of on-site storm sewers ranging from 250 mm to 1200 mm in diameter. The on-site storm sewers will outlet to the Oxbow adjacent to Mud Creek.
- Inlet control devices will be required to control peak flows and HGL elevations.
- Road Right-of-Ways will be used for surface storage (i.e. saw-toothed grading).
- The major system outlet is the pathway block towards the watercourse (the Oxbow) along the northern portion of the Subject Site.

Sanitary and Wastewater Collection System:

- The proposed off-site works will require a connection made into existing maintenance hole MHSA58925 of the trunk sanitary sewer within the Rideau Valley Drive ROW 15m northeast of the Subject Site.
- The proposed upgrade of the Manotick Pumping Station to allow for 170 L/s of peaked flow will be sufficient to service all current areas of Manotick currently serviced by the municipal wastewater collection system in addition to the 7.45 L/s added by the Subject Site.
- The proposed on-site works will require approximately 1000 m of on-site sanitary sewer to collect and direct wastewater flows to the outlet pipe located in the north-east corner of the Subject Site.

Water Supply System

- There will be two connections made to the 400mm watermain: Connection 1 will be near the sanitary pipe that will be connecting to an existing manhole, and Connection 2 will be approximately 35m further south on the same section of street.
- The proposed on-site works will require approximately 1130m of on-site watermain. The location of hydrants will be confirmed during detailed design.

Erosion and Sediment Control and Dewatering Measures

- Temporary erosion and sediment control measures will be implemented both prior to commencement and during construction in accordance with the "Guidelines on Erosion and Sediment Control for Urban Construction Sites" (Government of Ontario, May 1987).

Next Steps, Coordination, and Approvals

- MECP PTTW
- RVCA Approval and alteration to watercourses permit
- MECP Environmental Certificate of Approval (ECA) for the storm / sanitary sewers through the "Transfer of Review" program
- MECP Pre-authorized watermain alteration and extension program granted as part of City of Ottawa's Drinking Water Works Permit (F-1 Form)
- Tree Cutting Permit
- City of Ottawa Commence Work Notice
- Road Closure Permit
- Road Cut Permit

11.0 CLOSURE

This report is respectfully submitted for review and subsequent approval. Please contact the undersigned should you have questions or require additional information.

NOVATECH

Prepared by:

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Kallie Auld,
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Reviewed by:

LICENSED PROFESSIONAL ENGINEER
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January 23, 2023
PROVINCE OF ONTARIO

Bassam Bahia, M.Eng., P.Eng.
Senior Project Manager | Land Development

Appendix A
Correspondence

MEETING NOTES

Project:	Stinson Manotick	**Project No.:**	121153
Location:	4386 Rideau Valley Road	**Meeting No.:**	NA
Purpose:	Discuss Stormwater Management Strategy	**Date:**	June 22, 2022, 3:00pm to 4:30pm
Next Meeting:	June 29, 2022 for Geomorphology Follow Up		

Attendance:

Name	Representing
Jeff Ostafichuk (JO)	City of Ottawa, File Lead
Brian Morgan (BM)	City of Ottawa, Infrastructure Lead
Damien Whittaker (DW)	City of Ottawa, Senior Engineer
Matthew Hayley (MH)	City of Ottawa, Environmental Planner
Adam Brown (AB)	City of Ottawa, Rural Manager
Eldon Hutchings (EH)	City of Ottawa, Drainage Superintendent
Jasdeep Brar (JB)	City of Ottawa, Student Planner
Andy Robinson (AR)	Robinson Consultants (RCI), Municipal Drains
Eric Lalande (EL) *joined at end of meeting	Rideau Valley Conservation Authority, Planner
Sam Bahia (SB)	Novatech, Senior Project Manager - Engineering
Ben Sweet (BS)	Novatech, Project Coordinator - Engineering
Greg Winters (GW)	Novatech, Director - Planning
Ellen Potts (EP)	Novatech, Planner

Distribution: To Jeff Ostafichuk and Jasdeep Brar for consolidation of notes; to Ryan MacDougall for Uniform's file

Post meeting notes are indicated with blue italic text
Action Items are indicated with bold italic text

Description of Discussion	Action
SB provided a summary of the proposed development and stormwater management strategy: • SWM Outlet: ○ Proposed outlet for majority of post-development drainage is to the oxbow ditch which outlets to Mud Creek directly upstream of the confluence with the Rideau River ○ The proposed design intends to mimic existing conditions and reduce erosion to Wilson Cowan (WC) Drain and Mud Creek ○ Quality Control is proposed via a water quality treatment unit (Stormceptor / Verotechs) to achieve 80% TSS removal (enhanced protection), prior to discharge into the Oxbow. ○ No quantity control given the proximity to the Rideau River and time to peak • Bankfield Culvert Extension ○ The proposed 2m pathway along the northern right-of-way of Bankfield requires an extension of the existing culvert by approximately 2-3m or 1m beyond the Bankfield right-of-way • Access to Drains ○ The Draft Plan proposes an Open Space Block for the Wilson Cowan Drain defined by the proposed development limit, which is based on the most restrictive constraint line. This Open Space block would be transferred to the City.	

Description of Discussion	Action
○ GW clarified that the constraint limit is based on a combination of the most restrictive line between Blanding's Turtle habitat setbacks, the geotechnical & erosion access limit, the 15m from top of slope setback and the 30m from water's edge setback ○ Uniform would continue to maintain ownership of the portion of Mud Creek abutting the development lands ○ GW suggested that an easement could be created for access to the drain SB requested questions/comments on the proposed SWM Strategy from the other meeting attendees: <u>Municipal Drains</u> • EH commented on the watershed boundary and hydraulic design: ○ There may be an opportunity to incorporate the change to the watershed boundary for Wilson Cowan Drain through an existing report that is being completed for another development. The Mud Creek Municipal Drain is very old and doesn't feel that there is a current need to update its watershed boundary. ○ No major changes to the existing channel design are proposed for either drain; if there are no physical changes needed, EH has no further comments on the hydraulic design. • AR commented on the culvert extension noting that it needs to meet the level of service for Wilson Cowan Drain and added that he will need to review as part of his report. If changes to the culvert are needed, they could be incorporated under an existing report being prepared, if timing permits. • EH commented that the proposed Open Space Block would provide adequate space for access to the Wilson Cowan Drain • AR noted that the existing outlet to Wilson Cowan Drain near lot 5/6 of sketch will need to be filled and that the City will require a relatively flat area to access do maintenance works • GW confirmed that there is approximately 15m from the top of the slope to the proposed development limit • AR commented that 15m is relatively narrow for maintenance works • GW pointed out that there is also access to Wilson Cowan Drain from the other side via the abutting Lockmaster Crescent subdivision • AR stated that a change in land use triggers a requirement that they produce a Section 65 report; for Wilson Cowan Drain, they may be able to update it as part of an existing report. • ***SB stated that Novatech will confirm that the City has a flat enough access to safely operate an excavator for maintenance works*** • AR noted that a 5% slope seems reasonable for access • AR commented on the oxbow outlet stating that rip rap protection should be provided wherever it's tied in to avoid erosion along confluence with Mud Creek • SB asked whether a Draft Plan submission in late July/early August would work for the engineer's report and schedule of assessments • EH responded that if the submission is in early enough, it can be updated as part of the existing Section 78 report with Wilson Cowan Drain. • AR added that the sooner the better, but that it's not a critical timeframe; the present schedule for updating existing reports would occur before one year out and that it's dependent on the drainage information that's received from upstream developments. <u>Environment</u> • MH was glad to hear consideration for the Blanding's Turtle habitat; noted that the oxbow is environmental habitat, potentially for more than just Blanding's Turtles, and potential impacts from the outlet on the habitat should be assessed. <u>Fluvial Geomorphology</u> • DW stated that they need to determine if no quantity control at the SWM outlet is acceptable. More precision is needed than the fluvial that exists at the Subwatershed level to determine how dynamic or static a watercourse is and whether this impacts the development setback.	 ***Novatech***

Description of Discussion	Action
• GW noted that stability of the drains are usually addressed as part of the Geotechnical and Slope Stability Report and that it's not typically required for a subdivision that is impacting the drain. • DW stated that they need to know what the development setbacks are and that the fact that drainage is changing does not negate the fact that watercourses may be dynamic. ***DW announced that he had to leave the meeting at this point *** • MH stated that meander belts are more explicitly required in the new Official Plan and that it should be discussed with the RVCA • AR added that that the Minto subdivision has a requirement to do a geomorphological study, which AR will then use in their design. • SB requested clarification for the geomorphology submission requirements. • ***JO suggested that a separate meeting be scheduled to discuss the geomorphology requirements*** • *JO scheduled a meeting on June 29th to continue the Fluvial Geomorphological submission requirements*	***JO***
SB asked if there are any other items to discuss: ROW Widths • EP followed up on a previous discussion with JO regarding the ROW widths for local roads • JO said that he had discussed internally and acknowledged that there are existing local ROWs of less than 20m • GW provided examples of leniency with this Official Plan policy and EP added that the density requirement for the Subject Site is not feasible with 20m ROWs. • ***BM requested that Novatech provide a rationale for reduced local ROW widths for review by BM and DW***.	***Novatech***

Meeting concluded, but Eric Lalande (EL) stayed on with Novatech to get caught up on the above-noted discussion:
 	
• SB provided a brief overview of proposed drainage a development limits
• EL provided the following comments:
  ○ the RVCA typically defers quantity control requirements to the City
  ○ need to look at erosion impacts if not providing quantity control and demonstrate that erosion and sediment control are addressed, but EL reiterated that the RVCA will defer to the City on the quantity control requirements
  ○ The floodplain mapping was updated for Mud Creek and Wilson Cowan Drain at the end of 2019; it's largely the same for Mud Creek, but the floodplain for Wilson Cowan Drain now extends to Bankfield. The updates do not look like they will affect the proposed development.
• ***EL to send all* Mud Creek studies and information on file to Novatech *and provide comments on the SWM Drainage Strategy***
	EL

End of Notes
Please Report any Errors and/or Omissions to the Undersigned.

Prepared by:
NOVATECH

Ellen Potts
Planner

Meeting Attachments:
- Novatech Memorandum, SWM Strategy Outline, dated June 8, 2022

NOVATECH
Engineers, Planners & Landscape Architects

M E M O R A N D U M

DATE: JUNE 8, 2022

TO: BRIAN MORGAN, ELDON HUTCHINGS (CITY OF OTTAWA)

ERIC LALANDE (RVCA)

FROM: MICHAEL PETEPIECE & VAHID MEHDIPOUR

RE: 4386 RIDEAU VALLEY DRIVE N - STINSONS LANDS
SWM STRATEGY OUTLINE
121153

CC: SAM BAHIA, BEN SWEET, BRENDAN RUNDLE

This memo provides an overview of the proposed stormwater management strategy for the Stinson Lands Project, including model development, selection of design storms, and the proposed changes to the drainage areas and flows to the various outlets for the subject property under post-development conditions.

Drainage Areas

Under existing conditions, storm runoff from the proposed development is split between the Wilson-Cowan Drain, Mud Creek, an Oxbow Ditch that outlets to Mud Creek immediately upstream of the confluence with the Rideau River, and the roadside ditch on Rideau Valley Drive – refer to **Figure 1**.

Under proposed conditions, storm runoff from the majority of the development will be directed to the Oxbow Ditch. The flows and contributing drainage areas to the other outlets will be less than pre-development conditions – refer to **Figure 2**.

Model Development

The following provides a brief overview of the data sources used in the hydraulic analysis:

- Existing and proposed subcatchments boundaries were developed using Civil 3D and imported to PCSWMM.
- Paterson group has completed a geotechnical study for the site which was used to characterize the surficial soils and select the appropriate SCS Curve Numbers used in hydrologic model.
- The percent impervious values used in the post-development model were calculated using the Runoff Coefficients shown on the Storm Drainage Area Plan.
- Subcatchment parameters (times to peak, flow path widths, initial abstraction, etc.) were calculated as per City of Ottawa Sewer Design Guidelines.

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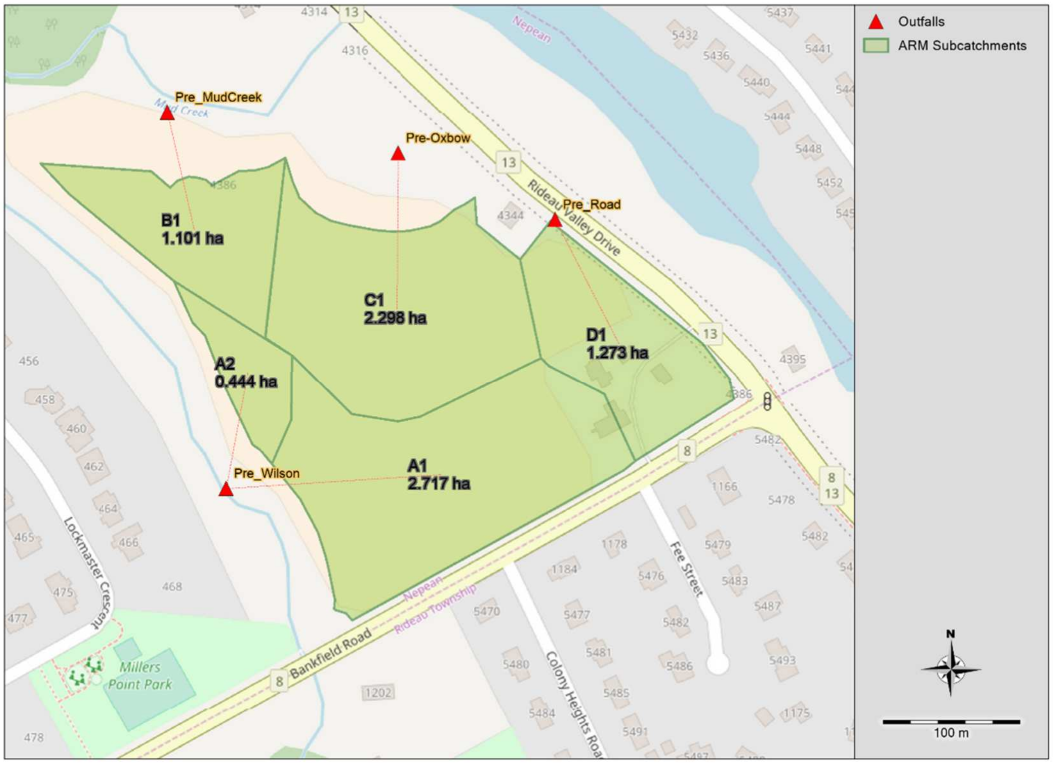

Figure 1: PCSWMM Model Schematic – Existing conditions

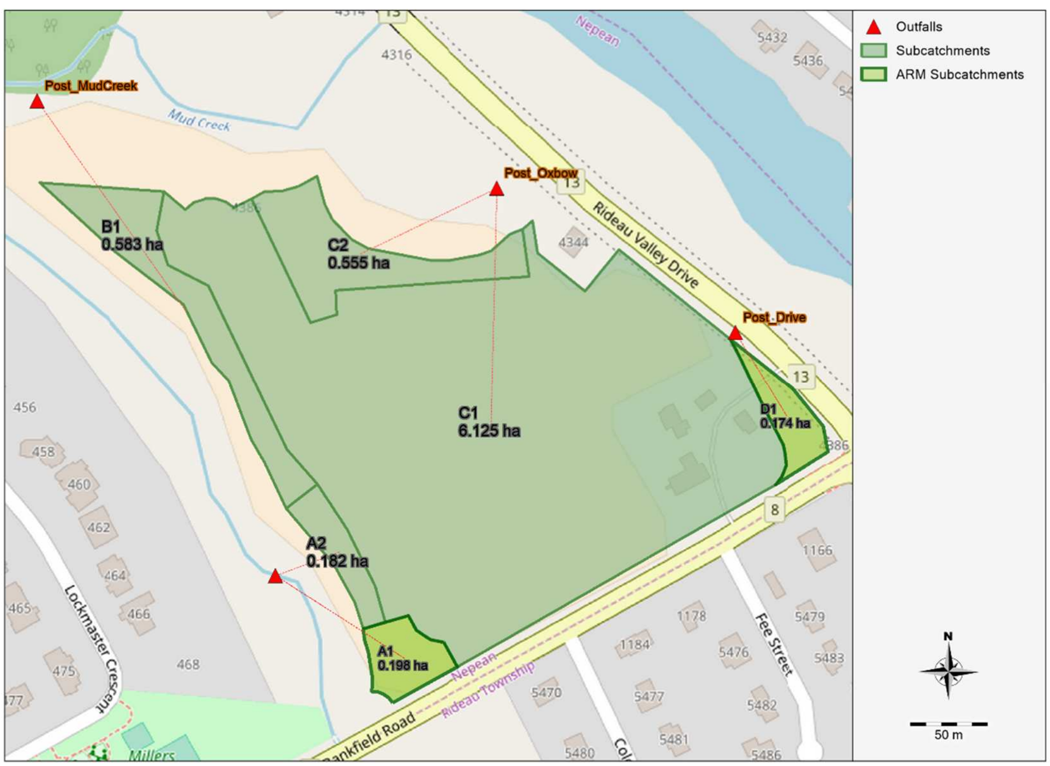

Figure 2: PCSWMM Model Schematic - Proposed Conditions

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Design Storm Selection

The 12hr and 24hr SCS and AES storm distributions have lower peak intensities and generate lower peak flows for impervious areas compared to the Chicago distribution. The 3hr, 4hr and 6hr Chicago storm distributions are most commonly used in the City of Ottawa. The 6hr Chicago is found to produce the highest peak runoff for post-development conditions and was used to calculate the peak flows presented below.

Quantity Control (Pre vs. Post-Development Peak Flows)

Under post-development conditions, the drainage areas and peak flows to Mud Creek, the Wilson Cowan Drain, and the Roadside ditch on Rideau Valley Drive will be significantly less than existing conditions. Storm runoff from the perimeter of the site will continue to flow to these outlets, but the majority of drainage will be routed to a proposed outlet to the Oxbow Ditch.

The Oxbow Ditch outlets to Mud Creek immediately upstream of the confluence with the Rideau River on the upstream side of the bridge under Rideau Valley Drive. Due to the proximity of the site to the Rideau River, no quantity control storage is proposed. The peak flows from the site will reach the Rideau River in advance of the peak flow from Mud Creek, so there should be no adverse impact to Mud Creek or the Wilson Cowan Drain resulting from the proposed development.

Table 1 illustrates storm runoff for existing and proposed conditions for storms with the 2, 5 and 100 years return period.

Table 1: Pre vs. Post-Development Peak Flows (2, 5 and 100 yr Events)

Return Period/Condition		Peak Flow (L/s) – 6hr Chicago Distribution				
		Mud Creek	Wilson Cowan Drain	Oxbow Ditch	Rideau Valley Dr. Roadside Ditch	Total
2 yr	**Existing**	60	133	125	65	**367**
	Proposed	36	12	697	4	**737**
5 yr	**Existing**	109	238	227	117	**658**
	Proposed	58	27	1166	9	**1262**
100 yr	**Existing**	262	570	547	286	**1611**
	Proposed	167	78	2405	27	**2677**

Water Quality Control

The water quality objective is to provide an *Enhanced* level of water quality control corresponding to 80% long-term removal of total suspended solids. Water quality treatment will be provided using a hydrodynamic separator (Stormceptor, Vortechnics, etc.) at the proposed storm outlet to the Oxbow Ditch. The Oxbow Ditch will provide additional inherent treatment through filtration and settling before discharging to Mud Creek/Rideau River. Lot level and conveyance best management practices will be implemented in the design of the subdivision.

Under post-development conditions, storm runoff to the other outlets will consist of rearyard and park areas. The runoff from these areas is typically considered 'clean' and no engineered water quality treatment measures should be required beyond best management practices.

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Rideau River & Mud Creek Floodplain

The proposed development will be fully outside the limits of the Rideau River and Mud Creek 100yr floodplains. Floodplain limits of Rideau River and Mud Creek are shown in the appended **Macro Servicing Plan.** The floodplain limits and associated setbacks have been taken into consideration in the concept plan for the subdivision.

The 100yr water levels will be used as downstream boundary conditions in the hydraulic analysis that will be completed as part of the Draft Plan application and detailed designs.

Impacts on Municipal Drains

The proposed development will have no adverse impacts on the Wilson Cowan and Mud Creek Municipal Drains. The drainage areas and peak flows to these watercourses will be less than existing conditions, so there should be no requirement revise the Engineer's Reports for these Municipal Drains at this time. Access to the Municipal Drains will be provided via easements as shown on the attached Plan.

Robinson Consultants Inc. (RCI) have already appointed as the Drainage Engineer to the Wilson-Cowan Drain. Additional communication and correspondence will be undertaken with Drainage Superintendent – Municipal Drainage and RCI to determine the impact and legislative requirements for both the Wilson-Cowan Drain and Mud Creek as a result of this development and land use change.

Notwithstanding the above, the **Macro Servicing Plan** indicates the proposed lot development limit, and top of slope for the existing drains, which demonstrates that access for future maintenance will be protected. Additional measures may be required in the form of easements or notice on title to ensure that that maintenance access will remain unencumbered.

Alterations to Watercourses

The proposed development will require some modifications to existing infrastructure and the construction of new outlets to the receiving watercourses:

- An extension of the Bankfield Road culvert will be required to facilitate a pathway along the north side of Bankfield Road.
- New outlets to the Wilson-Cowan MD will be required for the proposed park, and the rear yards of lots 1-22.
- New outlets to the Mud Creek MD will be required for the rear yards of 23-29 and 56-64.
- A new storm outlet to the Oxbow Ditch will be required. This storm outlet will be the primary outlet for the proposed development.

The proposed outlets and culvert extension will require an Application to RVCA for "Development, Interference with Wetlands and Alterations to Shorelines and Watercourses" (Ont. Reg. 174/06).

Summary

Runoff to the Mud Creek and Wilson-Cowan MDs will be less than existing conditions. The only increase in flow will be to the Oxbow Ditch, which is immediately upstream of the confluence with the Rideau River. No stormwater quantity controls are proposed.

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An Enhanced level of water quality treatment will be provided using a combination of lot level and conveyance BMPs, in conjunction with a hydrodynamic separator at the outlet to the Oxbow Ditch. No engineered water quality treatment measures will be required for rear yards and park areas draining directly to the Municipal Drains.

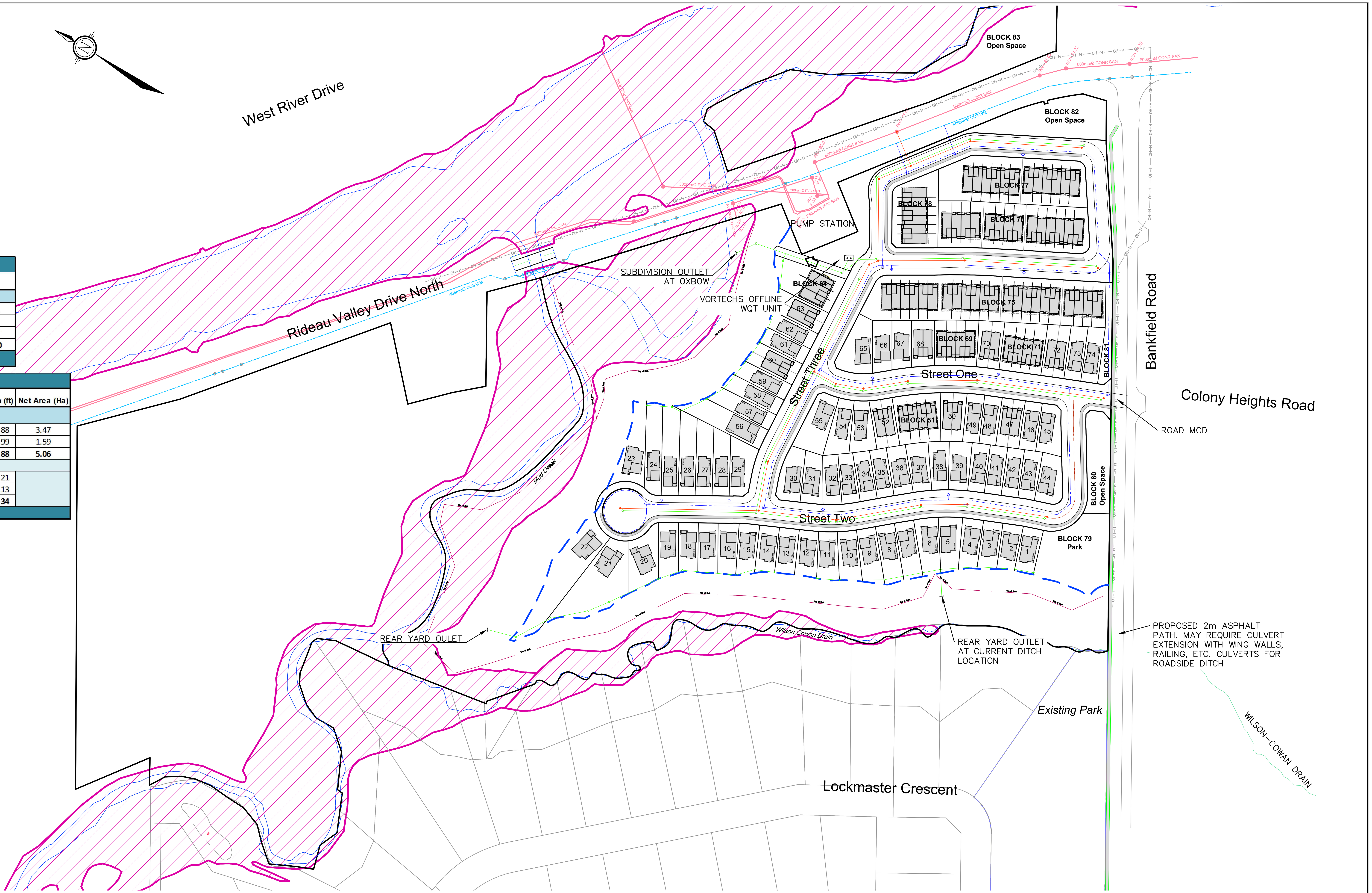
The proposed development will have no adverse impact on the Municipal Drains, and updates to the Engineer's Reports should not be required as part of the development application, although RCI and the Drainage Superintendent will review this from the Drainage Act perspective.

ATTACHMENT
Macro Servicing Plan

Suite 200, 240 Michael Cowpland Drive, Ottawa ON K2M 1P6 Tel: 613.254.9643 Fax: 613.254.5867 www.novatech-eng.com

LEGEND

- Site Boundary
- Drainage Line (River/Stream edge/centerline)
- 1:100 Floodplain Limit (RVCA GeoPortal)
- Proposed Lot Development Limit
- Top of Bank (AOV)
- Proposed 2.0m Pathway
- Proposed Storm Sewers
- Proposed Sanitary Sewers
- Proposed Watermain



4386 Rideau Valley Drive		
* Setback Assumptions: 6.0m FY, 6.0m RY, 3.0m Ext Side, 1.2m Int	Yield	%
Singles	62	42
Semis	16	11
Towns	69	47
Total	**147**	**100**

4386 Rideau Valley Drive			
* Setback Assumptions: 6.0m FY, 6.0m RY, 3.0m Ext Side, 1.2m Int	length (m)	length (ft)	Net Area (Ha)
Saleable Frontage			
Singles/Semis	879.62	2885.88	3.47
Towns	462.99	1518.99	1.59
Total Frontage	**1342.61**	**4404.88**	**5.06**
Road Lengths			
18m ROW	825.16	2707.21	
16.5m ROW	368.24	1208.13	
Total roads	**1193.40**	**3915.34**	

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

No.	REVISION	DATE	BY
1.	ISSUED FOR REVIEW	MAY 24/22	BHB

SCALE: 1:1250 (A1) / 1:2500 (11x17)

DESIGN: XXX
CHECKED: XXX
DRAWN: wls
CHECKED: XXX
APPROVED: XXX

FOR REVIEW ONLY

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CITY of OTTAWA
121153-STINTSON LANDS

DRAWING NAME
MACRO SERVICING PLAN

PROJECT No. 121153
REV: REV #1
DRAWING No. 121153-GP

PLAN24x36.DWG - 914.4mmx609.6mm

NOVATECH
Engineers, Planners & Landscape Architects

MEETING NOTES

Project:	Stinson Manotick	**Project No.:**	121153
Location:	4386 Rideau Valley Road	**Meeting No.:**	NA
Purpose:	Discuss Fluvial Geomorphology Requirements	**Date:**	June 29, 2022, 9:00am to 10:00am
Next Meeting:	N/A		

Attendance:

Name	Representing
Jeff Ostafichuk (JO)	City of Ottawa, File Lead
Brian Morgan (BM)	City of Ottawa, Infrastructure Lead
Damien Whittaker (DW)	City of Ottawa, Senior Engineer
Eric Lalande (EL)	Rideau Valley Conservation Authority, Planner
Sam Bahia (SB)	Novatech, Senior Project Manager - Engineering
Greg Winters (GW)	Novatech, Director - Planning
Ellen Potts (EP)	Novatech, Planner

Distribution: To Jeff Ostafichuk for consolidation of notes; to Ryan MacDougall for Uniform's file

Post meeting notes are indicated with blue italic text
Action Items are indicated with bold italic text

Description of Discussion	Action
This meeting was scheduled as a continuation of the geomorphology discussion from the Stormwater Management Strategy meeting that was held on June 22, 2022. The two key items for discussion at this meeting were (1) quantity control and (2) the requirement for a fluvial geomorphology study. <u>Quantity Control</u> • SB reiterated that the outlet for most of the post development drainage is into the oxbow, which outlets immediately upstream of the confluence of Mud Creek with the Rideau River; the water travels under the Rideau Valley Drive bridge and into the Rideau River. As such, he doesn't see issues with downstream impacts. The main concern expressed by Municipal Drains during the June 22, 2022 SWM meeting was erosion potential at the confluence with Mud Creek, but that rip rap could be provided for erosion protection. • DW explained that the City's main concerns with not providing quantity control is (1) the erosion capacity of the outlet and (2) the culvert capacity for conveyance. • SB clarified that there is no downstream culvert, Mud Creek flows freely under the Rideau Valley bridge. • DW responded that capacity under the bridge is likely not an issue. • SB suggested that we could assess the difference between pre-development discharge vs. post-development discharge/velocity to determine if quantity control is warranted and if erosion potential will be an issue. • DW responded that the water needs to get out of the subdivision without having negative impacts.	

Description of Discussion	Action
<u>Quality Control</u> • There may not be explicit quantity control requirements, but there may criteria for quality control (e.g. subwatershed study requirements, geotechnical and erosion control requirements, thermal requirements) that invoke a requirement for quantity control to address these various potential criteria. DW added that it's the quality control that makes SWM ponds large, not the quantity control. As such the City is concerned that the area shown on the Plan for a water quality treatment unit is not large enough. • EL confirmed that thermal mitigation is not required. • SB explained that an enhanced level of water quality protection to provide 80% TSS removal is proposed. Novatech will ensure that the area provided for water quality treatment meets size requirements. • DW added that Mathew Hayley may have environmental protection requirements that needs to be considered. • SB confirmed that work is underway to identify and address environmental requirements. <u>Fluvial Geomorphological Study Requirements</u> • SB noted that the City is requiring Minto to complete a fluvial study for Wilson Cowan Drain to the confluence of Mud Creek as part of the upstream Mahogany subdivision development and that work is being undertaken by Andy Robinson (RCI) for that. Since drainage to Wilson Cowan Drain is being reduced by Uniform's proposed development, SB asked if there is a need to study the Wilson Cowan Drain. For Mud Creek, SB noted that Parish had completed a study in 2004 *(Parish Geomorphic Ltd. Mud Creek Watershed Existing Conditions Report, Report No. 2003-034)* and asked if there are any requirements to study it now. • For Wilson Cowan Drain, DW responded that, subject to input from RCI, if flows to it are being reduced and sufficient rip-rap erosion protection is provided at the outlet, there may not be a need to study it further. • For Mud Creek, DW stated that the larger subwatershed study doesn't have the specificity needed for a subdivision; a fluvial geomorphological study is needed to look at erosion potential, meander belts, and whether the drain is static or dynamic to be able to determine a safe development limit for this application. • EL added that when the RVCA was updating the floodplain hazard mapping for the area, they stopped the work short of assessing fluvial geomorphology with the understanding that it would be completed by developers at the time of development application depending on the scale of the project. • GW asked who would review the fluvial geomorphological report. • DW responded that he would review it. • SB stated that Novatech will reach out to Matrix Solutions to undertake the fluvial geomorphological study.	***Novatech***

<u>Other Items</u>
• Impact Assessment of adjacent Municipal Depot (4244 Rideau Valley Drive):
    o JO noted that the City's pre-consult notes erred in requiring an impact assessment for a Holland Road Dump, but that a point was made by City Staff that there may be a requirement to conduct an impact assessment for the Municipal Depot.
    o GW explained that Phase 1 and 2 ESAs were conducted for 4386 Rideau Valley Drive. The Phase 1 ESA assessed the Municipal Depot and identified an APEC on the property. This APEC was assessed and cleared as part of the Phase 2 ESA.
    o DW responded that if Phase 1 and 2 ESAs have been conducted and assessed potential impacts from the adjacent Municipal Depot, the requirement for further impact assessment is cleared.
 	
• Rural Local ROW widths:
    o EP raised that BM had requested Novatech provide a rationale for reducing the standard 20m rural local ROW width to 18m and 14.75m (for window streets) during the June 22, 2022 meeting. EP referred to the City's pre-consult notes which state that *"While an 18 metre right-of-way might be acceptable, the City prefers a 20 metres. Acceptance of 18 metres will depend on whether all the underground services and tree requirements can be accommodated. Please provide details on how all these components can be accommodated."*	

Description of Discussion	Action
○ BM responded that it's a matter of demonstrating that the 18m ROWs can accommodate these requirements. ○ GW added that the 14.75m ROW for window streets is equivalent to the 18m ROW and the City is developing a cross-section for the 14.75m ROW. ○ DW added that the City is accepting of 18m ROWs, but not 16.5m ROWs, and that the City's new cross-sections will be released very shortly. The 18m and 14.75m ROWs are okay if Novatech can prove that they work.	

End of Notes
Please Report any Errors and/or Omissions to the Undersigned.

Prepared by:
NOVATECH

Ellen Potts
Planner

Appendix B
Servicing Report Checklist

Development Servicing Study Checklist

Project Name: Wright Lands
Project Number: 116037
Date: July 23, 2020

4.1 General Content	Addressed (Y/N/NA)	Section	Comments
Executive Summary (for larger reports only).	NA		
Date and revision number of the report.	Y	Cover	
Location map and plan showing municipal address, boundary, and layout of proposed development.	Y	Fig 1.1, 1.2, 1.3	
Plan showing the site and location of all existing services.	Y	Fig 1.2, 3.1	
Development statistics, land use, density, adherence to zoning and official plan, and reference to applicable subwatershed and watershed plans that provide context to which individual developments must adhere.	NA		
Summary of Pre-consultation Meetings with City and other approval agencies.	Y	1	
Reference and confirm conformance to higher level studies and reports (Master Servicing Studies, Environmental Assessments, Community Design Plans), or in the case where it is not in conformance, the proponent must provide justification and develop a defendable design criteria.	Y	1, 2	
Statement of objectives and servicing criteria.	Y	1	
Identification of existing and proposed infrastructure available in the immediate area.	Y	3,4,5,6,7	
Identification of Environmentally Significant Areas, watercourses and Municipal Drains potentially impacted by the proposed development (Reference can be made to the Natural Heritage Studies, if available).	Y	4	
Concept level master grading plan to confirm existing and proposed grades in the development. This is required to confirm the feasibility of proposed stormwater management and drainage, soil removal and fill constraints, and potential impacts to neighboring properties. This is also required to confirm that the proposed grading will not impede existing major system flow paths.	Y	Fig 3.2	

4.1 General Content	**Addressed (Y/N/NA)**	**Section**	**Comments**
Identification of potential impacts of proposed piped services on private services (such as wells and septic fields on adjacent lands) and mitigation required to address potential impacts.	NA		
Proposed phasing of the development, if applicable.	NA		
Reference to geotechnical studies and recommendations concerning servicing.	Y	2.2	
All preliminary and formal site plan submissions should have the following information:			
Metric scale	NA		
North arrow (including construction North)	NA		
Key plan	NA		
Name and contact information of applicant and property owner	NA		
Property limits including bearings and dimensions	NA		
Existing and proposed structures and parking areas	NA		
Easements, road widening and rights-of-way	NA		
Adjacent street names	NA		

4.2 Water	**Addressed (Y/N/NA)**	**Section**	**Comments**
Confirm consistency with Master Servicing Study, if available.	Y	6	
Availability of public infrastructure to service proposed development.	Y	6	
Identification of system constraints.	Y	6	
Identify boundary conditions.	Y	6	
Confirmation of adequate domestic supply and pressure.	Y	6	
Confirmation of adequate fire flow protection and confirmation that fire flow is calculated as per the Fire Underwriter's Survey. Output should show available fire flow at locations throughout the development.	Y	6	
Provide a check of high pressures. If pressure is found to be high, an assessment is required to confirm the application of pressure reducing valves.	Y	6	
Definition of phasing constraints. Hydraulic modeling is required to confirm servicing for all defined phases of the project including the ultimate design.	Y	6	
Address reliability requirements such as appropriate location of shut-off valves.	Y	6	
Check on the necessity of a pressure zone boundary modification.	NA		
Reference to water supply analysis to show that major infrastructure is capable of delivering sufficient water for the proposed land use. This includes data that shows that the expected demands under average day, peak hour and fire flow conditions provide water within the required pressure range.	Y	6	
Description of the proposed water distribution network, including locations of proposed connections to the existing system, provisions for necessary looping, and appurtenances (valves, pressure reducing valves, valve chambers, and fire hydrants) including special metering provisions.	Y	6, Fig 3.1	
Description of off-site required feedermains, booster pumping stations, and other water infrastructure that will be ultimately required to service proposed development, including financing, interim facilities, and timing of implementation.	Y	6	
Confirmation that water demands are calculated based on the City of Ottawa Design Guidelines.	Y	6	
Provision of a model schematic showing the boundary conditions locations, streets, parcels, and building locations for reference.	Y	Fig 6.1	

4.3 Wastewater	**Addressed (Y/N/NA)**	**Section**	**Comments**
Summary of proposed design criteria (Note: Wet-weather flow criteria should not deviate from the City of Ottawa Sewer Design Guidelines. Monitored flow data from relatively new infrastructure cannot be used to justify capacity requirements for proposed infrastructure).	Y	5	
Confirm consistency with Master Servicing Study and/or justifications for deviations.	Y	5	
Consideration of local conditions that may contribute to extraneous flows that are higher than the recommended flows in the guidelines. This includes groundwater and soil conditions, and age and condition of sewers.	NA		
Description of existing sanitary sewer available for discharge of wastewater from proposed development.	Y	5	
Verify available capacity in downstream sanitary sewer and/or identification of upgrades necessary to service the proposed development. (Reference can be made to previously completed Master Servicing Study if applicable)	Y	5	
Calculations related to dry-weather and wet-weather flow rates from the development in standard MOE sanitary sewer design table (Appendix 'C') format.	N		
Description of proposed sewer network including sewers, pumping stations, and forcemains.	Y	5	
Discussion of previously identified environmental constraints and impact on servicing (environmental constraints are related to limitations imposed on the development in order to preserve the physical condition of watercourses, vegetation, soil cover, as well as protecting against water quantity and quality).	NA		
Pumping stations: impacts of proposed development on existing pumping stations or requirements for new pumping station to service development.	NA		
Forcemain capacity in terms of operational redundancy, surge pressure and maximum flow velocity.	NA		
Identification and implementation of the emergency overflow from sanitary pumping stations in relation to the hydraulic grade line to protect against basement flooding.	NA		
Special considerations such as contamination, corrosive environment etc.	NA		

4.4 Stormwater	**Addressed (Y/N/NA)**	**Section**	**Comments**
Description of drainage outlets and downstream constraints including legality of outlet (i.e. municipal drain, right-of-way, watercourse, or private property).	Y	4	
Analysis of the available capacity in existing public infrastructure.	Y	4	
A drawing showing the subject lands, its surroundings, the receiving watercourse, existing drainage patterns and proposed drainage patterns.	Y	Fig 4.1	
Water quantity control objective (e.g. controlling post-development peak flows to pre-development level for storm events ranging from the 2 or 5 year event (dependent on the receiving sewer design) to 100 year return period); if other objectives are being applied, a rationale must be included with reference to hydrologic analyses of the potentially affected subwatersheds, taking into account long-term cumulative effects.	Y	4	
Water Quality control objective (basic, normal or enhanced level of protection based on the sensitivities of the receiving watercourse) and storage requirements.	Y	4	
Description of stormwater management concept with facility locations and descriptions with references and supporting information.	Y	4	
Set-back from private sewage disposal systems.	NA		
Watercourse and hazard lands setbacks.	Y	Fig 1.3	
Record of pre-consultation with the Ontario Ministry of Environment and the Conservation Authority that has jurisdiction on the affected watershed.	NA		
Confirm consistency with sub-watershed and Master Servicing Study, if applicable study exists.	Y	4	
Storage requirements (complete with calcs) and conveyance capacity for 5 yr and 100 yr events.	Y	4	
Identification of watercourse within the proposed development and how watercourses will be protected, or, if necessary, altered by the proposed development with applicable approvals.	Y	4	
Calculate pre and post development peak flow rates including a description of existing site conditions and proposed impervious areas and drainage catchments in comparison to existing conditions.	Y	4	
Any proposed diversion of drainage catchment areas from one outlet to another.	Y	4	
Proposed minor and major systems including locations and sizes of stormwater trunk sewers, and SWM facilities.	Y	4	
If quantity control is not proposed, demonstration that downstream system has adequate capacity for the post-development flows up to and including the 100-year return period storm event.	Y	4	

4.4 Stormwater	**Addressed (Y/N/NA)**	**Section**	**Comments**
Identification of municipal drains and related approval requirements.	Y	4	
Description of how the conveyance and storage capacity will be achieved for the development.	Y	4	
100 year flood levels and major flow routing to protect proposed development from flooding for establishing minimum building elevations (MBE) and overall grading.	Y	4	
Inclusion of hydraulic analysis including HGL elevations.	Y	4	
Description of approach to erosion and sediment control during construction for the protection of receiving watercourse or drainage corridors.	Y	8	
Identification of floodplains – proponent to obtain relevant floodplain information from the appropriate Conservation Authority. The proponent may be required to delineate floodplain elevations to the satisfaction of the Conservation Authority if such information is not available or if information does not match current conditions.	Y	4	
Identification of fill constrains related to floodplain and geotechnical investigation.	Y	2, Fig 1.3	

4.5 Approval and Permit Requirements	**Addressed (Y/N/NA)**	**Section**	**Comments**
Conservation Authority as the designated approval agency for modification of floodplain, potential impact on fish habitat, proposed works in or adjacent to a watercourse, cut/fill permits and Approval under Lakes and Rivers Improvement Act. The Conservation Authority is not the approval authority for the Lakes and Rivers Improvement Act. Where there are Conservation Authority regulations in place, approval under the Lakes and Rivers Improvement Act is not required, except in cases of dams as defined in the Act.	Y	9	
Application for Certificate of Approval (CofA) under the Ontario Water Resources Act.	Y	9	
Changes to Municipal Drains.	NA		
Other permits (National Capital Commission, Parks Canada, Public Works and Government Services Canada, Ministry of Transportation etc.)	Y	9	

4.6 Conclusion	**Addressed (Y/N/NA)**	**Section**	**Comments**
Clearly stated conclusions and recommendations.	Y	10	
Comments received from review agencies including the City of Ottawa and information on how the comments were addressed. Final sign-off from the responsible reviewing agency.	NA		
All draft and final reports shall be signed and stamped by a professional Engineer registered in Ontario.	Y	11	

Appendix C
Storm Sewer Design Sheets and Stormwater Management Calculations

STORM SEWER DESIGN SHEET

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #:	121153
Project Name:	Stinson Lands
Date Prepared:	9/6/2022
Date Revised:	
Input By:	Brendan Rundle
Reviewed By:	Ben Sweet/Sam Bahia
Drawing Reference:	121153-GPO AND 121153-STM

Legend:
- PROJECT SPECIFIC INFO
- USER DESIGN INPUT
- CUMILATIVE CELL
- CALCULATED DESIGN CELL OUTPUT
- USER AS-BUILT INPUT

DEMAND

LOCATION			AREA								FLOW						
STREET	FROM MH	TO MH	AREA ID	HIGH DENSITY	ROAD 1	REAR YARD 1	REAR YARD 2	PARK	TOTAL AREA	WEIGHTED RUNOFF COEFFICIENT	INDIVI 2.78 AR	ACCUM 2.78 AR	TIME OF CONC	RAIN INTENSITY (mm/hr) 2yr	5yr	100yr	PEAK FLOW
				0.85	0.70	0.50	0.45	0.20	(ha)				(min.)				(L/s)
Street 1	2	4	C13		0.29				0.29	0.70	0.56	0.56	10.00	76.81			43.34
									0.00		0.00	0.00	10.00				0.00
									0.00		0.00	0.00	10.00				0.00
Street 1	4	6	-						0.00		0.00	0.56	10.99	73.21			41.31
									0.00		0.00	0.00	10.99				0.00
									0.00		0.00	0.00	10.99				0.00
Street 1	6	8	C12		0.33				0.33	0.70	0.64	1.21	11.53	71.38			86.12
									0.00		0.00	0.00	11.53				0.00
									0.00		0.00	0.00	11.53				0.00
Street 1	8	10	-						0.00		0.00	1.21	11.83	70.43			84.98
									0.00		0.00	0.00	11.83				0.00
									0.00		0.00	0.00	11.83				0.00
Street 1	10	12	C10		0.22				0.22	0.70	0.43	1.63	11.99	69.93			114.30
									0.00		0.00	0.00	11.99				0.00
									0.00		0.00	0.00	11.99				0.00
Street 2	14	16	C01		0.28				0.28	0.70	0.54	0.54	10.00	76.81			41.85
									0.00		0.00	0.00	10.00				0.00
									0.00		0.00	0.00	10.00				0.00
Street 2	16	18	C02		0.33				0.33	0.70	0.64	1.19	10.88	73.57			87.33
									0.00		0.00	0.00	10.88				0.00
									0.00		0.00	0.00	10.88				0.00
Street 2	18	20	C03		0.28				0.28	0.70	0.54	1.73	11.33	72.04			124.78
									0.00		0.00	0.00	11.33				0.00
									0.00		0.00	0.00	11.33				0.00
Street 2	20	22	C04		0.31				0.31	0.70	0.60	2.34	11.94	70.08			163.66
									0.00		0.00	0.00	11.94				0.00
									0.00		0.00	0.00	11.94				0.00
Street 2	24	48	C06		0.34				0.34	0.70	0.66	0.66	10.00	76.81			50.82
									0.00		0.00	0.00	10.00				0.00
									0.00		0.00	0.00	10.00				0.00
Street 2	48	22	C05		0.24				0.24	0.70	0.47	1.13	11.17	72.57			81.91
									0.00		0.00	0.00	11.17				0.00
									0.00		0.00	0.00	11.17				0.00
Street 3	22	26	-						0.00		0.00	3.46	12.52	68.30			236.59
									0.00		0.00	0.00	12.52				0.00
									0.00		0.00	0.00	12.52				0.00
Street 3	26	12	C11			0.65			0.65	0.50	0.90	4.37	12.99	66.97			292.49
									0.00		0.00	0.00	12.99				0.00
									0.00		0.00	0.00	12.99				0.00
Street 3	12	28	C08,C14		0.27	0.56			0.83	0.57	1.30	7.31	13.95	64.37			470.31
									0.00		0.00	0.00	13.95				0.00
									0.00		0.00	0.00	13.95				0.00
Street 3	28	30	C09		0.23				0.23	0.70	0.45	7.75	15.22	61.26			474.94
									0.00		0.00	0.00	15.22				0.00
									0.00		0.00	0.00	15.22				0.00
Street 3	42	40	C20		0.21				0.21	0.70	0.41	0.41	10.00	76.81			31.39
									0.00		0.00	0.00	10.00				0.00
									0.00		0.00	0.00	10.00				0.00
Street 3	40	38	C18,C19		0.17	0.40			0.57	0.56	0.89	1.30	10.49	74.99			97.15
									0.00		0.00	0.00	10.49				0.00
									0.00		0.00	0.00	10.49				0.00
Street 3	38	32	C17		0.32				0.32	0.70	0.62	1.92	11.00	73.18			140.38
									0.00		0.00	0.00	11.00				0.00
									0.00		0.00	0.00	11.00				0.00
Street 3	36	34	C16		0.35				0.35	0.70	0.68	0.68	10.00	76.81			52.31
									0.00		0.00	0.00	10.00				0.00
									0.00		0.00	0.00	10.00				0.00
Street 3	34	32	C15		0.34				0.34	0.70	0.66	1.34	11.67	70.95			95.27
									0.00		0.00	0.00	11.67				0.00
									0.00		0.00	0.00	11.67				0.00
Street 3	32	30	-						0.00		0.00	3.26	13.30	66.09			215.53
									0.00		0.00	0.00	13.30				0.00
									0.00		0.00	0.00	13.30				0.00
Easement Block	30	44	-						0.00		0.00	11.01	15.39	60.87			670.45
									0.00		0.00	0.00	15.39				0.00
									0.00		0.00	0.00	15.39				0.00
Easement Block	44	46	C07,C21			0.63			0.63	0.50	0.88	11.89	15.52	60.56			720.06
									0.00		0.00	0.00	15.52				0.00
									0.00		0.00	0.00	15.52				0.00

CAPACITY

LOCATION			FLOW		PROPOSED SEWER PIPE SIZING / DESIGN								
					PIPE PROPERTIES								
STREET	FROM MH	TO MH	TOTAL UNCONTROLLED PEAK FLOW (QDesign) (L/s)	TOTAL RESTRICTED PEAK FLOW (Q) (L/s)	LENGTH (m)	SIZE / MATERIAL (mm / type)	ID ACTUAL (m)	ROUGHNESS	DESIGN GRADE (%)	FULL FLOW CAPACITY (L/s)	FULL FLOW VELOCITY (m/s)	TIME OF FLOW (min.)	QPEAK DESIGN / QFULL (%)
Street 1	2	4	43.3		82.8	250 PVC	0.254	0.013	1.30	70.7	1.40	0.99	61.3%
Street 1	4	6	41.3		45.7	250 PVC	0.254	0.013	1.30	70.7	1.40	0.55	58.4%
Street 1	6	8	86.1		29.9	300 PVC	0.305	0.013	1.50	123.6	1.69	0.29	69.7%
Street 1	8	10	85.0		19.0	375 PVC	0.381	0.013	1.50	224.0	1.96	0.16	37.9%
Street 1	10	12	114.3		18.1	450 PVC	0.457	0.013	1.50	364.3	2.22	0.14	31.4%
Street 2	14	16	41.8		51.9	300 PVC	0.305	0.013	0.50	71.3	0.98	0.88	58.7%
Street 2	16	18	87.3		30.4	375 PVC	0.381	0.013	0.50	129.3	1.13	0.45	67.5%
Street 2	18	20	124.8		51.8	525 CONC	0.533	0.013	0.50	317.2	1.42	0.61	39.3%
Street 2	20	22	163.7		54.5	600 CONC	0.610	0.013	0.50	452.9	1.55	0.59	36.1%
Street 2	24	48	50.8		63.8	450 PVC	0.457	0.013	0.25	148.7	0.91	1.17	34.2%
Street 2	48	22	81.9		28.7	450 PVC	0.457	0.013	0.25	148.7	0.91	0.53	55.1%
Street 3	22	26	236.6		30.3	600 CONC	0.610	0.013	0.25	320.3	1.10	0.46	73.9%
Street 3	26	12	292.5		68.4	675 CONC	0.686	0.013	0.25	438.5	1.19	0.96	66.7%
Street 3	12	28	470.3		69.6	750 CONC	0.762	0.013	0.25	580.7	1.27	0.91	81.0%
Street 3	28	30	474.9		12.8	750 CONC	0.762	0.013	0.25	580.7	1.27	0.17	81.8%
Street 3	42	40	31.4		75.6	250 PVC	0.254	0.013	4.50	131.6	2.60	0.49	23.8%
Street 3	40	38	97.1		66.9	300 PVC	0.305	0.013	2.50	159.5	2.19	0.51	60.9%
Street 3	38	32	140.4		54.1	525 CONC	0.533	0.013	0.25	224.3	1.00	0.90	62.6%
Street 3	36	34	52.3		90.7	375 PVC	0.381	0.013	0.32	103.5	0.91	1.67	50.6%
Street 3	34	32	95.3		100.5	450 PVC	0.457	0.013	0.32	168.3	1.02	1.63	56.6%
Street 3	32	30	215.5		11.1	600 CONC	0.610	0.013	0.25	320.3	1.10	0.17	67.3%
Easement Block	30	44	670.4		14.3	1200 CONC	1.219	0.013	0.25	2033.7	1.74	0.14	33.0%
Easement Block	44	46	720.1		39.6	1200 CONC	1.219	0.013	0.25	2033.7	1.74	0.38	35.4%

DEMAND EQUATION
Q = 2.78 AIR
Where : Q = Peak flow in litres per second (L/s)
A = Area in hectares (ha)
R = Weighted runoff coefficient (increased by 25% for 100-year)
I = Rainfall intensity in millimeters per hour (mm/hr)
Rainfall Intensity (I) is based on City of Ottawa IDF data presented in the City of Ottawa Sewer Design Guidelines (Oct. 2012)

CAPACITY EQUATION
Q full= (1/n) A R^(2/3)So^(1/2)
Where : Q full = Capacity (L/s)
n = Manning coefficient of roughness (0.013)
A = Flow area (m^2)
R = Wetter perimenter (m)
So = Pipe Slope/gradient

NOTE(S)
Highlighted sewer sections represent future design considerations that are not applicable to this MECP ECA application.

Stinson Lands
Pre-Development Model Parameters

Time to Peak Calculations
(Uplands Overland Flow Method)
Existing Conditions

		Overland Flow						Concentrated Overland Flow						Overall					
Area ID	Area (ha)	Length (m)	Elevation U/S (m)	Elevation D/S (m)	Slope (%)	Velocity (m/s)	Travel Time (min)	Length (m)	Elevation U/S (m)	Elevation D/S (m)	Slope (%)	Velocity (m/s)	Travel Time (min)	Time of Concentration (min)	Time to Peak (min)	Time to Peak (min)	Time to Peak (hrs)	Flow Length	Slope
A1	2.717	100	94	89	5.0%	0.33	5.05	150	89	88	0.5%	0.19	13.16	18	12	12	0.20	250	2%
A2	0.444	40	88	88	0.7%	0.14	4.76	0	0	0	0.0%	0	0.00	5	3	10	0.17	40	1%
B1	1.101	80	88	85	4.1%	0.3	4.44	0	0	0	0.0%	0	0.00	4	3	10	0.17	80	4%
C1	2.298	100	88	86	2.0%	0.21	7.94	25	86	86	2.0%	0.4	1.04	9	6	10	0.17	125	2%
D1	1.273	100	94	89	5.0%	0.33	5.05	70	89	86	4.3%	0.57	2.05	7	5	10	0.17	170	5%
TOTAL:	**7.83**																		

Weighted Curve Number Calculations
Soil type Silty Clay = D

Area ID	Land Use 1	Area	CN	Land Use 2	Area	CN	Land Use 3	Area	CN	Weighted CN
A1	Building & Road	4%	86	Tree Farm	1%	82	Row Crops	95%	89	89
A2	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
B1	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
C1	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
D1	Building & Road	12%	86	Tree Farm	28%	82	Row Crops	60%	89	87

Weighted IA Calculations

Area ID	Land Use 1	Area	S	IA	Land Use 2	Area	S	IA	Land Use 3	Area	S	IA	Weighted IA
A1	Building & Roads	4%	41.35	6.20	Tree Farm	1%	55.76	8.36	Row Crops	95%	31.39	6.28	6.32
A2	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
B1	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
C1	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
D1	Building & Roads	28%	41.35	6.20	Tree Farm	12%	55.76	8.36	Row Crops	60%	31.39	6.28	6.51

Project Name
Pre-Development Model Schematic

NOVATECH
Engineers, Planners & Landscape Architects

8/2/2022
PREPARED BY: NOVATECH

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Stinson Lands
Post-Development Model Parameters

NOVATECH
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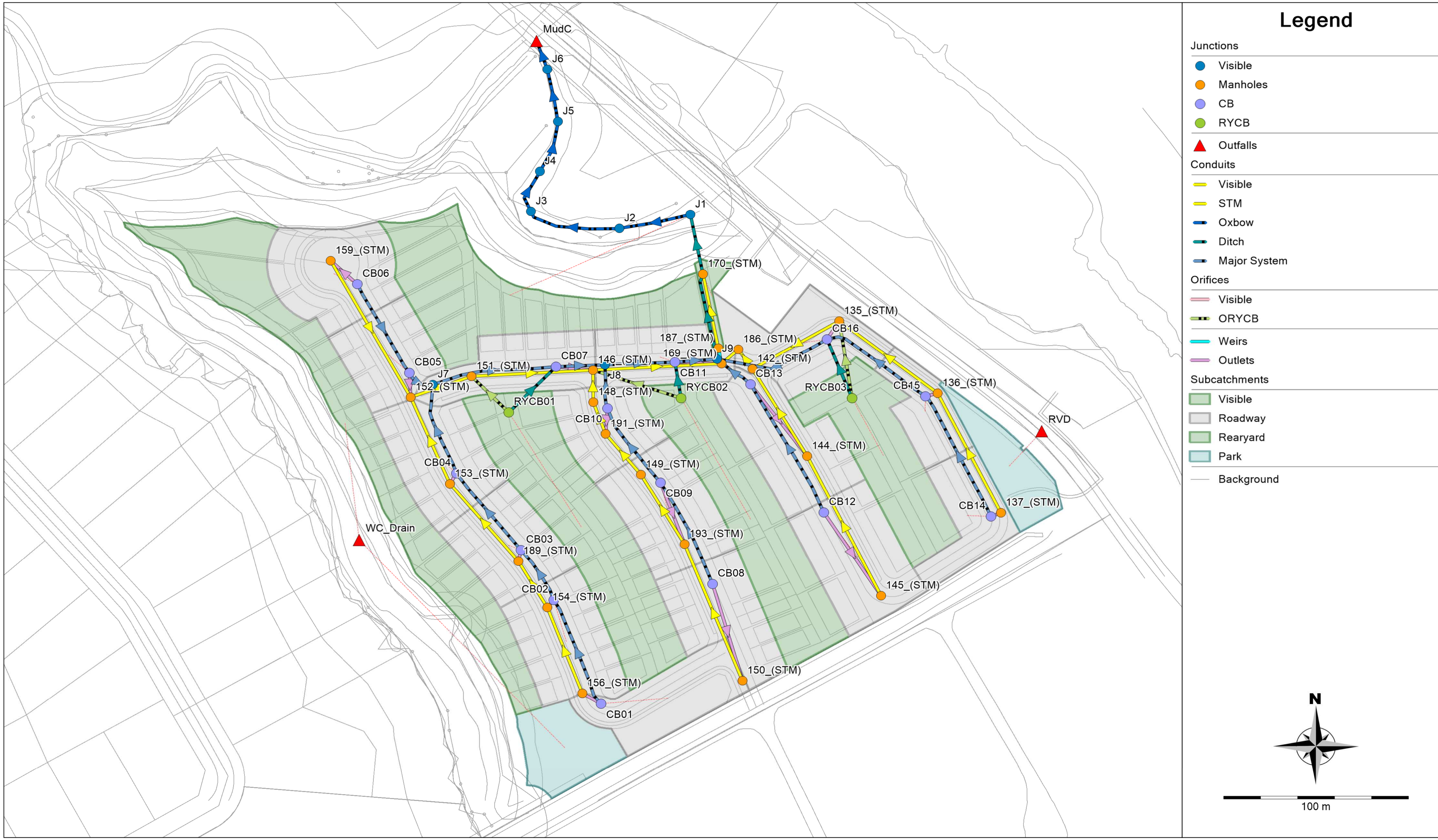
Area ID	Catchment Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.200	0.70	71%	0%	38.18	52.38	1.0%
B-01	0.740	0.45	36%	100%	21.26	348.14	1.0%
C-01	0.280	0.70	71%	45%	23.78	117.77	1.0%
C-02	0.330	0.70	71%	45%	24.82	132.96	1.0%
C-03	0.280	0.70	71%	45%	23.62	118.54	1.0%
C-04	0.310	0.70	71%	45%	22.83	135.79	1.0%
C-05	0.240	0.70	71%	45%	23.31	102.96	1.0%
C-06	0.340	0.70	71%	45%	34.45	98.70	1.0%
C-07	0.590	0.45	36%	100%	39.87	148.00	1.0%
C-08	0.270	0.70	71%	45%	22.14	121.97	1.0%
C-09	0.230	0.70	71%	45%	22.42	102.59	1.0%
C-10	0.220	0.70	71%	45%	22.45	98.00	1.0%
C-11	0.650	0.45	36%	100%	19.05	341.19	1.0%
C-12	0.330	0.70	71%	45%	19.77	166.94	1.0%
C-13	0.290	0.70	71%	45%	23.66	122.59	1.0%
C-14	0.560	0.45	36%	100%	14.10	397.05	1.0%
C-15	0.340	0.70	71%	45%	22.21	153.06	1.0%
C-16	0.350	0.70	71%	45%	21.78	160.71	1.0%
C-17	0.320	0.70	71%	45%	22.95	139.43	1.0%
C-18	0.400	0.45	36%	100%	13.81	289.75	1.0%
C-19	0.170	0.70	71%	45%	19.18	88.63	1.0%
C-20	0.210	0.70	71%	45%	18.84	111.49	1.0%
C-21	0.040	0.45	36%	100%	45.28	8.83	1.0%
D-01	0.180	0.20	0%	0%	20.51	87.76	1.0%
TOTAL:	**7.87**						
		0.59	56%				

Project Name
Overall Model Schematic

NOVATECH
Engineers, Planners & Landscape Architects



8/2/2022
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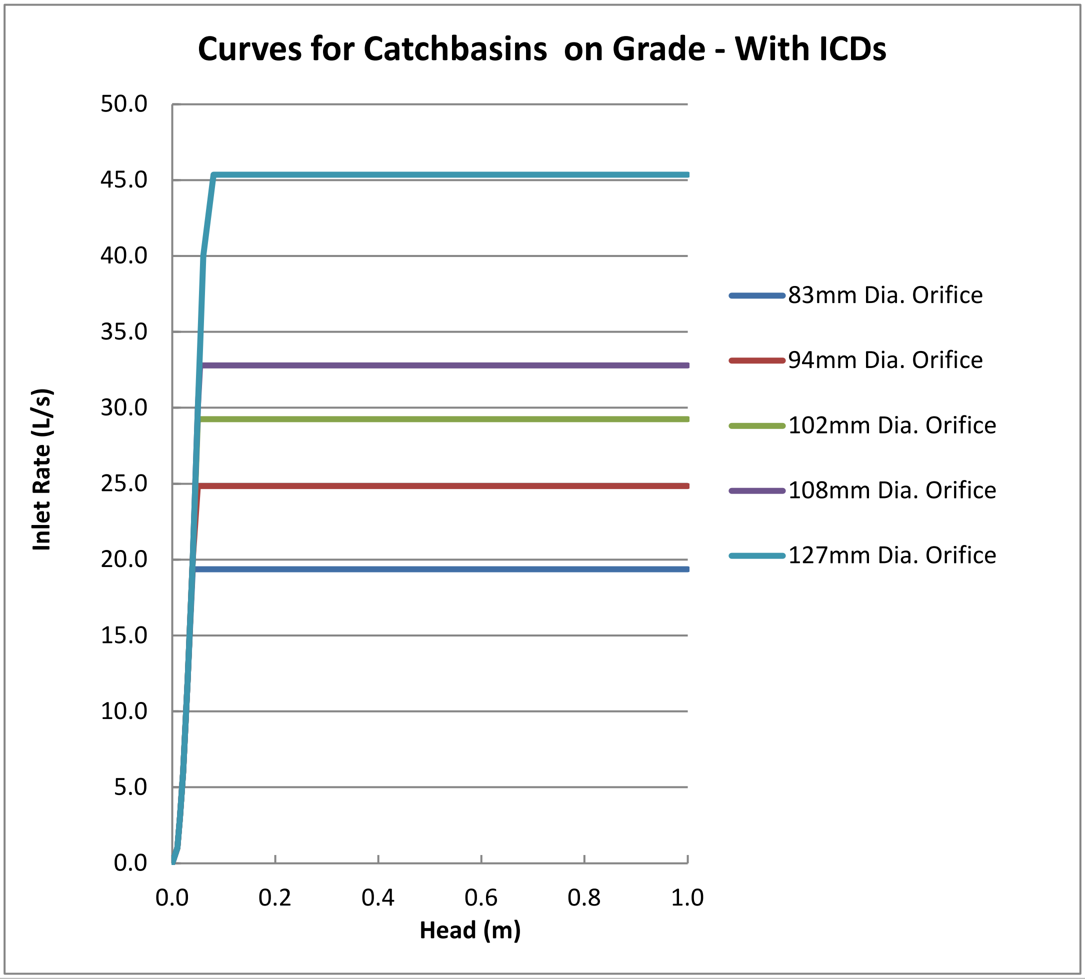


Curb Inlet Catchbasins on Continuous Grade

Depth vs. Captured Flow Curve

A standard depth vs. captured flow curve for catch basins on a continuous grade was provided to Novatech by City staff for use in a dual-drainage model of an existing residential neighbourhood. This standard curve was derived using the inlet curves in Appendix 7A of the Ottawa Sewer Design Guidelines.

Novatech reviewed the methodology used to create this standard curve (described below) and determined that it was suitable for general use in other dual-drainage models.

- MTO Design Chart 4.04 provides the relationship between the gutter flow rate (Q_t) and flow spread (T) for Barrier Curb.
- MTO Design Chart 4.12 provides the relationship between flow spread (T) and flow depth (D).
- The relationship between the gutter flow rate (Q_t) and flow depth (D) was determined for different road slopes using the above charts and Manning's equation (refer to pages 58-60 of the MTO Drainage Management Manual – Part 2);
- The relationship between approach flow (Q_t) and captured flow (Q_c) was determined for different road slopes using the design chart for Barrier Curb with Gutter (Appendix 7-A.2).
- Using the above information, a family of curves was developed to characterize the relationship between flow depth and captured flow for curb inlet catchbasins on different road slopes. The results of this exercise can be summarized as follows:
  - For a given flow depth, the gutter flow rate (Q_t) increases as the road slope increases.
  - The capture efficiency (Q_c) of curb inlet catchbasins decrease as the road slope increases.
  - The net result is that the relationship between flow depth and capture rate is largely independent of road slope: While approach flow vs. captured flow (Q_t vs. Q_c) varies significantly with road grade, flow depth vs. captured flow (D vs. Q_c) does not.

Since there was very little difference in the flow depth vs. captured flow curves for different road slopes, this family of curves was averaged to create a single standard curve for use in dual-drainage models.

Inlet Control Devices

The standard depth vs. capture flow curve was modified to account for the installation of ICDs in curb inlet catchbasins on continuous grade. Separate inlet curves were created for each standard ICD orifice size by capping the inlet rate on the depth vs. capture flow curve at the maximum flow rate through the ICD at a head of 1.2m (depth from centerline of CB lead to top of CICB frame).

Stinson Lands
HGL Elevations

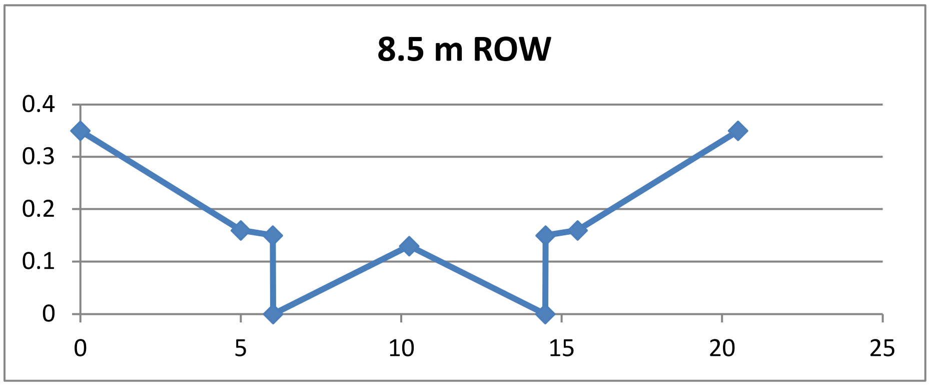
NOVATECH
Engineers, Planners & Landscape Architects

Manhole ID	MH Invert Elevation (m)	T/G Elevation (m)	Outlet pipe invert (m)	Outlet Pipe Diameter (m)	Outlet Pipe Obvert (m)	HGL Elevation (Chicago) (m)	WL Above Obvert (Chicago) (m)
135_(STM)	84.54	87.82	84.84	0.53	85.37	85.41	0.04
136_(STM)	86.44	89.77	86.74	0.30	87.04	86.90	-0.14
137_(STM)	89.89	93.22	90.19	0.25	90.44	90.29	-0.15
142_(STM)	84.33	87.92	84.63	0.60	85.23	85.24	0.01
144_(STM)	86.49	90.32	86.79	0.45	87.24	86.95	-0.29
145_(STM)	89.65	93.28	89.95	0.38	90.33	90.06	-0.27
146_(STM)	84.38	87.82	84.68	0.75	85.43	85.68	0.25
148_(STM)	85.03	87.85	85.33	0.38	85.71	85.83	0.13
149_(STM)	85.87	88.62	86.17	0.30	86.47	86.45	-0.02
150_(STM)	87.62	90.70	87.92	0.25	88.17	88.08	-0.09
151_(STM)	84.65	87.96	84.95	0.68	85.63	85.84	0.22
152_(STM)	84.81	88.20	85.11	0.60	85.71	85.97	0.26
153_(STM)	85.23	88.52	85.53	0.45	85.98	86.11	0.13
154_(STM)	85.74	89.18	86.04	0.38	86.42	86.33	-0.09
156_(STM)	86.08	89.56	86.38	0.30	86.68	86.57	-0.11
159_(STM)	85.27	88.31	85.57	0.45	86.02	86.02	0.00
169_(STM)	84.04	87.65	84.48	0.75	85.23	85.01	-0.22
170_(STM)	83.50	86.83	83.80	1.20	85.00	83.91	-1.09
186_(STM)	83.67	88.05	83.97	1.20	85.17	84.76	-0.41
187_(STM)	83.90	86.96	83.90	1.20	85.10	84.53	-0.57
189_(STM)	85.51	88.96	85.81	0.45	86.26	86.19	-0.07
191_(STM)	85.40	88.13	85.70	0.30	86.00	86.06	0.06
193_(STM)	86.51	89.34	86.81	0.25	87.06	86.97	-0.09

Stinson Lands
Cross-Sections

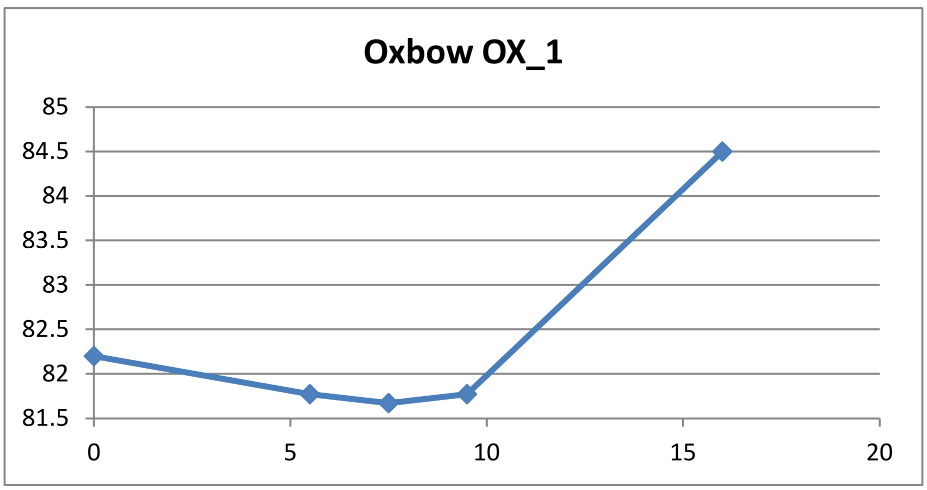
Local ROW

0	0.35
5	0.16
6	0.15
6.01	0
10.25	0.13
14.49	0
14.5	0.15
15.5	0.16
20.5	0.35



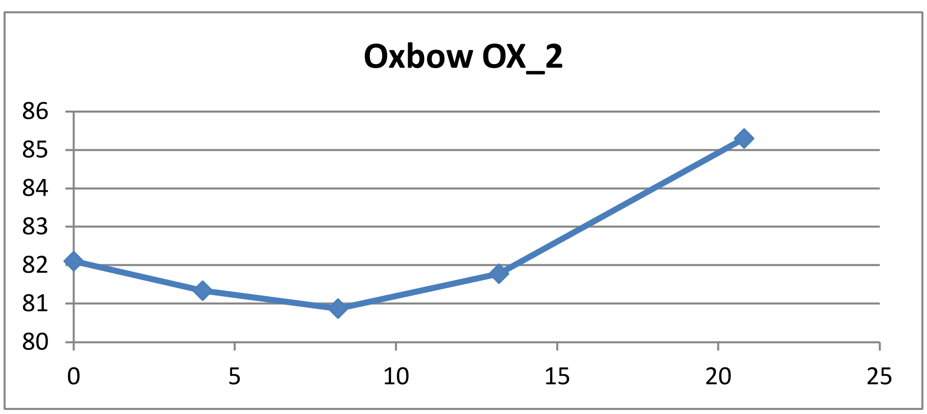
Oxbow OX_1

0	82.2
5.5	81.77
7.5	81.67
9.5	81.77
16	84.5



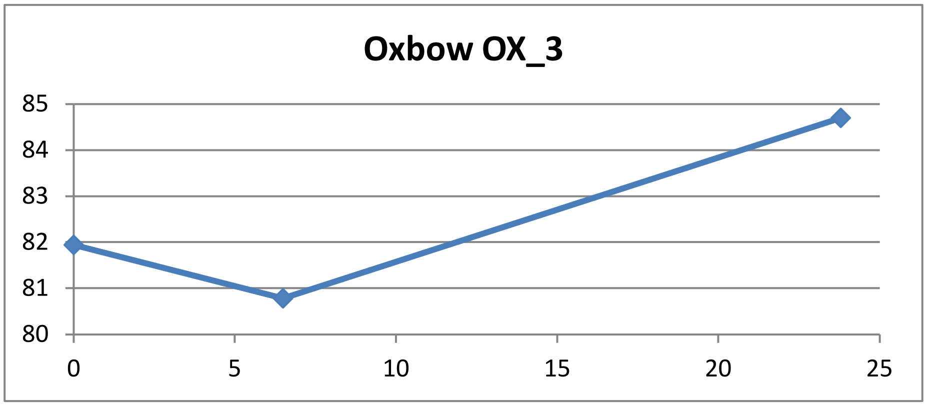
Oxbow OX_2

0	82.1
4	81.34
8.2	80.87
13.2	81.78
20.8	85.3

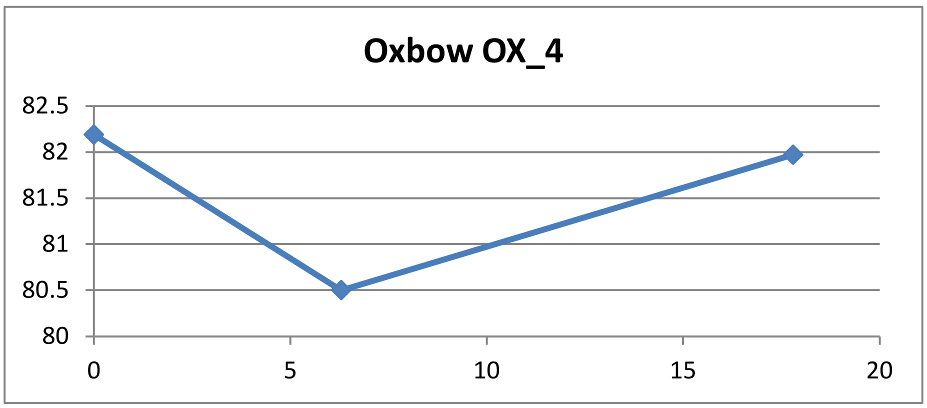


Stinson Lands
Cross-Sections

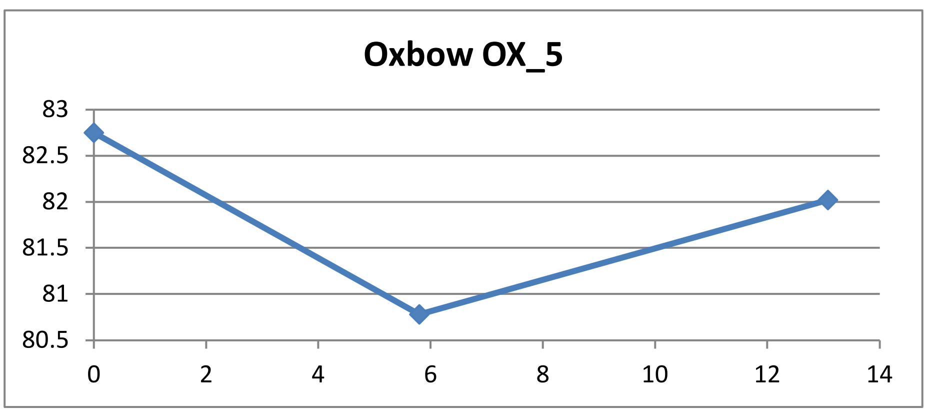
Oxbow OX_3	
0	81.94
6.5	80.78
23.8	84.7



Oxbow OX_4	
0	82.19
6.3	80.5
17.8	81.97

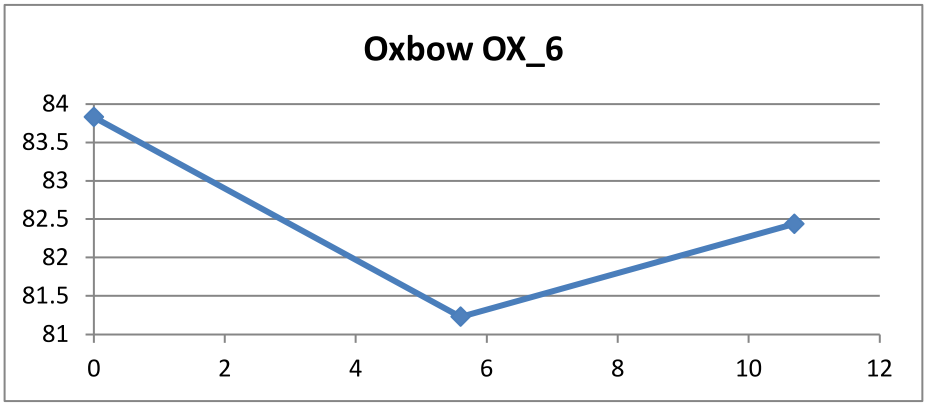


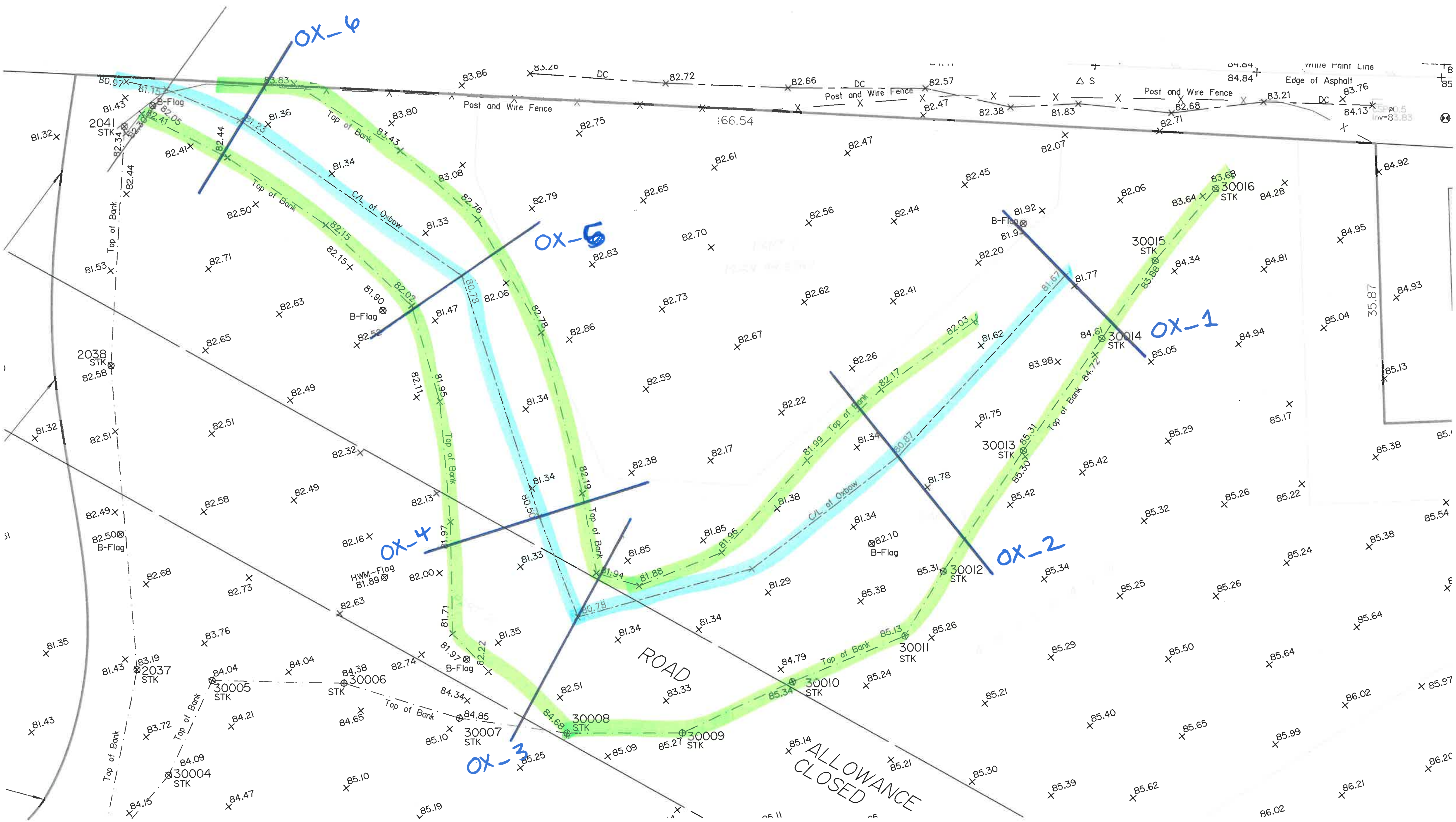
Oxbow OX_5	
0	82.75
5.8	80.78
13.08	82.02



Oxbow OX_6

0	83.83
5.6	81.23
10.7	82.44





oxbow ditch x-sections for PCSWMM.

Stinson Lands
Design Storm Time Series Data
6-hour Chicago Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

C25mm-6.stm		C2-6.stm		C5-6.stm	
Duration	Intensity	Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr	min	mm/hr
0:00	0	0:00	0	0:00	0
0:10	0.9292336	0:10	1.37	0:10	1.78
0:20	1.0106263	0:20	1.49	0:20	1.94
0:30	1.1055844	0:30	1.63	0:30	2.13
0:40	1.2344563	0:40	1.82	0:40	2.37
0:50	1.390459	0:50	2.05	0:50	2.68
1:00	1.6075062	1:00	2.37	1:00	3.1
1:10	1.9059462	1:10	2.81	1:10	3.68
1:20	2.3739543	1:20	3.5	1:20	4.58
1:30	3.1810988	1:30	4.69	1:30	6.15
1:40	4.9513905	1:40	7.3	1:40	9.61
1:50	12.351345	1:50	18.21	1:50	24.17
2:00	52.098123	2:00	76.81	2:00	104.19
2:10	16.332806	2:10	24.08	2:10	32.04
2:20	8.3834501	2:20	12.36	2:20	16.34
2:30	5.6432286	2:30	8.32	2:30	10.96
2:40	4.2731178	2:40	6.3	2:40	8.29
2:50	3.4524079	2:50	5.09	2:50	6.69
3:00	2.9097897	3:00	4.29	3:00	5.63
3:10	2.5231743	3:10	3.72	3:10	4.87
3:20	2.2315171	3:20	3.29	3:20	4.3
3:30	2.0009044	3:30	2.95	3:30	3.86
3:40	1.8177707	3:40	2.68	3:40	3.51
3:50	1.6685508	3:50	2.46	3:50	3.22
4:00	1.5464617	4:00	2.28	4:00	2.98
4:10	1.4379381	4:10	2.12	4:10	2.77
4:20	1.3497626	4:20	1.99	4:20	2.6
4:30	1.2683699	4:30	1.87	4:30	2.44
4:40	1.2005426	4:40	1.77	4:40	2.31
4:50	1.1394981	4:50	1.68	4:50	2.19
5:00	1.0852363	5:00	1.6	5:00	2.08
5:10	1.0309745	5:10	1.52	5:10	1.99
5:20	0.9902781	5:20	1.46	5:20	1.9
5:30	0.9495817	5:30	1.4	5:30	1.82
5:40	0.9088854	5:40	1.34	5:40	1.75
5:50	0.8749717	5:50	1.29	5:50	1.68
6:00	0.8410581	6:00	1.24	6:00	1.62

Stinson Lands
Design Storm Time Series Data
6-hour Chicago Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

C100-6.stm		C100-6+20%.stm	
Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr
0:00	0.00	0:00	0.00
0:10	2.90	0:10	3.48
0:50	3.16	0:50	3.79
1:30	3.48	1:30	4.18
2:10	3.88	2:10	4.66
2:50	4.39	2:50	5.27
3:30	5.07	3:30	6.08
4:10	6.05	4:10	7.26
4:50	7.54	4:50	9.05
5:30	10.16	5:30	12.19
6:10	15.97	6:10	19.16
6:50	40.65	6:50	48.78
7:30	178.56	7:30	214.27
8:10	54.05	8:10	64.86
8:50	27.32	8:50	32.78
9:30	18.24	9:30	21.89
10:10	13.74	10:10	16.49
10:50	11.06	10:50	13.27
11:30	9.29	11:30	11.15
12:10	8.02	12:10	9.62
12:50	7.08	12:50	8.50
13:30	6.35	13:30	7.62
14:10	5.76	14:10	6.91
14:50	5.28	14:50	6.34
15:30	4.88	15:30	5.86
16:10	4.54	16:10	5.45
16:50	4.25	16:50	5.10
17:30	3.99	17:30	4.79
18:10	3.77	18:10	4.52
18:50	3.57	18:50	4.28
19:30	3.40	19:30	4.08
20:10	3.24	20:10	3.89
20:50	3.10	20:50	3.72
21:30	2.97	21:30	3.56
22:10	2.85	22:10	3.42
22:50	2.74	22:50	3.29
23:30	2.64	23:30	3.17

Stinson Lands
Design Storm Time Series Data
SCS Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

S2-12.stm		S5-12.stm		S100-12.stm	
Duration	Intensity	Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr	min	mm/hr
0:00	0.00	0:00	0	0:00	0
0:30	1.27	0:30	1.69	0:30	2.82
1:00	0.59	1:00	0.79	1:00	1.31
1:30	1.10	1:30	1.46	1:30	2.44
2:00	1.10	2:00	1.46	2:00	2.44
2:30	1.44	2:30	1.91	2:30	3.19
3:00	1.27	3:00	1.69	3:00	2.82
3:30	1.69	3:30	2.25	3:30	3.76
4:00	1.69	4:00	2.25	4:00	3.76
4:30	2.29	4:30	3.03	4:30	5.07
5:00	2.88	5:00	3.82	5:00	6.39
5:30	4.57	5:30	6.07	5:30	10.14
6:00	36.24	6:00	48.08	6:00	80.38
6:30	9.23	6:30	12.25	6:30	20.47
7:00	4.06	7:00	5.39	7:00	9.01
7:30	2.71	7:30	3.59	7:30	6.01
8:00	2.37	8:00	3.15	8:00	5.26
8:30	1.86	8:30	2.47	8:30	4.13
9:00	1.95	9:00	2.58	9:00	4.32
9:30	1.27	9:30	1.69	9:30	2.82
10:00	1.02	10:00	1.35	10:00	2.25
10:30	1.44	10:30	1.91	10:30	3.19
11:00	0.93	11:00	1.24	11:00	2.07
11:30	0.85	11:30	1.12	11:30	1.88
12:00	0.85	12:00	1.12	12:00	1.88

Stinson Lands
Design Storm Time Series Data
SCS Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

S2-24.stm		S5-24.stm		S100-24.stm	
Duration min	Intensity mm/hr	Duration min	Intensity mm/hr	Duration min	Intensity mm/hr
0:00	0.00	0:00	0	0:00	0
1:00	0.72	1:00	0.44	1:00	0.6
2:00	0.34	2:00	0.44	2:00	0.75
3:00	0.63	3:00	0.81	3:00	1.39
4:00	0.63	4:00	0.81	4:00	1.39
5:00	0.81	5:00	1.06	5:00	1.81
6:00	0.72	6:00	0.94	6:00	1.6
7:00	0.96	7:00	1.25	7:00	2.13
8:00	0.96	8:00	1.25	8:00	2.13
9:00	1.30	9:00	1.68	9:00	2.88
10:00	1.63	10:00	2.12	10:00	3.63
11:00	2.59	11:00	3.37	11:00	5.76
12:00	20.55	12:00	26.71	12:00	45.69
13:00	5.23	13:00	6.8	13:00	11.64
14:00	2.30	14:00	2.99	14:00	5.12
15:00	1.54	15:00	2	15:00	3.42
16:00	1.34	16:00	1.75	16:00	2.99
17:00	1.06	17:00	1.37	17:00	2.35
18:00	1.11	18:00	1.44	18:00	2.46
19:00	0.72	19:00	0.94	19:00	1.6
20:00	0.58	20:00	0.75	20:00	1.28
21:00	0.81	21:00	1.06	21:00	1.81
22:00	0.53	22:00	0.68	22:00	1.17
23:00	0.48	23:00	0.63	23:00	1.07
0:00	0.48	0:00	0.63	0:00	1.07

VORTECHS SYSTEM® ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS
STINSON SUBDIVISION (4386 RIDEAU VALLEY DRIVE)
OTTAWA, ON
MODEL PC1421 OFF-LINE

CONTECH ENGINEERED SOLUTIONS

Design Ratio[1] = $\frac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(14.3 \text{ m2})}$ **= 0.79**

Rainfall Intensity mm/hr	Operating Rate[2] % of capacity	Flow Treated (l/s)	% Total Rainfall Volume[3]	Rmvl. Effcy[4] (%)	Rel. Effcy (%)
0.5	0.6	5.8	9.2%	100.0%	9.2%
1.0	1.2	11.5	10.6%	98.0%	10.4%
1.5	1.7	17.3	9.9%	98.0%	9.7%
2.0	2.3	23.0	8.4%	98.0%	8.2%
2.5	2.9	28.8	7.7%	98.0%	7.5%
3.0	3.5	34.5	5.9%	98.0%	5.8%
3.5	4.1	40.3	4.4%	98.0%	4.3%
4.0	4.6	46.0	4.7%	98.0%	4.6%
4.5	5.2	51.8	3.3%	98.0%	3.3%
5.0	5.8	57.6	3.0%	98.0%	3.0%
6.0	7.0	69.1	5.4%	98.0%	5.3%
7.0	8.1	80.6	4.4%	96.9%	4.2%
8.0	9.3	92.1	3.5%	96.3%	3.4%
9.0	10.5	103.6	2.8%	96.0%	2.7%
10.0	11.6	115.1	2.2%	95.3%	2.1%
15.0	17.4	172.7	7.0%	89.9%	6.3%
20.0	23.2	230.2	4.5%	85.7%	3.9%
25.0	29.0	287.8	1.4%	82.6%	1.2%
30.0	34.8	345.4	0.7%	80.0%	0.5%
35.0	40.6	402.9	0.5%	76.0%	0.4%
40.0	46.5	460.5	0.5%	69.0%	0.4%
					96.2%

Predicted Annual Runoff Volume Treated =	93.5%
Assumed Removal Efficiency of remaining % =	0.0%
Removal Efficiency Adjustment[5] =	6.5%
Predicted Net Annual Load Removal Efficiency =	**90%**

1 - Design Ratio = (Total Drainage Area) x (Runoff Coefficient) x (Rational Method Conversion) / Grit Chamber Area
- The Total Drainage Area and Runoff Coefficient are specified by the site engineer.
- The rational method conversion based on the units in the above equation is 2.775.

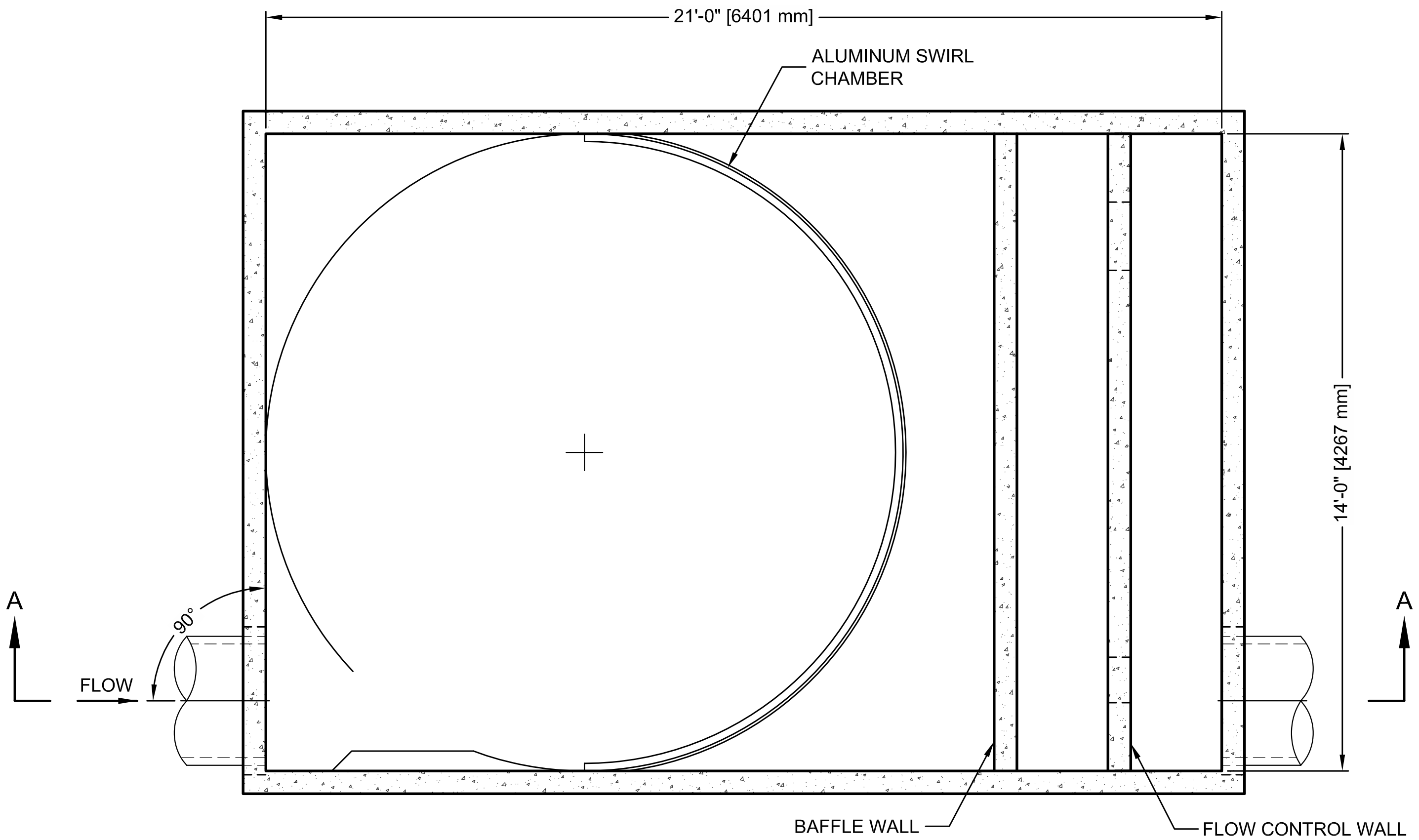
2 - Operating Rate (% of capacity) = percentage of peak operating rate of 68 l/s/m^2.
3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON
4 - Based on Contech Construction Products laboratory verified removal of an average particle size of 80 microns (see Technical Bulletin #1).
5- Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.

Calculated by:	JAK	7/26	Checked by:

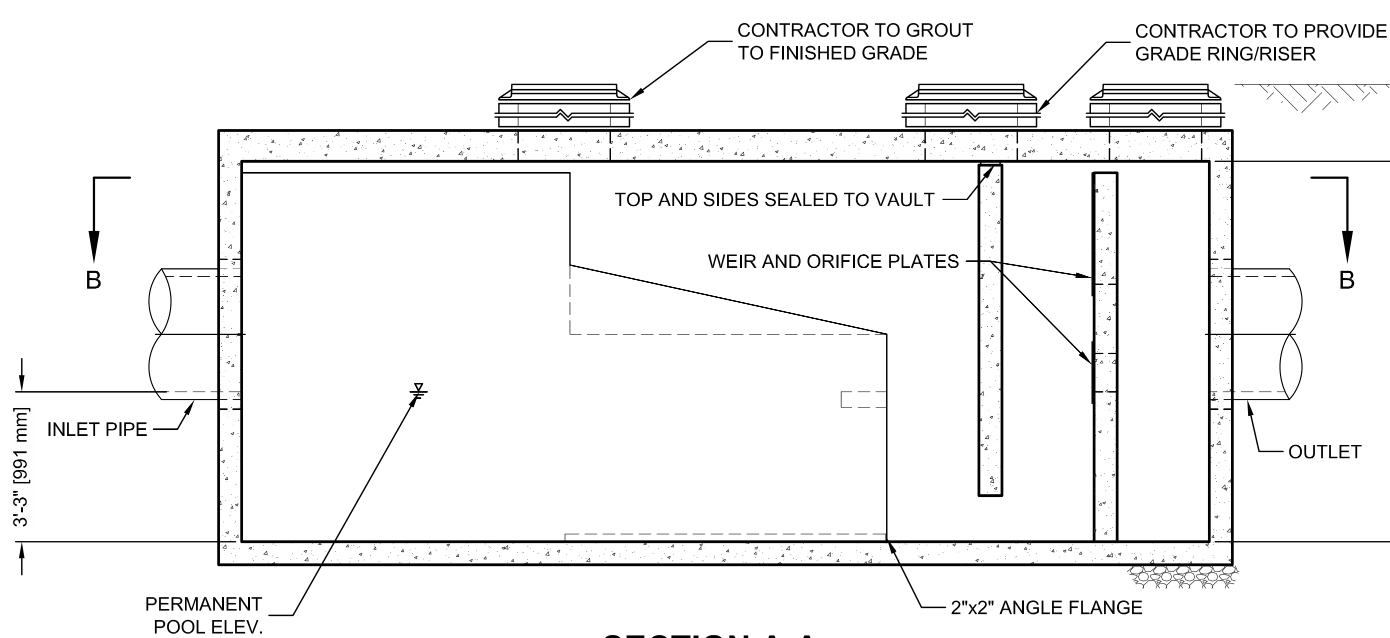
VORTECHS PC1421 DESIGN NOTES

VORTECHS PC1421 RATED TREATMENT CAPACITY IS 34 CFS, OR PER LOCAL REGULATIONS. IF THE SITE CONDITIONS EXCEED RATED TREATMENT CAPACITY, AN UPSTREAM BYPASS STRUCTURE IS REQUIRED.

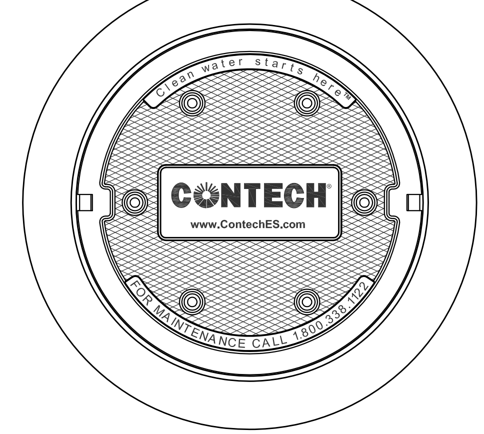
THE STANDARD INLET/OUTLET CONFIGURATION IS SHOWN. FOR OTHER CONFIGURATION OPTIONS , PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS, LLC REPRESENTATIVE. www.ContechES.com



SECTION B-B



SECTION A-A



FRAME AND COVER
(DIAMETER VARIES)
N.T.S.

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID			*
WATER QUALITY FLOW RATE (CFS)			*
PEAK FLOW RATE (CFS)			*
RETURN PERIOD OF PEAK FLOW (YRS)			*
PIPE DATA:	I.E.	MATERIAL	DIAMETER
INLET PIPE 1	*	*	*
INLET PIPE 2	*	*	*
OUTLET PIPE	*	*	*
RIM ELEVATION			*
ANTI-FLOTATION BALLAST		WIDTH	HEIGHT
		*	*

NOTES/SPECIAL REQUIREMENTS:

* PER ENGINEER OF RECORD

GENERAL NOTES
1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHT, PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS, LLC REPRESENTATIVE. www.ContechES.com
4. VORTECHS WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.
5. STRUCTURE SHALL MEET AASHTO HS20 AND CASTINGS SHALL MEET AASHTO M306 LOAD RATING, ASSUMING GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION.
6. INLET PIPE(S) MUST BE PERPEDICULAR TO THE VAULT AND AT THE CORNER TO INTRODUCE THE FLOW TANGENTIALLY TO THE SWIRL CHAMBER. DUAL INLETS NOT TO HAVE OPPOSING TANGENTIAL FLOW DIRECTIONS.
7. OUTLET PIPE(S) MUST BE DOWN STREAM OF THE FLOW CONTROL BAFFLE AND MAY BE LOCATED ON THE SIDE OR END OF THE VAULT. THE FLOW CONTROL WALL MAY BE TURNED TO ACCOMODATE OUTLET PIPE KNOCKOUTS ON THE SIDE OF THE VAULT.

INSTALLATION NOTES
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE VORTECHS STRUCTURE (LIFTING CLUTCHES PROVIDED).
C. CONTRACTOR TO INSTALL JOINT SEALANT BETWEEN ALL STRUCTURE SECTIONS AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES. MATCH PIPE INVERTS WITH ELEVATIONS SHOWN.
E. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO ASSURE UNIT IS WATER TIGHT, HOLDING WATER TO FLOWLINE INVERT MINIMUM. IT IS SUGGESTED THAT ALL JOINTS BELOW PIPE INVERTS ARE GROUTED.

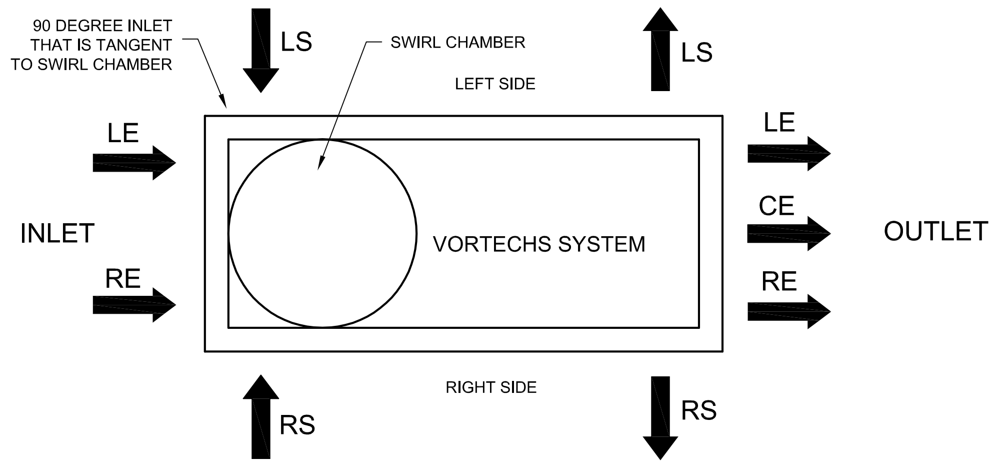
Vortechs®

CONTECH® ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

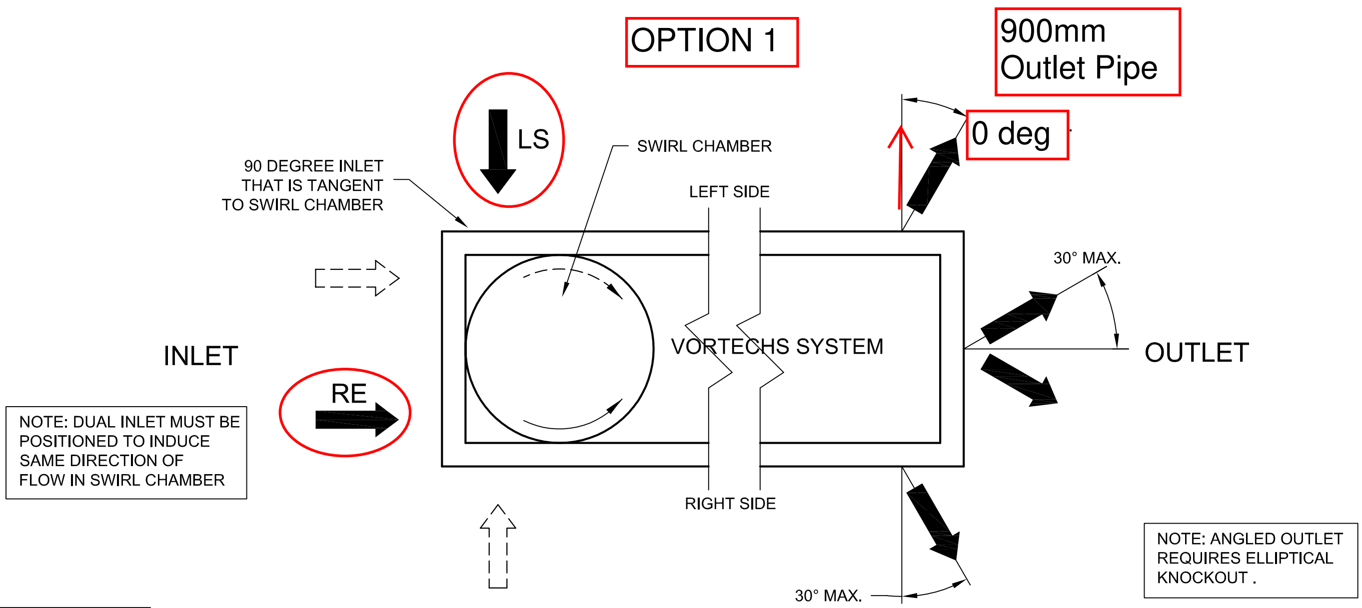
VORTECHS PC1421
STANDARD DETAIL

I:\COMMON\CAD\TREATMENT\20 VORTECHS\40 STANDARD DRAWINGS\DWG\VX-PC1421-DTL.DWG 8/6/2014 2:45 PM

NOTE: INLET PIPE MUST BE PERPENDICULAR TO WALL IT IS ENTERING ON



TYPICAL INLET/OUTLET ORIENTATION OPTIONS



ORIENTATION KEY

LE = LEFT END
RE = RIGHT END
LS = LEFT SIDE
RS = RIGHT SIDE
CE = CENTER END

DUAL INLET ORIENTATION OPTIONS

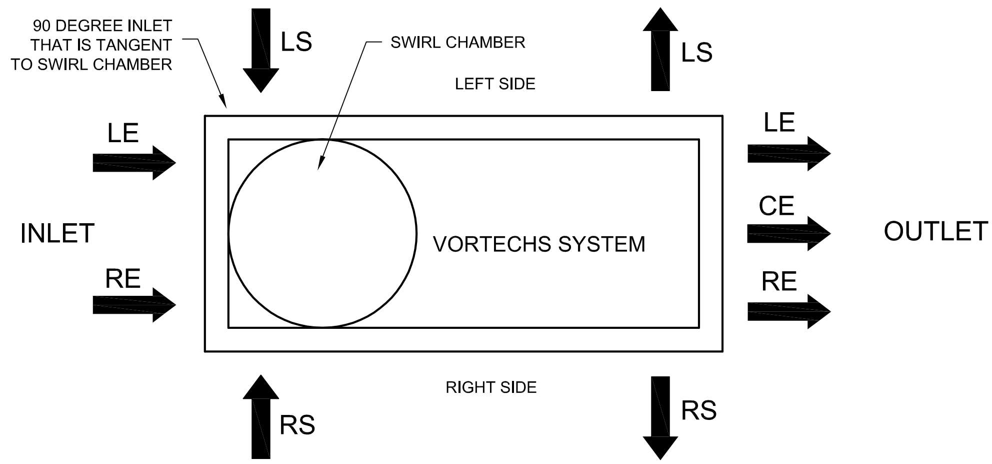
ANGLED OUTLET ORIENTATION OPTIONS

CONTECH® STORMWATER SOLUTIONS INC.
contechstormwater.com

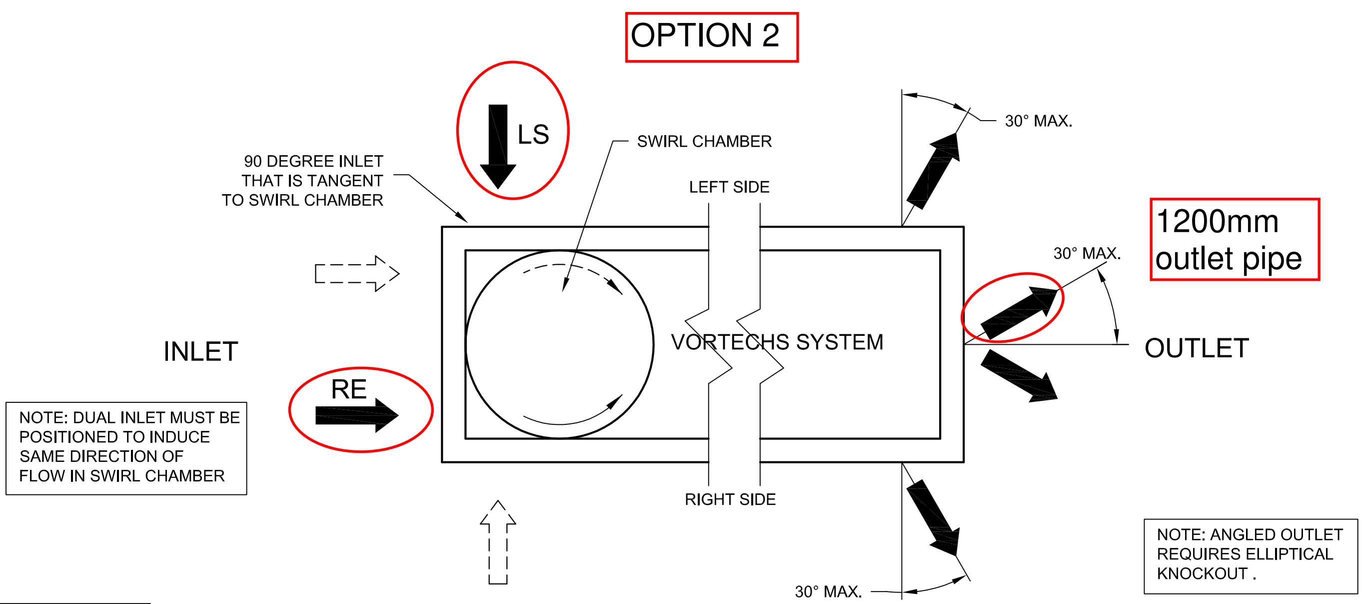
TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06	SCALE: NONE	FILE NAME: TYPVX ORIENTATION	DRAWN: GMC	CHECKED: NDG

NOTE: INLET PIPE MUST BE PERPENDICULAR TO WALL IT IS ENTERING ON



TYPICAL INLET/OUTLET ORIENTATION OPTIONS



DUAL INLET ORIENTATION OPTIONS

ANGLED OUTLET ORIENTATION OPTIONS

ORIENTATION KEY

LE = LEFT END
RE = RIGHT END
LS = LEFT SIDE
RS = RIGHT SIDE
CE = CENTER END

CONTECH® STORMWATER SOLUTIONS INC.
contechstormwater.com

TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06	SCALE: NONE	FILE NAME: TYPVX ORIENTATION	DRAWN: GMC	CHECKED: NDG

VORTECHS SYSTEM® ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS

Stinson Subdivision (4386 Rideau Valley Drive)
Ottawa, ON
Model 1522CIP In-line

CONTECH ENGINEERED SOLUTIONS

Design Ratio[1] = $\frac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(16.4 \text{ m2})}$ **= 0.69**

Rainfall Intensity mm/hr	Operating Rate[2] % of capacity	Flow Treated (l/s)	% Total Rainfall Volume[3]	Rmvl. Effcy[4] (%)	Rel. Effcy (%)
0.5	0.5	5.6	9.2%	98.0%	9.0%
1.0	1.0	11.2	10.6%	98.0%	10.4%
1.5	1.5	16.8	9.9%	98.0%	9.7%
2.0	2.0	22.4	8.4%	98.0%	8.2%
2.5	2.5	27.9	7.7%	98.0%	7.5%
3.0	3.0	33.5	5.9%	97.9%	5.8%
3.5	3.5	39.1	4.4%	97.9%	4.3%
4.0	4.0	44.7	4.7%	97.1%	4.5%
4.5	4.6	50.3	3.3%	97.1%	3.2%
5.0	5.1	55.9	3.0%	96.3%	2.9%
6.0	6.1	67.1	5.4%	95.6%	5.1%
7.0	7.1	78.2	4.4%	95.0%	4.1%
8.0	8.1	89.4	3.5%	93.7%	3.3%
9.0	9.1	100.6	2.8%	92.6%	2.6%
10.0	10.1	111.8	2.2%	91.9%	2.0%
15.0	15.2	167.6	7.0%	86.7%	6.1%
20.0	20.2	223.5	4.5%	81.4%	3.7%
25.0	25.3	279.4	1.4%	77.0%	1.1%
30.0	30.4	335.3	0.7%	73.1%	0.5%
35.0	35.4	391.1	0.5%	69.7%	0.3%
					94.5%
				Predicted Annual Runoff Volume Treated =	99.5%
				Assumed Removal Efficiency of remaining % =	0.0%
				Removal Efficiency Adjustment[5] =	0.0%
				Predicted Net Annual Load Removal Efficiency =	**94%**

1 - Design Ratio = (Total Drainage Area) x (Runoff Coefficient) x (Rational Method Conversion) / Grit Chamber Area
- The Total Drainage Area and Runoff Coefficient are specified by the site engineer.
- The rational method conversion based on the units in the above equation is 2.775.

2 - Operating Rate (% of capacity) = percentage of peak operating rate of 68 l/s/m^2.
3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON
4 - Based on Contech Stormwater Solutions laboratory verified removal of an average particle size of 80 microns (see Technical Bulletin #1).
5- Increase due to comparison of flows based on historical rational rainfall method and actual modeled by specifying engineer.

Calculated by: JAK 8/1/2022	Checked by:

Plunge Pool Calculations

Reference calculations are from the FHWA Hydraulic Design of Energy Dissipators for Culverts and Channels, Chapter 10: Riprap Basins and Aprons. Section 10 has been provided following these calculations.

Preliminary calculations for the sizing of the basin follow the recommendations outlined in Section 10.1 and as referencing Figures 10.1 and 10.2 as follows:

- The basin is pre-shaped and lined with riprap approximately $2D_{50}$ thick.
  - 300mm riprap has been selected, so D_{50} is 150mm. Proposed thickness of the basin is 600mm, which exceeds this recommendation.
- The riprap floor is constricted at the approximate depth of scour, h_S, that would occur in a thick pad of riprap. The h_S/D_{50} of the material should be greater than 2.
  - Plunge pool is designed to have a depth of 350mm, this gives h_S/D_{50} of >2.
- The length of the energy dissipating pool, Ls, is $10h_S$, but no less than $3W_O$; the length of the apron, L_A, is $5h_S$, but no less than W_O. The overall length of the basin (pool plus apron), L_B, is $15h_S$, but no less than $4W_O$.
  - For the energy dissipating pool:
    - $10h_S = 10*0.60m = 6.0$ m, or $3W_O = 3*1.2m = 3.6m$ minimum
    - Designed L_S is 5.7m, which is > $3W_O$ and just 0.3m shy of $10h_S$.
  - Length of the apron:
    - $L_A = 5h_S = 5*0.60m = 1.75m$, which is > W_O
  - Overall length of the basin:
    - $15hS = 15*0.35m = 5.25m$, which is > $4W_O$
    - Actual overall length of the basin is 7.45m
- A riprap cutoff wall or sloping apron can be constricted if downstream channel degradation is anticipated as shown in Figure 10.1.

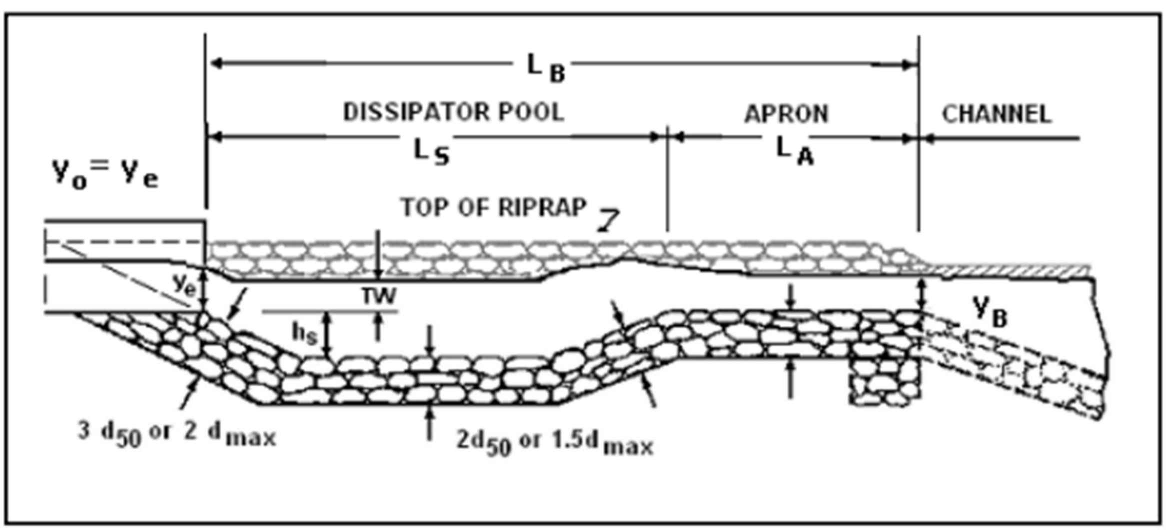


Figure 10.1. Profile of Riprap Basin

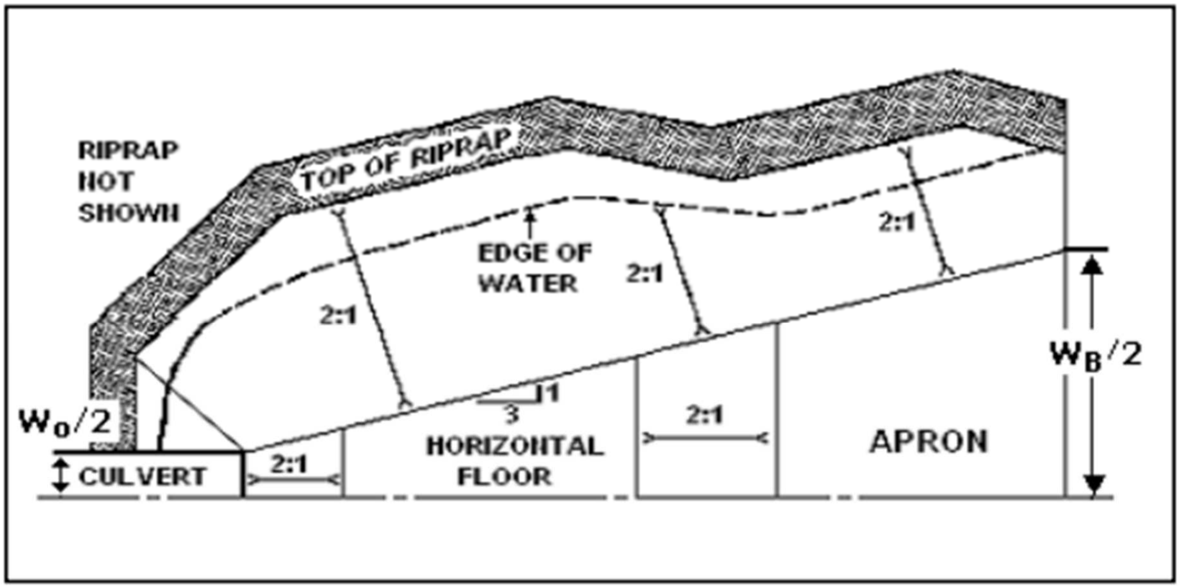


Figure 10.2. Half Plan of Riprap Basin

Using the proposed plunge pool cross-sectional dimensions, the outlet velocity from the maximum outlet peak flow (100-year) has been calculated using V=Q/A

Cross-sectional area calculated using the equation for the area of a trapezoid:

$$A = \left(\frac{W_T + W_B}{2}\right) * D$$

$$A = \left(\frac{3.87 + 10.57}{2}\right) * 0.35$$

$$A = 2.53m^3$$

Using the 100-year combined peak flow entering the plunge pool (2.3cms)

$$V = \frac{Q}{A}$$

$$V = \frac{2.3cms}{2.53m^3}$$

$$V = 0.91m/s$$

CHAPTER 10: RIPRAP BASINS AND APRONS

Riprap is a material that has long been used to protect against the forces of water. The material can be pit-run (as provided by the supplier) or specified (standard or special). State DOTs have standard specifications for a number of classes (sizes or gradations) of riprap. Suppliers maintain an inventory of frequently used classes. Special gradations of riprap are produced on-demand and are therefore more expensive than both pit-run and standard classes.

This chapter includes discussion of both riprap aprons and riprap basin energy dissipators. Both can be used at the outlet of a culvert or chute (channel) by themselves or at the exit of a stilling basin or other energy dissipator to protect against erosion downstream. Section 10.1 provides a design procedure for the riprap basin energy dissipator that is based on armoring a pre-formed scour hole. The riprap for this basin is a special gradation. Section 10.2 includes discussion of riprap aprons that provide a flat armored surface as the only dissipator or as additional protection at the exit of other dissipators. The riprap for these aprons is generally from State DOT standard classes. Section 10.3 provides additional discussion of riprap placement downstream of energy dissipators.

10.1 RIPRAP BASIN

The design procedure for the riprap basin is based on research conducted at Colorado State University (Simons, et al., 1970; Stevens and Simons, 1971) that was sponsored by the Wyoming Highway Department. The recommended riprap basin that is shown on Figure 10.1 and Figure 10.2 has the following features:

- The basin is pre-shaped and lined with riprap that is at least $2D_{50}$ thick.
- The riprap floor is constructed at the approximate depth of scour, h_s, that would occur in a thick pad of riprap. The h_s/D_{50} of the material should be greater than 2.
- The length of the energy dissipating pool, L_s, is $10h_s$, but no less than $3W_o$; the length of the apron, L_A, is $5h_s$, but no less than W_o. The overall length of the basin (pool plus apron), L_B, is $15h_s$, but no less than $4W_o$.
- A riprap cutoff wall or sloping apron can be constructed if downstream channel degradation is anticipated as shown in Figure 10.1.

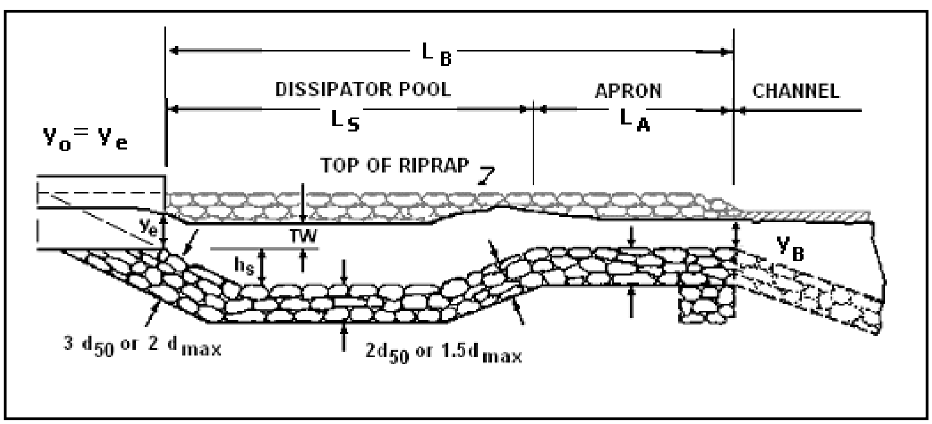


Figure 10.1. Profile of Riprap Basin

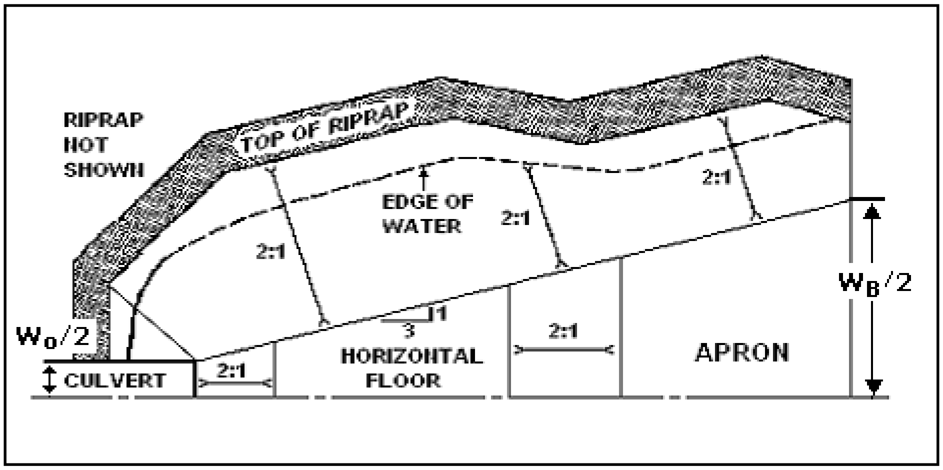


Figure 10.2. Half Plan of Riprap Basin

10.1.1 Design Development

Tests were conducted with pipes from 152 mm (6 in) to 914 mm (24 in) and 152 mm (6 in) high model box culverts from 305 mm (12 in) to 610 mm (24 in) in width. Discharges ranged from 0.003 to 2.8 m^3/s (0.1 to 100 ft^3/s). Both angular and rounded rock with an average size, D_{50}, ranging from 6 mm (1.4 in) to 177 mm (7 in) and gradation coefficients ranging from 1.05 to 2.66 were tested. Two pipe slopes were considered, 0 and 3.75%. In all, 459 model basins were studied. The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics:

- The scour hole depth, h_s; length, L_s; and width, W_s, are related to the size of riprap, D_{50}; discharge, Q; brink depth, y_o; and tailwater depth, TW.
- Rounded material performs approximately the same as angular rock.
- For low tailwater ($TW/y_o < 0.75$), the scour hole functions well as an energy dissipator if $h_s/D_{50} > 2$. The flow at the culvert brink plunges into the hole, a jump forms and flow is generally well dispersed.
- For high tailwater ($TW/y_o > 0.75$), the high velocity core of water passes through the basin and diffuses downstream. As a result, the scour hole is shallower and longer.
- The mound of material that forms downstream contributes to the dissipation of energy and reduces the size of the scour hole. If the mound is removed, the scour hole enlarges somewhat.

Plots were constructed of h_s/y_e versus $V_o/(gy_e)^{1/2}$ with D_{50}/y_e as the third variable. Equivalent brink depth, y_e, is defined to permit use of the same design relationships for rectangular and circular culverts. For rectangular culverts, $y_e = y_o$ (culvert brink depth). For circular culverts, $y_e = (A/2)^{1/2}$, where A is the brink area.

Anticipating that standard or modified end sections would not likely be used when a riprap basin is located at a culvert outlet, the data with these configurations were not used to develop the design relationships. This assumption reduced the number of applicable runs to 346. A total of 128 runs had a D_{50}/y_e of less than 0.1. These data did not exhibit relationships that appeared useful for design and were eliminated. An additional 69 runs where $h_s/D_{50}<2$ were also eliminated by the authors of this edition of HEC 14. These runs were not considered reliable for design, especially those with $h_s = 0$. Therefore, the final design development used 149 runs from the study. Of these, 106 were for pipe culverts and 43 were for box culverts. Based on these data, two design relationships are presented here: an envelope design and a best fit design.

To balance the need for avoiding an underdesigned basin against the costs of oversizing a basin, an envelope design relationship in the form of Equation 10.1 and Equation 10.2 was developed. These equations provide a design envelope for the experimental data equivalent to the design figure (Figure XI-2) provided in the previous edition of HEC 14 (Corry, et al., 1983). Equations 10.1 and 10.2, however, improve the fit to the experimental data reducing the root-mean-square (RMS) error from 1.24 to 0.83.

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o \qquad (10.1)$$

where,

h_s	=	dissipator pool depth, m (ft)
y_e	=	equivalent brink (outlet) depth, m (ft)
D_{50}	=	median rock size by weight, m (ft)
C_o	=	tailwater parameter

The tailwater parameter, C_o, is defined as:

$$\begin{aligned} C_o &= 1.4 && TW/y_e < 0.75 \\ C_o &= 4.0(TW/y_e) - 1.6 && 0.75 < TW/y_e < 1.0 \\ C_o &= 2.4 && 1.0 < TW/y_e \end{aligned} \qquad (10.2)$$

A best fit design relationship that minimizes the RMS error when applied to the experimental data was also developed. Equation 10.1 still applies, but the description of the tailwater parameter, C_o, is defined in Equation 10.3. The best fit relationship for Equations 10.1 and 10.3 exhibits a RMS error on the experimental data of 0.56.

$$\begin{aligned} C_o &= 2.0 && TW/y_e < 0.75 \\ C_o &= 4.0(TW/y_e) - 1.0 && 0.75 < TW/y_e < 1.0 \\ C_o &= 3.0 && 1.0 < TW/y_e \end{aligned} \qquad (10.3)$$

Use of the envelope design relationship (Equations 10.1 and 10.2) is recommended when the consequences of failure at or near the design flow are severe. Use of the best fit design relationship (Equations 10.1 and 10.3) is recommended when basin failure may easily be addressed as part of routine maintenance. Intermediate risk levels can be adopted by the use of intermediate values of C_o.

10.1.2 Basin Length

Frequency tables for both box culvert data and pipe culvert data of relative length of scour hole ($L_s/h_s < 6$, $6 < L_s/h_s < 7$, $7 < L_s/h_s < 8$ . . . $25 < L_s/h_s < 30$), with relative tailwater depth TW/y_e in increments of 0.03 m (0.1 ft) as a third variable, were constructed using data from 346 experimental runs. For box culvert runs L_s/h_s was less than 10 for 78% of the data and L_s/h_s was less than 15 for 98% of the data. For pipe culverts, L_s/h_s was less than 10 for 91% of the data and, L_s/h_s was less than 15 for all data. A 3:1 flare angle is recommended for the basins walls. This angle will provide a sufficiently wide energy dissipating pool for good basin operation.

10.1.3 High Tailwater

Tailwater influenced formation of the scour hole and performance of the dissipator. For tailwater depths less than 0.75 times the brink depth, scour hole dimensions were unaffected by tailwater. Above this the scour hole became longer and narrower. The tailwater parameter defined in Equations 10.2 and 10.3 captures this observation. In addition, under high tailwater conditions, it is appropriate to estimate the attenuation of the flow velocity downstream of the culvert outlet using Figure 10.3. This attenuation can be used to determine the extent of riprap protection required. HEC 11 (Brown and Clyde, 1989) or the method provided in Section 10.3 can be used for sizing riprap.

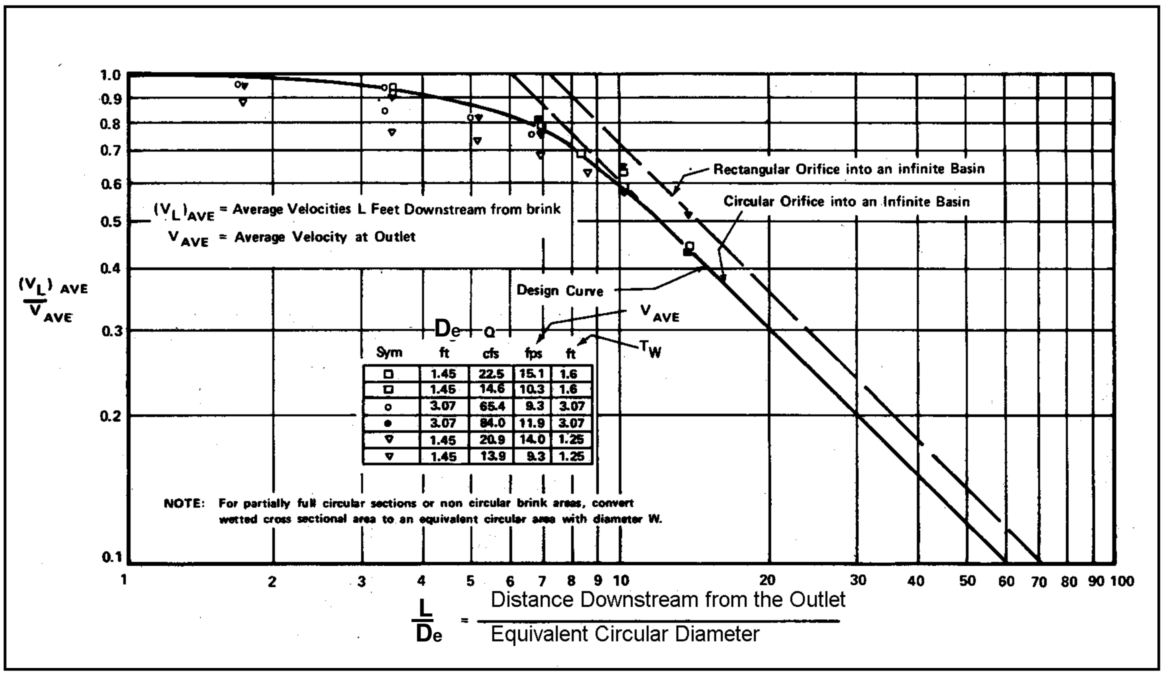


Figure 10.3. Distribution of Centerline Velocity for Flow from Submerged Outlets

10.1.4 Riprap Details

Based on experience with conventional riprap design, the recommended thickness of riprap for the floor and sides of the basin is $2D_{50}$ or $1.50D_{max}$, where D_{max} is the maximum size of rock in the riprap mixture. Thickening of the riprap layer to $3D_{50}$ or $2D_{max}$ on the foreslope of the roadway culvert outlet is warranted because of the severity of attack in the area and the necessity for preventing undermining and consequent collapse of the culvert. Figure 10.1 illustrates these riprap details. The mixture of stone used for riprap and need for a filter should meet the specifications described in HEC 11 (Brown and Clyde, 1989).

10.1.5 Design Procedure

The design procedure for a riprap basin is as follows:

Step 1. Compute the culvert outlet velocity, V_o, and depth, y_o.

For subcritical flow (culvert on mild or horizontal slope), use Figure 3.3 or Figure 3.4 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert or diameter of a circular culvert.

For supercritical flow (culvert on a steep slope), V_o will be the normal velocity obtained by using the Manning's Equation for appropriate slope, section, and discharge.

Compute the Froude number, Fr, for brink conditions using brink depth for box culverts (y_e=y_o) and equivalent depth ($y_e = (A/2)^{1/2}$) for non-rectangular sections.

Step 2. Select D_{50} appropriate for locally available riprap. Determine C_o from Equation 10.2 or 10.3 and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$. If h_s/D_{50} or D_{50}/y_e is out of this range, try a different riprap size. (Basins sized where h_s/D_{50} is greater than, but close to, 2 are often the most economical choice.)

Step 3. Determine the length of the dissipation pool (scour hole), L_s, total basin length, L_B, and basin width at the basin exit, W_B, as shown in Figures 10.1 and 10.2. The walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

Step 4. Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$ and compare with the allowable exit velocity, V_{allow}. The allowable exit velocity may be taken as the estimated normal velocity in the tailwater channel or a velocity specified based on stability criteria, whichever is larger. Critical depth at the basin exit may be determined iteratively using Equation 7.14:

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$ by trial and success to determine y_B.

$V_c = Q/A_c$

z = basin side slope, z:1 (H:V)

If $V_c \leq V_{allow}$, the basin dimensions developed in step 3 are acceptable. However, it may be possible to reduce the size of the dissipator pool and/or the apron with a larger riprap size. It may also be possible to maintain the dissipator pool, but reduce the flare on the apron to reduce the exit width to better fit the downstream channel. Steps 2 through 4 are repeated to evaluate alternative dissipator designs.

Step 5. Assess need for additional riprap downstream of the dissipator exit. If $TW/y_o \leq 0.75$, no additional riprap is needed. With high tailwater ($TW/y_o \geq 0.75$), estimate centerline velocity at a series of downstream cross sections using Figure 10.3 to determine the size and extent of additional protection. The riprap design details should be in accordance with specifications in HEC 11 (Brown and Clyde, 1989) or similar highway department specifications.

Two design examples are provided. The first features a box culvert on a steep slope while the second shows a pipe culvert on a mild slope.

Design Example: Riprap Basin (Culvert on a Steep Slope) (SI)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and 10.2) for a 2440 mm by 1830 mm reinforced concrete box (RCB) culvert that is in inlet control with supercritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 2.1 m/s. Riprap is available with a D_{50} of 0.50, 0.55, and 0.75 m. Consider two tailwater conditions: 1) TW = 0.85 m and 2) TW = 1.28 m. Given:

Q = 22.7 m^3/s

y_o = 1.22 m (normal flow depth) = brink depth

Solution

Step 1. Compute the culvert outlet velocity, V_o, depth, y_o, and Froude number for brink conditions. For supercritical flow (culvert on a steep slope), V_o will be V_n

$y_o = y_e = 1.22$ m

$V_o = Q/A = 22.7/ [1.22 (2.44)] = 7.63$ m/s

$Fr = V_o / (9.81y_e)^{1/2} = 7.63/ [9.81(1.22)]^{1/2} = 2.21$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 0.55$ m; $D_{50}/y_e = 0.55/1.22 = 0.45$ (≥ 0.1 OK)

Two tailwater elevations are given; use the lowest to determine the basin size that will serve the tailwater range, that is, TW = 0.85 m.

$TW/y_e = 0.85/1.22 = 0.7$, which is less than 0.75. Therefore, from Equation 10.2, $C_o = 1.4$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.45)^{-0.55}(2.21) - 1.4 = 1.55$$

$h_S = (h_S /y_e)y_e = 1.55 (1.22) = 1.89$ m

$h_S/D_{50} = 1.89/0.55 = 3.4$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(1.89) = 18.9$ m

L_S min $= 3W_o = 3(2.44) = 7.3$ m, use $L_S = 18.9$ m

$L_B = 15h_S = 15(1.89) = 28.4$ m

L_B min $= 4W_o = 4(2.44) = 9.8$ m, use $L_B = 28.4$ m

$W_B = W_o + 2(L_B/3) = 2.44 + 2(28.4/3) = 21.4$ m

Step 4. Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$22.7^2/9.81 = 52.5 = [y_c(21.4 + 2y_c)]^3/ (21.4 + 4y_c)$

By trial and success, $y_c = 0.48$ m, $T_c = 23.3$ m, $A_c = 10.7\ m^2$

$V_B = V_c = Q/A_c = 22.7/10.7 = 2.1$ m/s (acceptable)

The initial trial of riprap ($D_{50} = 0.55$ m) results in a 28.4 m basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 0.75$ m; $D_{50}/y_e = 0.75/1.22 = 0.61$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.61)^{-0.55}(2.21) - 1.4 = 1.09$$

$h_S = (h_S/y_e)y_e = 1.09\ (1.22) = 1.34$ m

$h_S/D_{50} = 1.34/0.75 = 1.8$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than, 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 0.71$ m; $D_{50}/y_e = 0.71/1.22 = 0.58$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.58)^{-0.55}(2.21) - 1.4 = 1.16$$

$h_S = (h_S/y_e)y_e = 1.16\ (1.22) = 1.42$ m

$h_S/D_{50} = 1.42/0.71 = 2.0$ and $h_S/D_{50} \geq 2$ is satisfied.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(1.42) = 14.2$ m

L_S min $= 3W_o = 3(2.44) = 7.3$ m, use $L_S = 14.2$ m

$L_B = 15h_S = 15(1.42) = 21.3$ m

L_B min $= 4W_o = 4(2.44) = 9.8$ m, use $L_B = 21.3$ m

$W_B = W_o + 2(L_B/3) = 2.44 + 2(21.3/3) = 16.6$ m

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 0.75$ m) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$22.7^2/9.81 = 52.5 = [y_c(16.6 + 2y_c)]^3/ (16.6 + 4y_c)$

By trial and success, $y_c = 0.56$ m, $T_c = 18.8$ m, $A_c = 9.9\ m^2$

$V_B = V_c = Q/A_c = 22.7/9.9 = 2.3$ m/s (greater than 2.1 m/s; not acceptable). If the apron were extended (with a continued flare) such that the total basin length was 28.4 m, the velocity would be reduced to the allowable level.

Two feasible options have been identified. First, a 1.89 m deep, 18.9 m long pool, with a 9.5 m apron using $D_{50} = 0.55$ m. Second, a 1.42 m deep, 14.2 m long pool, with a 14.2 m apron using $D_{50} = 0.75$ m. Because the overall length is the same, the first option is likely to be more economical.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$.

For the first tailwater condition, $TW/y_o = 0.85/1.22 = 0.70$, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed downstream.

For the second tailwater condition, $TW/y_o = 1.28/1.22 = 1.05$, which does not satisfy $TW/y_o \leq 0.75$. To determine required riprap, estimate centerline velocity at a series of downstream cross sections using Figure 10.3.

Compute equivalent circular diameter, D_e, for brink area:

$A = \pi D_e^2 /4 = (y_o)(W_o) = (1.22)(2.44) = 3.00 \text{ m}^2$

$D_e = [3.00(4)/ \pi]^{1/2} = 1.95$ m

Rock size can be determined using the procedures in Section 10.3 (Equation 10.6) or other suitable method. The computations are summarized below.

L/D_e	L (m)	V_L/V_o (Figure 10.3)	V_L (m/s)	Rock size, D_{50} (m)
10	19.5	0.59	4.50	0.43
15	29.3	0.42	3.20	0.22
20	39.0	0.30	2.29	0.11
21	41.0	0.28	2.13	0.10

The calculations above continue until $V_L \leq V_{allow}$. Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 41.0 m downstream from the culvert brink, which is 12.6 m beyond the basin exit. Riprap should be installed in accordance with details shown in HEC 11.

<u>Design Example: Riprap Basin (Culvert on a Steep Slope) (CU)</u>

Determine riprap basin dimensions using the envelope design (Equations 10.1 and10.2) for an 8 ft by 6 ft reinforced concrete box (RCB) culvert that is in inlet control with supercritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 7 ft/s. Riprap is available with a D_{50} of 1.67, 1.83, and 2.5 ft. Consider two tailwater conditions: 1) TW = 2.8 ft and 2) TW = 4.2 ft. Given:

Q = 800 ft^3/s

y_o = 4 ft (normal flow depth) = brink depth

Solution

Step 1. Compute the culvert outlet velocity, V_o, depth, y_o, and Froude number for brink conditions. For supercritical flow (culvert on a steep slope), V_o will be V_n.

$y_o = y_e = 4$ ft

$V_o = Q/A = 800/ [4 (8)] = 25$ ft/s

$Fr = V_o / (32.2y_e)^{1/2} = 25/ [32.2(4)]^{1/2} = 2.2$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 1.83$ ft; $D_{50}/y_e = 1.83/4 = 0.46$ (≥ 0.1 OK)

Two tailwater elevations are given; use the lowest to determine the basin size that will serve the tailwater range, that is, TW = 2.8 ft.

$TW/y_e = 2.8/4 = 0.7$, which is less than 0.75. From Equation 10.2, $C_o = 1.4$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.46)^{-0.55}(2.2) - 1.4 = 1.50$$

$h_S = (h_S /y_e)y_e = 1.50 (4) = 6.0$ ft

$h_S/D_{50} = 6.0/1.83 = 3.3$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(6.0) = 60$ ft

L_S min $= 3W_o = 3(8) = 24$ ft, use $L_S = 60$ ft

$L_B = 15h_S = 15(6.0) = 90$ ft

L_B min $= 4W_o = 4(8) = 32$ ft, use $L_B = 90$ ft

$W_B = W_o + 2(L_B/3) = 8 + 2(90/3) = 68$ ft

Step 4. Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$800^2/32.2 = 19{,}876 = [y_c(68 + 2y_c)]^3/ (68 + 4y_c)$

By trial and success, $y_c = 1.60$ ft, $T_c = 74.4$ ft, $A_c = 113.9$ ft^2

$V_B = V_c = Q/A_c = 800/113.9 = 7.0$ ft/s (acceptable)

The initial trial of riprap (D_{50} = 1.83 ft) results in a 90 ft basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 2.5$ ft; $D_{50}/y_e = 2.5/4 = 0.63$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.63)^{-0.55}(2.2) - 1.4 = 1.04$$

$h_S = (h_S/y_e)y_e = 1.04\ (4) = 4.2$ ft

$h_S/D_{50} = 4.2/2.5 = 1.7$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than, 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 2.3$ ft; $D_{50}/y_e = 2.3/4 = 0.58$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.58)^{-0.55}(2.2) - 1.4 = 1.15$$

$h_S = (h_S/y_e)y_e = 1.15\ (4) = 4.6$ ft

$h_S/D_{50} = 4.6/2.3 = 2.0$ and $h_S/D_{50} \geq 2$ is satisfied.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(4.6) = 46$ ft

L_S min $= 3W_o = 3(8) = 24$ ft, use $L_S = 46$ ft

$L_B = 15h_S = 15(4.6) = 69$ ft

L_B min $= 4W_o = 4(8) = 32$ ft, use $L_B = 69$ ft

$W_B = W_o + 2(L_B/3) = 8 + 2(69/3) = 54$ ft

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 2.5$ ft) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/\ (W_B + 2zy_c)$

$800^2/32.2 = 19{,}876 = [y_c(54 + 2y_c)]^3/\ (54 + 4y_c)$

By trial and success, $y_c = 1.85$ ft, $T_c = 61.4$ ft, $A_c = 106.9$ ft^2

$V_B = V_c = Q/A_c = 800/106.9 = 7.5$ ft/s (not acceptable). If the apron were extended (with a continued flare) such that the total basin length was 90 ft, the velocity would be reduced to the allowable level.

Two feasible options have been identified. First, a 6-ft-deep, 60-ft-long pool, with a 30-ft-apron using $D_{50} = 1.83$ ft. Second, a 4.6-ft-deep, 46-ft-long pool, with a 44-ft-apron using $D_{50} = 2.5$ ft. Because the overall length is the same, the first option is likely to be more economical.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$.

For the first tailwater condition, $TW/y_o = 2.8/4.0 = 0.70$, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed downstream.

For the second tailwater condition, $TW/y_o = 4.2/4.0 = 1.05$, which does not satisfy $TW/y_o \leq 0.75$. To determine required riprap, estimate centerline velocity at a series of downstream cross sections using Figure 10.3.

Compute equivalent circular diameter, D_e, for brink area:

$A = \pi D_e^2 /4 = (y_o)(W_o) = (4)(8) = 32 \text{ ft}^2$

$D_e = [32(4)/ \pi]^{1/2} = 6.4$ ft

Rock size can be determined using the procedures in Section 10.3 (Equation 10.6) or other suitable method. The computations are summarized below.

L/D_e	L (ft)	V_L/V_o (Figure 10.3)	V_L (ft/s)	Rock size, D_{50} (ft)
10	64	0.59	14.7	1.42
15	96	0.42	10.5	0.72
20	128	0.30	7.5	0.37
21	135	0.28	7.0	0.32

The calculations above continue until $V_L \leq V_{allow}$. Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink, which is 45 ft beyond the basin exit. Riprap should be installed in accordance with details shown in HEC 11.

Design Example: Riprap Basin (Culvert on a Mild Slope) (SI)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and 10.2) for a pipe culvert that is in outlet control with subcritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 2.1 m/s. Riprap is available with a D_{50} of 0.125, 0.150, and 0.250 m. Given:

D = 1.83 m CMP with Manning's n = 0.024

S_o = 0.004 m/m

Q = 3.82 m^3/s

y_n = 1.37 m (normal flow depth in the pipe)

V_n = 1.80 m/s (normal velocity in the pipe)

TW = 0.61 m (tailwater depth)

Solution

Step 1. Compute the culvert outlet velocity, V_o, and depth, y_o.

For subcritical flow (culvert on mild slope), use Figure 3.4 to obtain y_o/D, then calculate V_o by dividing Q by the wetted area for y_o.

$K_u Q/D^{2.5} = 1.81 (3.82)/1.83^{2.5} = 1.53$

$TW/D = 0.61/1.83 = 0.33$

From Figure 3.4, $y_o/D = 0.45$

$y_o = (y_o/D)D = 0.45(1.83) = 0.823$ m (brink depth)

From Table B.2, for $y_o/D = 0.45$, the brink area ratio $A/D^2 = 0.343$

$A = (A/D^2)D^2 = 0.343(1.83)^2 = 1.15\ m^2$

$V_o = Q/A = 3.82/1.15 = 3.32$ m/s

$y_e = (A/2)^{1/2} = (1.15/2)^{1/2} = 0.76$ m

$Fr = V_o / [9.81(y_e)]^{1/2} = 3.32/ [9.81(0.76)]^{1/2} = 1.22$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 0.15$ m; $D_{50}/y_e = 0.15/0.76 = 0.20$ (≥ 0.1 OK)

$TW/y_e = 0.61/0.76 = 0.80$. Therefore, from Equation 10.2,

$C_o = 4.0(TW/y_e) - 1.6 = 4.0(0.80) - 1.6 = 1.61$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.20)^{-0.55}(1.22) - 1.61 = 0.933$$

$h_S = (h_S/y_e)y_e = 0.933\ (0.76) = 0.71$ m

$h_S/D_{50} = 0.71/0.15 = 4.7$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(0.71) = 7.1$ m

L_S min $= 3W_o = 3(1.83) = 5.5$ m, use $L_S = 7.1$ m

$L_B = 15h_S = 15(0.71) = 10.7$ m

L_B min $= 4W_o = 4(1.83) = 7.3$ m, use $L_B = 10.7$ m

$W_B = W_o + 2(L_B/3) = 1.83 + 2(10.7/3) = 9.0$ m

Step 4. Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$3.82^2/9.81 = 1.49 = [y_c(9.0 + 2y_c)]^3/ (9.0 + 4y_c)$

By trial and success, $y_c = 0.26$ m, $T_c = 10.0$ m, $A_c = 2.48\ m^2$

$V_c = Q/A_c = 3.82/2.48 = 1.5$ m/s (acceptable)

The initial trial of riprap ($D_{50} = 0.15$ m) results in a 10.7 m basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.25$ m; $D_{50}/y_e = 0.25/0.76 = 0.33$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.33)^{-0.55}(1.22) - 1.61 = 0.320$$

$h_S = (h_S / y_e) y_e = 0.320\ (0.76) = 0.24$ m

$h_S/D_{50} = 0.24/0.25 = 0.96$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.205$ m; $D_{50}/y_e = 0.205/0.76 = 0.27$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.27)^{-0.55}(1.22) - 1.61 = 0.545$$

$h_S = (h_S / y_e) y_e = 0.545\ (0.76) = 0.41$ m

$h_S/D_{50} = 0.41/0.205 = 2.0$ and $h_S/D_{50} \geq 2$ is satisfied. Continue to step 3.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(0.41) = 4.1$ m

L_S min $= 3W_o = 3(1.83) = 5.5$ m, use $L_S = 5.5$ m

$L_B = 15h_S = 15(0.41) = 6.2$ m

L_B min $= 4W_o = 4(1.83) = 7.3$ m, use $L_B = 7.3$ m

$W_B = W_o + 2(L_B/3) = 1.83 + 2(7.3/3) = 6.7$ m

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 0.25$ m) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/\ (W_B + 2zy_c)$

$3.82^2/9.81 = 1.49 = [y_c(6.7 + 2y_c)]^3/\ (6.7 + 4y_c)$

By trial and success, $y_c = 0.31$ m, $T_c = 7.94$ m, $A_c = 2.28$ m^2

$V_c = Q/A_c = 3.82/2.28 = 1.7$ m/s (acceptable)

Two feasible options have been identified. First, a 0.71 m deep, 7.1 m long pool, with an 3.6 m apron using $D_{50} = 0.15$ m. Second, a 0.41 m deep, 5.5 m long pool, with a 1.8 m apron using $D_{50} = 0.25$ m. The choice between these two options will likely depend on the available space and the cost of riprap.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$

$TW/y_o = 0.61/0.823 = 0.74$, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed.

Design Example: Riprap Basin (Culvert on a Mild Slope) (CU)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and 10.2) for a pipe culvert that is in outlet control with subcritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 7.0 ft/s. Riprap is available with a D_{50} of 0.42, 0.50, and 0.83 ft. Given:

D	=	6 ft CMP with Manning's n = 0.024
S_o	=	0.004 ft/ft
Q	=	135 ft^3/s
y_n	=	4.5 ft (normal flow depth in the pipe)
V_n	=	5.9 ft/s (normal velocity in the pipe)
TW	=	2.0 ft (tailwater depth)

Solution

Step 1. Compute the culvert outlet velocity, V_o, depth, y_o and Froude number.

For subcritical flow (culvert on mild slope), use Figure 3.4 to obtain y_o/D, then calculate V_o by dividing Q by the wetted area for y_o.

$K_uQ/D^{2.5} = 1.0(135)/6^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

From Figure 3.4, $y_o/D = 0.45$

$y_o = (y_o/D)D = 0.45(6) = 2.7$ ft (brink depth)

From Table B.2 for $y_o/D = 0.45$, the brink area ratio $A/D^2 = 0.343$

$A = (A/D^2)D^2 = 0.343(6)^2 = 12.35\ ft^2$

$V_o = Q/A = 135/12.35 = 10.9$ ft/s

$y_e = (A/2)^{1/2} = (12.35/2)^{1/2} = 2.48$ ft

$Fr = V_o / [32.2(y_e)]^{1/2} = 10.9/ [32.2(2.48)]^{1/2} = 1.22$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 0.5$ ft; $D_{50}/y_e = 0.5/2.48 = 0.20$ (≥ 0.1 OK)

$TW/y_e = 2.0/2.48 = 0.806$. Therefore, from Equation 10.2,

$C_o = 4.0(TW/y_e) -1.6 = 4.0(0.806) -1.6 = 1.62$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.20)^{-0.55}(1.22) - 1.62 = 0.923$$

$h_S = (h_S/y_e)y_e = 0.923\ (2.48) = 2.3$ ft

$h_S/D_{50} = 2.3/0.5 = 4.6$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(2.3) = 23$ ft

L_S min $= 3W_o = 3(6) = 18$ ft, use $L_S = 23$ ft

$L_B = 15h_S = 15(2.3) = 34.5$ ft

L_B min $= 4W_o = 4(6) = 24$ ft, use $L_B = 34.5$ ft

$W_B = W_o + 2(L_B/3) = 6 + 2(34.5/3) = 29$ ft

Step 4. Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$135^2/32.2 = 566 = [y_c(29 + 2y_c)]^3/ (29 + 4y_c)$

By trial and success, $y_c = 0.86$ ft, $T_c = 32.4$ ft, $A_c = 26.4$ ft^2

$V_c = Q/A_c = 135/26.4 = 5.1$ ft/s (acceptable)

The initial trial of riprap ($D_{50} = 0.5$ ft) results in a 34.5 ft basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.83$ ft; $D_{50}/y_e = 0.83/2.48 = 0.33$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.33)^{-0.55}(1.22) - 1.62 = 0.311$$

$h_S = (h_S/y_e)y_e = 0.311\ (2.48) = 0.8$ ft

$h_S/D_{50} = 0.8/0.83 = 0.96$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.65$ ft; $D_{50}/y_e = 0.65/2.48 = 0.26$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.26)^{-0.55}(1.22) - 1.62 = 0.581$$

$h_S = (h_S/y_e)y_e = 0.581\ (2.48) = 1.4$ ft

$h_S/D_{50} = 1.4/0.65 = 2.15$ and $h_S/D_{50} \geq 2$ is satisfied. Continue to step 3.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(1.4) = 14$ ft

L_S min $= 3W_o = 3(6) = 18$ ft, use $L_S = 18$ ft

$L_B = 15h_S = 15(1.4) = 21$ ft

L_B min = $4W_o$ = 4(6) = 24 ft, use L_B = 24 ft

$W_B = W_o + 2(L_B/3) = 6 + 2(24/3) = 22$ ft

However, since the trial D_{50} is not available, the next larger riprap size (D_{50} = 0.83 ft) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$135^2/32.2 = 566 = [y_c(22 + 2y_c)]^3/ (22 + 4y_c)$

By trial and success, y_c = 1.02 ft, T_c =26.1 ft, A_c = 24.5 ft^2

$V_c = Q/A_c$ = 135/24.5 = 5.5 ft/s (acceptable)

Two feasible options have been identified. First, a 2.3-ft-deep, 23-ft-long pool, with an 11.5-ft-apron using D_{50} = 0.5 ft. Second, a 1.4-ft-deep, 18-ft-long pool, with a 6-ft-apron using D_{50} = 0.83 ft. The choice between these two options will likely depend on the available space and the cost of riprap.

Step 5. For the design discharge, determine if $TW/y_o \le 0.75$

TW/y_o = 2.0/2.7 = 0.74, which satisfies $TW/y_o \le 0.75$. No additional riprap needed.

10.2 RIPRAP APRON

The most commonly used device for outlet protection, primarily for culverts 1500 mm (60 in) or smaller, is a riprap apron. An example schematic of an apron taken from the Federal Lands Division of the Federal Highway Administration is shown in Figure 10.4.

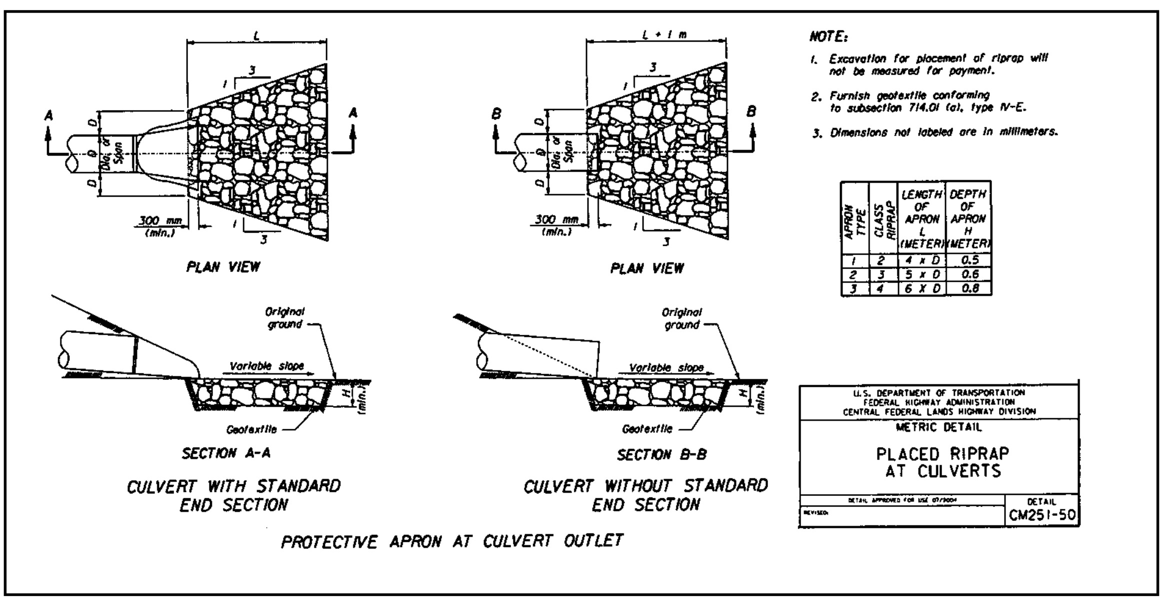


Figure 10.4. Placed Riprap at Culverts (Central Federal Lands Highway Division)

They are constructed of riprap or grouted riprap at a zero grade for a distance that is often related to the outlet pipe diameter. These aprons do not dissipate significant energy except through increased roughness for a short distance. However, they do serve to spread the flow helping to transition to the natural drainage way or to sheet flow where no natural drainage way exists. However, if they are too short, or otherwise ineffective, they simply move the location of potential erosion downstream. The key design elements of the riprap apron are the riprap size as well as the length, width, and depth of the apron.

Several relationships have been proposed for riprap sizing for culvert aprons and several of these are discussed in greater detail in Appendix D. The independent variables in these relationships include one or more of the following variables: outlet velocity, rock specific gravity, pipe dimension (e.g. diameter), outlet Froude number, and tailwater. The following equation (Fletcher and Grace, 1972) is recommended for circular culverts:

$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) \tag{10.4}$$

where,

D_{50}	=	riprap size, m (ft)
Q	=	design discharge, m^3/s (ft^3/s)
D	=	culvert diameter (circular), m (ft)
TW	=	tailwater depth, m (ft)
g	=	acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

Tailwater depth for Equation 10.4 should be limited to between 0.4D and 1.0D. If tailwater is unknown, use 0.4D.

Whenever the flow is supercritical in the culvert, the culvert diameter is adjusted as follows:

$$D' = \frac{D + y_n}{2} \tag{10.5}$$

where,

D'	=	adjusted culvert rise, m (ft)
y_n	=	normal (supercritical) depth in the culvert, m (ft)

Equation 10.4 assumes that the rock specific gravity is 2.65. If the actual specific gravity differs significantly from this value, the D_{50} should be adjusted inversely to specific gravity.

The designer should calculate D_{50} using Equation 10.4 and compare with available riprap classes. A project or design standard can be developed such as the example from the Federal Highway Administration Federal Lands Highway Division (FHWA, 2003) shown in Table 10.1 (first two columns). The class of riprap to be specified is that which has a D_{50} greater than or equal to the required size. For projects with several riprap aprons, it is often cost effective to use fewer riprap classes to simplify acquiring and installing the riprap at multiple locations. In such a case, the designer must evaluate the tradeoffs between over sizing riprap at some locations in order to reduce the number of classes required on a project.

Table 10.1. Example Riprap Classes and Apron Dimensions

Class	D_{50} (mm)	D_{50} (in)	Apron Length[1]	Apron Depth
1	125	5	4D	$3.5D_{50}$
2	150	6	4D	$3.3D_{50}$
3	250	10	5D	$2.4D_{50}$
4	350	14	6D	$2.2D_{50}$
5	500	20	7D	$2.0D_{50}$
6	550	22	8D	$2.0D_{50}$

[1]D is the culvert rise.

The apron dimensions must also be specified. Table 10.1 provides guidance on the apron length and depth. Apron length is given as a function of the culvert rise and the riprap size. Apron depth ranges from $3.5D_{50}$ for the smallest riprap to a limit of $2.0D_{50}$ for the larger riprap sizes. The final dimension, width, may be determined using the 1:3 flare shown in Figure 10.4 and should conform to the dimensions of the downstream channel. A filter blanket should also be provided as described in HEC 11 (Brown and Clyde, 1989).

For tailwater conditions above the acceptable range for Equation 10.4 (TW > 1.0D), Figure 10.3 should be used to determine the velocity downstream of the culvert. The guidance in Section 10.3 may be used for sizing the riprap. The apron length is determined based on the allowable velocity and the location at which it occurs based on Figure 10.3.

Over their service life, riprap aprons experience a wide variety of flow and tailwater conditions. In addition, the relations summarized in Table 10.1 do not fully account for the many variables in culvert design. To ensure continued satisfactory operation, maintenance personnel should inspect them after major flood events. If repeated severe damage occurs, the location may be a candidate for extending the apron or another type of energy dissipator.

Design Example: Riprap Apron (SI)

Design a riprap apron for the following CMP installation. Available riprap classes are provided in Table 10.1. Given:

$Q = 2.33\ m^3/s$

$D = 1.5\ m$

$TW = 0.5\ m$

Solution

Step 1. Calculate D_{50} from Equation 10.4. First verify that tailwater is within range.

TW/D = 0.5/1.5 = 0.33. This is less than 0.4D, therefore,

use TW = 0.4D = 0.4(1.5) = 0.6 m

$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) = 0.2\,(1.5)\left(\frac{2.33}{\sqrt{9.81}(1.5)^{2.5}}\right)^{4/3}\left(\frac{1.5}{0.6}\right) = 0.13\ m$$

Step 2. Determine riprap class. From Table 10.1, riprap class 2 (D_{50} = 0.15 m) is required.

Step 3. Estimate apron dimensions.

From Table 10.1 for riprap class 2,

Length, L = 4D = 4(1.5) = 6 m

Depth = $3.3D_{50}$ = 3.3 (0.15) = 0.50 m

Width (at apron end) = 3D + (2/3)L = 3(1.5) + (2/3)(6) = 8.5 m

Design Example: Riprap Apron (CU)

Design a riprap apron for the following CMP installation. Available riprap classes are provided in Table 10.1. Given:

Q = 85 ft^3/s

D = 5.0 ft

TW = 1.6 ft

Solution

Step 1. Calculate D_{50} from Equation 10.4. First verify that tailwater is within range.

TW/D = 1.6/5.0 = 0.32. This is less than 0.4D, therefore,

use TW = 0.4D = 0.4(5) = 2.0 ft

$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) = 0.2\,(5.0)\left(\frac{85}{\sqrt{32.2}(5.0)^{2.5}}\right)^{4/3}\left(\frac{5.0}{2.0}\right) = 0.43\text{ ft} = 5.2\text{ in}$$

Step 2. Determine riprap class. From Table 10.1, riprap class 2 (D_{50} = 6 in) is required.

Step 3. Estimate apron dimensions.

From Table 10.1 for riprap class 2,

Length, L = 4D = 4(5) = 20 ft

Depth = $3.3D_{50}$ = 3.3 (6) = 19.8 in = 1.65 ft

Width (at apron end) = 3D + (2/3)L = 3(5) + (2/3)(20) = 28.3 ft

10.3 RIPRAP APRONS AFTER ENERGY DISSIPATORS

Some energy dissipators provide exit conditions, velocity and depth, near critical. This flow condition rapidly adjusts to the downstream or natural channel regime; however, critical velocity may be sufficient to cause erosion problems requiring protection adjacent to the energy dissipator. Equation 10.6 provides the riprap size recommended for use downstream of energy dissipators. This relationship is from Searcy (1967) and is the same equation used in HEC 11 (Brown and Clyde, 1989) for riprap protection around bridge piers.

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) \tag{10.6}$$

where,

D_{50} = median rock size, m (ft)

V = velocity at the exit of the dissipator, m/s (ft/s)

S = riprap specific gravity

The length of protection can be judged based on the magnitude of the exit velocity compared with the natural channel velocity. The greater this difference, the longer will be the length required for the exit flow to adjust to the natural channel condition. A filter blanket should also be provided as described in HEC 11 (Brown and Clyde, 1989).

STORM SEWER DESIGN SHEET

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands
Date Prepared: 9/6/2022
Date Revised:
Input By: Brendan Rundle
Reviewed By: Ben Sweet/Sam Bahia
Drawing Reference: 121153-GPO AND 121153-STM

Legend:
PROJECT SPECIFIC INFO
USER DESIGN INPUT
CUMILATIVE CELL
CALCULATED DESIGN CELL OUTPUT
USER AS-BUILT INPUT

LOCATION			DEMAND: AREA								DEMAND: FLOW									CAPACITY: PROPOSED SEWER PIPE SIZING / DESIGN								
STREET	FROM MH	TO MH	AREA ID	HIGH DENSITY	ROAD 1	REAR YARD 1	REAR YARD 2	PARK	TOTAL AREA	WEIGHTED RUNOFF COEFFICIENT	INDIVI 2.78 AR	ACCUM 2.78 AR	TIME OF CONC	RAIN INTENSITY (mm/hr) 2yr	5yr	100yr	PEAK FLOW	TOTAL UNCONTROLLED PEAK FLOW (QDesign)	TOTAL RESTRICTED PEAK FLOW (Q)	PIPE PROPERTIES: LENGTH	SIZE / MATERIAL	ID ACTUAL	ROUGHNESS	DESIGN GRADE	FULL FLOW CAPACITY	FULL FLOW VELOCITY	TIME OF FLOW	QPEAK DESIGN / QFULL
				0.85	0.70	0.50	0.45	0.20	(ha)				(min.)				(L/s)	(L/s)	(L/s)	(m)	(mm / type)	(m)		(%)	(L/s)	(m/s)	(min.)	(%)
Street 1	2	4	C13		0.29				0.29	0.70	0.56	0.56	10.00	76.81			43.34	43.3		82.8	250 PVC	0.254	0.013	1.30	70.7	1.40	0.99	61.3%
									0.00		0.00	0.00	10.00				0.00											
									0.00		0.00	0.00	10.00				0.00											
Street 1	4	6	-						0.00		0.00	0.56	10.99	73.21			41.31	41.3		45.7	250 PVC	0.254	0.013	1.30	70.7	1.40	0.55	58.4%
									0.00		0.00	0.00	10.99				0.00											
									0.00		0.00	0.00	10.99				0.00											
Street 1	6	8	C12		0.33				0.33	0.70	0.64	1.21	11.53	71.38			86.12	86.1		29.9	300 PVC	0.305	0.013	1.50	123.6	1.69	0.29	69.7%
									0.00		0.00	0.00	11.53				0.00											
									0.00		0.00	0.00	11.53				0.00											
Street 1	8	10	-						0.00		0.00	1.21	11.83	70.43			84.98	85.0		19.0	375 PVC	0.381	0.013	1.50	224.0	1.96	0.16	37.9%
									0.00		0.00	0.00	11.83				0.00											
									0.00		0.00	0.00	11.83				0.00											
Street 1	10	12	C10		0.22				0.22	0.70	0.43	1.63	11.99	69.93			114.30	114.3		18.1	450 PVC	0.457	0.013	1.50	364.3	2.22	0.14	31.4%
									0.00		0.00	0.00	11.99				0.00											
									0.00		0.00	0.00	11.99				0.00											
Street 2	14	16	C01		0.28				0.28	0.70	0.54	0.54	10.00	76.81			41.85	41.8		51.9	300 PVC	0.305	0.013	0.50	71.3	0.98	0.88	58.7%
									0.00		0.00	0.00	10.00				0.00											
									0.00		0.00	0.00	10.00				0.00											
Street 2	16	18	C02		0.33				0.33	0.70	0.64	1.19	10.88	73.57			87.33	87.3		30.4	375 PVC	0.381	0.013	0.50	129.3	1.13	0.45	67.5%
									0.00		0.00	0.00	10.88				0.00											
									0.00		0.00	0.00	10.88				0.00											
Street 2	18	20	C03		0.28				0.28	0.70	0.54	1.73	11.33	72.04			124.78	124.8		51.8	525 CONC	0.533	0.013	0.50	317.2	1.42	0.61	39.3%
									0.00		0.00	0.00	11.33				0.00											
									0.00		0.00	0.00	11.33				0.00											
Street 2	20	22	C04		0.31				0.31	0.70	0.60	2.34	11.94	70.08			163.66	163.7		54.5	600 CONC	0.610	0.013	0.50	452.9	1.55	0.59	36.1%
									0.00		0.00	0.00	11.94				0.00											
									0.00		0.00	0.00	11.94				0.00											
Street 2	24	48	C06		0.34				0.34	0.70	0.66	0.66	10.00	76.81			50.82	50.8		63.8	450 PVC	0.457	0.013	0.25	148.7	0.91	1.17	34.2%
									0.00		0.00	0.00	10.00				0.00											
									0.00		0.00	0.00	10.00				0.00											
Street 2	48	22	C05		0.24				0.24	0.70	0.47	1.13	11.17	72.57			81.91	81.9		28.7	450 PVC	0.457	0.013	0.25	148.7	0.91	0.53	55.1%
									0.00		0.00	0.00	11.17				0.00											
									0.00		0.00	0.00	11.17				0.00											
Street 3	22	26	-						0.00		0.00	3.46	12.52	68.30			236.59	236.6		30.3	600 CONC	0.610	0.013	0.25	320.3	1.10	0.46	73.9%
									0.00		0.00	0.00	12.52				0.00											
									0.00		0.00	0.00	12.52				0.00											
Street 3	26	12	C11			0.65			0.65	0.50	0.90	4.37	12.99	66.97			292.49	292.5		68.4	675 CONC	0.686	0.013	0.25	438.5	1.19	0.96	66.7%
									0.00		0.00	0.00	12.99				0.00											
									0.00		0.00	0.00	12.99				0.00											
Street 3	12	28	C08,C14		0.27	0.56			0.83	0.57	1.30	7.31	13.95	64.37			470.31	470.3		69.6	750 CONC	0.762	0.013	0.25	580.7	1.27	0.91	81.0%
									0.00		0.00	0.00	13.95				0.00											
									0.00		0.00	0.00	13.95				0.00											
Street 3	28	30	C09		0.23				0.23	0.70	0.45	7.75	15.22	61.26			474.94	474.9		12.8	750 CONC	0.762	0.013	0.25	580.7	1.27	0.17	81.8%
									0.00		0.00	0.00	15.22				0.00											
									0.00		0.00	0.00	15.22				0.00											
Street 3	42	40	C20		0.21				0.21	0.70	0.41	0.41	10.00	76.81			31.39	31.4		75.6	250 PVC	0.254	0.013	4.50	131.6	2.60	0.49	23.8%
									0.00		0.00	0.00	10.00				0.00											
									0.00		0.00	0.00	10.00				0.00											
Street 3	40	38	C18,C19		0.17	0.40			0.57	0.56	0.89	1.30	10.49	74.99			97.15	97.1		66.9	300 PVC	0.305	0.013	2.50	159.5	2.19	0.51	60.9%
									0.00		0.00	0.00	10.49				0.00											
									0.00		0.00	0.00	10.49				0.00											
Street 3	38	32	C17		0.32				0.32	0.70	0.62	1.92	11.00	73.18			140.38	140.4		54.1	525 CONC	0.533	0.013	0.25	224.3	1.00	0.90	62.6%
									0.00		0.00	0.00	11.00				0.00											
									0.00		0.00	0.00	11.00				0.00											
Street 3	36	34	C16		0.35				0.35	0.70	0.68	0.68	10.00	76.81			52.31	52.3		90.7	375 PVC	0.381	0.013	0.32	103.5	0.91	1.67	50.6%
									0.00		0.00	0.00	10.00				0.00											
									0.00		0.00	0.00	10.00				0.00											
Street 3	34	32	C15		0.34				0.34	0.70	0.66	1.34	11.67	70.95			95.27	95.3		100.5	450 PVC	0.457	0.013	0.32	168.3	1.02	1.63	56.6%
									0.00		0.00	0.00	11.67				0.00											
									0.00		0.00	0.00	11.67				0.00											
Street 3	32	30	-						0.00		0.00	3.26	13.30	66.09			215.53	215.5		11.1	600 CONC	0.610	0.013	0.25	320.3	1.10	0.17	67.3%
									0.00		0.00	0.00	13.30				0.00											
									0.00		0.00	0.00	13.30				0.00											
Easement Block	30	44	-						0.00		0.00	11.01	15.39	60.87			670.45	670.4		14.3	1200 CONC	1.219	0.013	0.25	2033.7	1.74	0.14	33.0%
									0.00		0.00	0.00	15.39				0.00											
									0.00		0.00	0.00	15.39				0.00											
Easement Block	44	46	C07,C21			0.63			0.63	0.50	0.88	11.89	15.52	60.56			720.06	720.1		39.6	1200 CONC	1.219	0.013	0.25	2033.7	1.74	0.38	35.4%
									0.00		0.00	0.00	15.52				0.00											
									0.00		0.00	0.00	15.52				0.00											

DEMAND EQUATION
Q = 2.78 AIR

Where : Q = Peak flow in litres per second (L/s)
A = Area in hectares (ha)
R = Weighted runoff coefficient (increased by 25% for 100-year)
I = Rainfall intensity in millimeters per hour (mm/hr)
Rainfall Intensity (I) is based on City of Ottawa IDF data presented in the City of Ottawa Sewer Design Guidelines (Oct. 2012)

CAPACITY EQUATION
Q full= (1/n) A R^(2/3)So^(1/2)

Where : Q full = Capacity (L/s)
n = Manning coefficient of roughness (0.013)
A = Flow area (m^2)
R = Wetter perimenter (m)
So = Pipe Slope/gradient

NOTE(S)
Highlighted sewer sections represent future design considerations that are not applicable to this MECP ECA application.

Stinson Lands
Pre-Development Model Parameters

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Time to Peak Calculations
(Uplands Overland Flow Method)
Existing Conditions

Area ID	Area (ha)	Overland Flow: Length (m)	Overland Flow: Elevation U/S (m)	Overland Flow: Elevation D/S (m)	Overland Flow: Slope (%)	Overland Flow: Velocity (m/s)	Overland Flow: Travel Time (min)	Concentrated Overland Flow: Length (m)	Concentrated Overland Flow: Elevation U/S (m)	Concentrated Overland Flow: Elevation D/S (m)	Concentrated Overland Flow: Slope (%)	Concentrated Overland Flow: Velocity (m/s)	Concentrated Overland Flow: Travel Time (min)	Overall: Time of Concentration (min)	Overall: Time to Peak (min)	Overall: Time to Peak (min)	Overall: Time to Peak (hrs)	Flow Length	Slope
A1	2.717	100	94	89	5.0%	0.33	5.05	150	89	88	0.5%	0.19	13.16	18	12	12	0.20	250	2%
A2	0.444	40	88	88	0.7%	0.14	4.76	0	0	0	0.0%	0	0.00	5	3	10	0.17	40	1%
B1	1.101	80	88	85	4.1%	0.3	4.44	0	0	0	0.0%	0	0.00	4	3	10	0.17	80	4%
C1	2.298	100	88	86	2.0%	0.21	7.94	25	86	86	2.0%	0.4	1.04	9	6	10	0.17	125	2%
D1	1.273	100	94	89	5.0%	0.33	5.05	70	89	86	4.3%	0.57	2.05	7	5	10	0.17	170	5%
TOTAL:	**7.83**																		

Weighted Curve Number Calculations
Soil type Silty Clay = D

Area ID	Land Use 1	Area	CN	Land Use 2	Area	CN	Land Use 3	Area	CN	Weighted CN
A1	Building & Road	4%	86	Tree Farm	1%	82	Row Crops	95%	89	89
A2	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
B1	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
C1	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
D1	Building & Road	12%	86	Tree Farm	28%	82	Row Crops	60%	89	87

Weighted IA Calculations

Area ID	Land Use 1	Area	S	IA	Land Use 2	Area	S	IA	Land Use 3	Area	S	IA	Weighted IA
A1	Building & Roads	4%	41.35	6.20	Tree Farm	1%	55.76	8.36	Row Crops	95%	31.39	6.28	6.32
A2	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
B1	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
C1	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
D1	Building & Roads	28%	41.35	6.20	Tree Farm	12%	55.76	8.36	Row Crops	60%	31.39	6.28	6.51

Project Name
Pre-Development Model Schematic

NOVATECH
Engineers, Planners & Landscape Architects

8/2/2022
PREPARED BY: NOVATECH

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Stinson Lands
Post-Development Model Parameters

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Area ID	Catchment Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.200	0.70	71%	0%	38.18	52.38	1.0%
B-01	0.740	0.45	36%	100%	21.26	348.14	1.0%
C-01	0.280	0.70	71%	45%	23.78	117.77	1.0%
C-02	0.330	0.70	71%	45%	24.82	132.96	1.0%
C-03	0.280	0.70	71%	45%	23.62	118.54	1.0%
C-04	0.310	0.70	71%	45%	22.83	135.79	1.0%
C-05	0.240	0.70	71%	45%	23.31	102.96	1.0%
C-06	0.340	0.70	71%	45%	34.45	98.70	1.0%
C-07	0.590	0.45	36%	100%	39.87	148.00	1.0%
C-08	0.270	0.70	71%	45%	22.14	121.97	1.0%
C-09	0.230	0.70	71%	45%	22.42	102.59	1.0%
C-10	0.220	0.70	71%	45%	22.45	98.00	1.0%
C-11	0.650	0.45	36%	100%	19.05	341.19	1.0%
C-12	0.330	0.70	71%	45%	19.77	166.94	1.0%
C-13	0.290	0.70	71%	45%	23.66	122.59	1.0%
C-14	0.560	0.45	36%	100%	14.10	397.05	1.0%
C-15	0.340	0.70	71%	45%	22.21	153.06	1.0%
C-16	0.350	0.70	71%	45%	21.78	160.71	1.0%
C-17	0.320	0.70	71%	45%	22.95	139.43	1.0%
C-18	0.400	0.45	36%	100%	13.81	289.75	1.0%
C-19	0.170	0.70	71%	45%	19.18	88.63	1.0%
C-20	0.210	0.70	71%	45%	18.84	111.49	1.0%
C-21	0.040	0.45	36%	100%	45.28	8.83	1.0%
D-01	0.180	0.20	0%	0%	20.51	87.76	1.0%
TOTAL:	**7.87**						
		0.59	56%				

Project Name
Overall Model Schematic

NOVATECH
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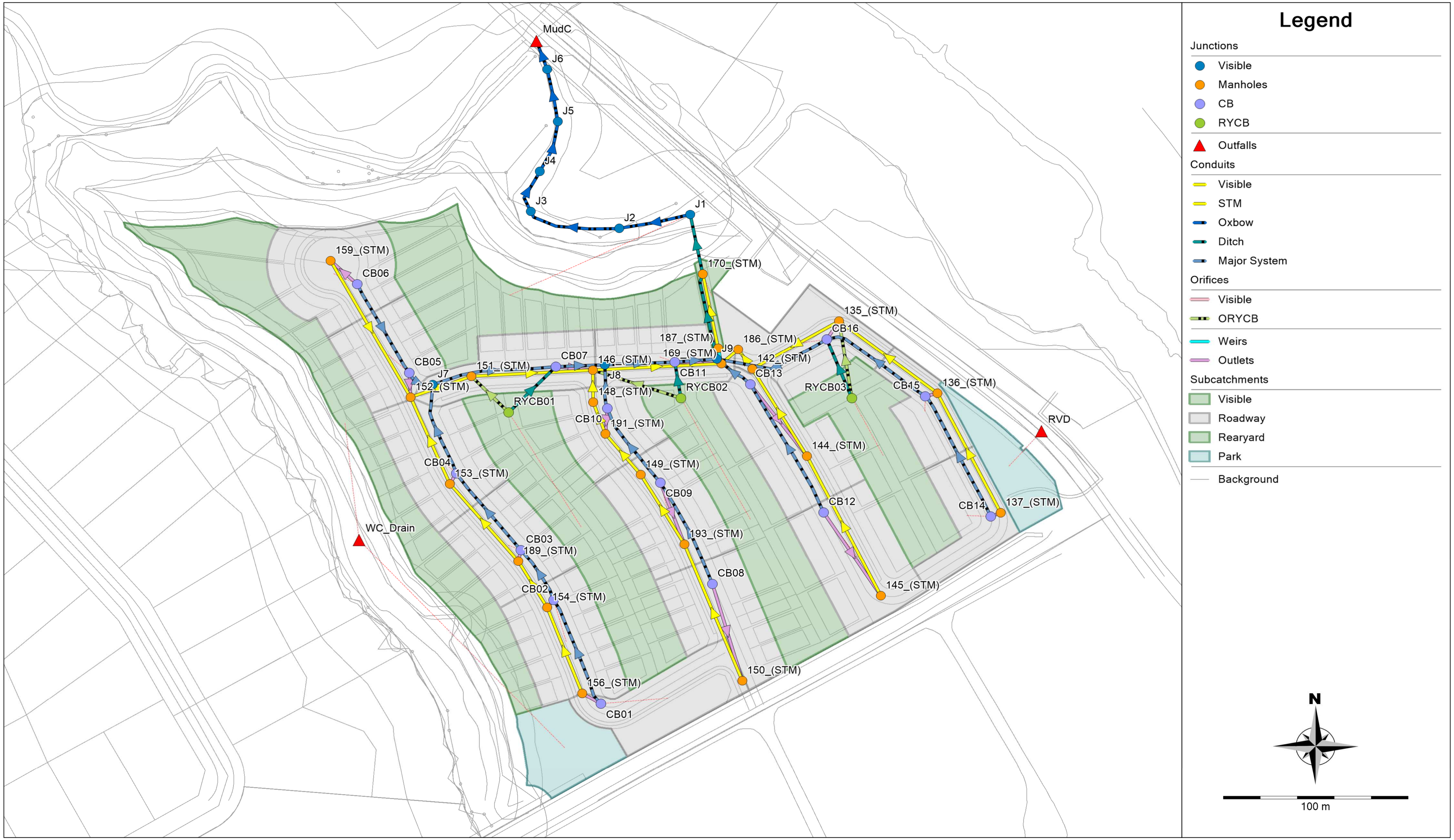

8/2/2022
PREPARED BY: NOVATECH

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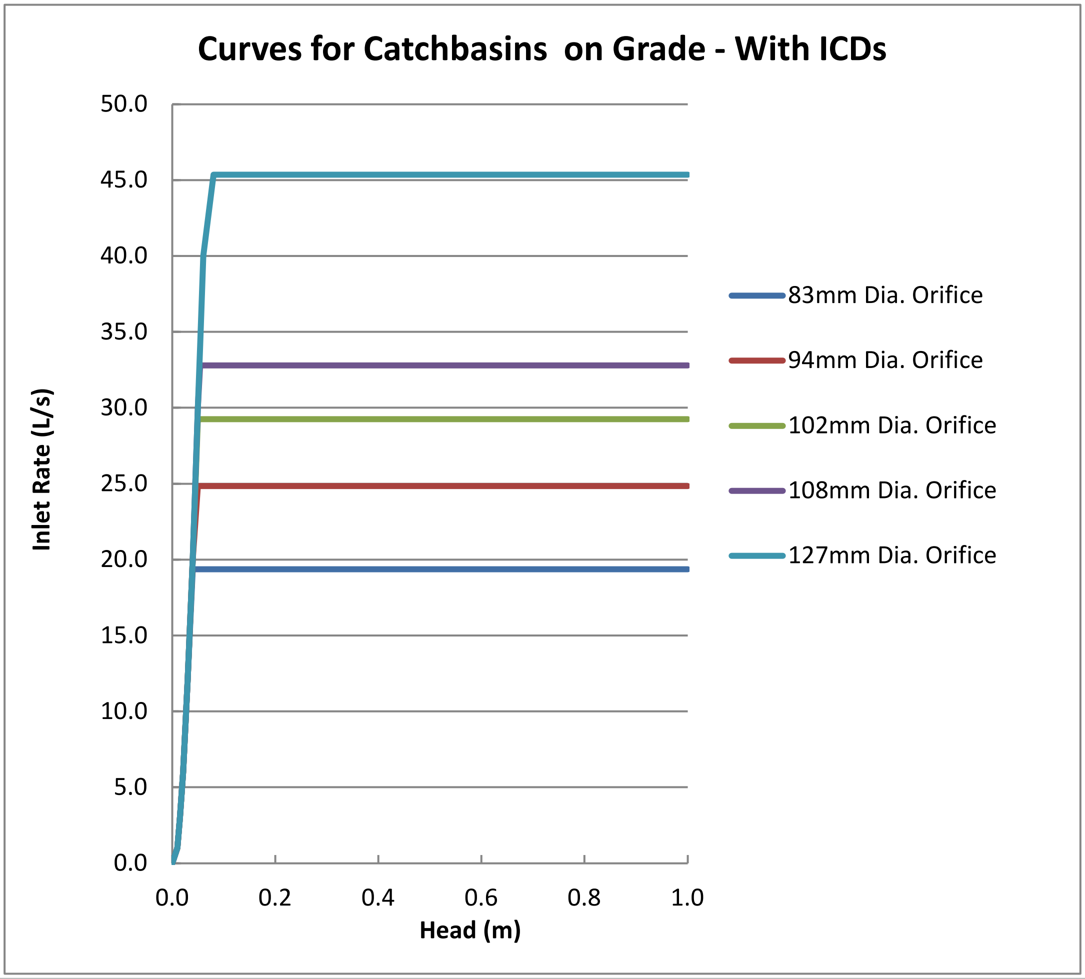


Curb Inlet Catchbasins on Continuous Grade

Depth vs. Captured Flow Curve

A standard depth vs. captured flow curve for catch basins on a continuous grade was provided to Novatech by City staff for use in a dual-drainage model of an existing residential neighbourhood. This standard curve was derived using the inlet curves in Appendix 7A of the Ottawa Sewer Design Guidelines.

Novatech reviewed the methodology used to create this standard curve (described below) and determined that it was suitable for general use in other dual-drainage models.

- MTO Design Chart 4.04 provides the relationship between the gutter flow rate (Q_t) and flow spread (T) for Barrier Curb.
- MTO Design Chart 4.12 provides the relationship between flow spread (T) and flow depth (D).
- The relationship between the gutter flow rate (Q_t) and flow depth (D) was determined for different road slopes using the above charts and Manning's equation (refer to pages 58-60 of the MTO Drainage Management Manual – Part 2);
- The relationship between approach flow (Q_t) and captured flow (Q_c) was determined for different road slopes using the design chart for Barrier Curb with Gutter (Appendix 7-A.2).
- Using the above information, a family of curves was developed to characterize the relationship between flow depth and captured flow for curb inlet catchbasins on different road slopes. The results of this exercise can be summarized as follows:
    - For a given flow depth, the gutter flow rate (Q_t) increases as the road slope increases.
    - The capture efficiency (Q_c) of curb inlet catchbasins decrease as the road slope increases.
    - The net result is that the relationship between flow depth and capture rate is largely independent of road slope: While approach flow vs. captured flow (Q_t vs. Q_c) varies significantly with road grade, flow depth vs. captured flow (D vs. Q_c) does not.

Since there was very little difference in the flow depth vs. captured flow curves for different road slopes, this family of curves was averaged to create a single standard curve for use in dual-drainage models.

Inlet Control Devices

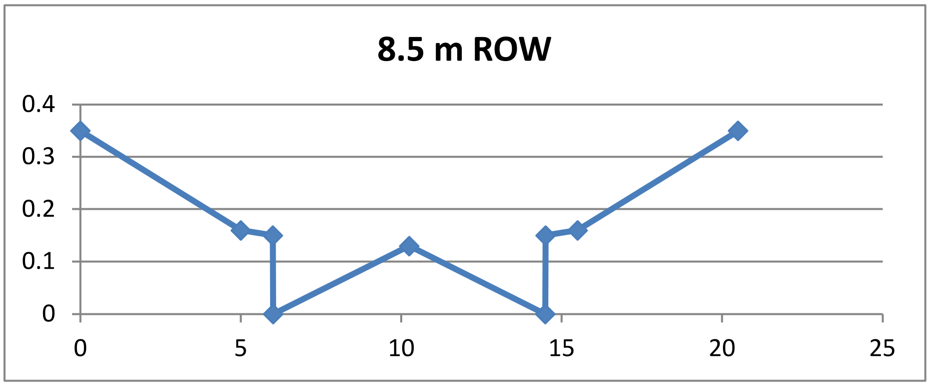
The standard depth vs. capture flow curve was modified to account for the installation of ICDs in curb inlet catchbasins on continuous grade. Separate inlet curves were created for each standard ICD orifice size by capping the inlet rate on the depth vs. capture flow curve at the maximum flow rate through the ICD at a head of 1.2m (depth from centerline of CB lead to top of CICB frame).

Stinson Lands
HGL Elevations

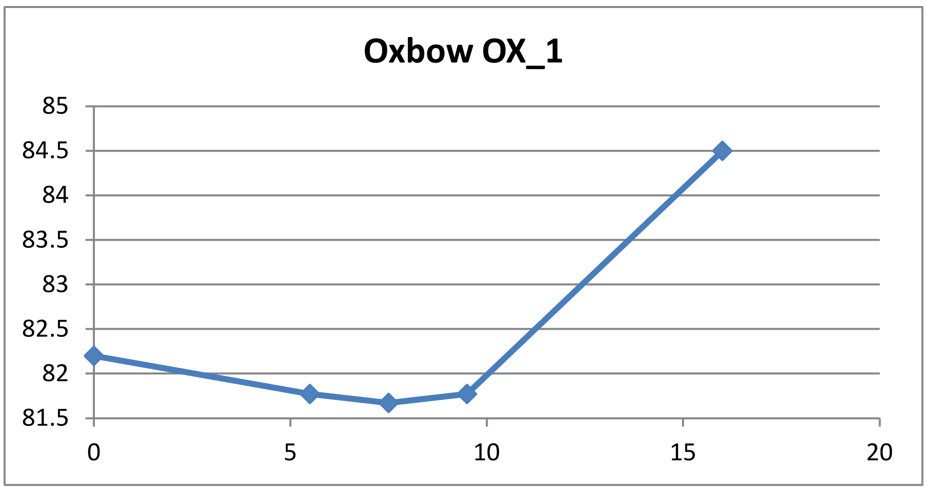
NOVATECH
Engineers, Planners & Landscape Architects

Manhole ID	MH Invert Elevation (m)	T/G Elevation (m)	Outlet pipe invert (m)	Outlet Pipe Diameter (m)	Outlet Pipe Obvert (m)	HGL Elevation (Chicago) (m)	WL Above Obvert (Chicago) (m)
135_(STM)	84.54	87.82	84.84	0.53	85.37	85.41	0.04
136_(STM)	86.44	89.77	86.74	0.30	87.04	86.90	-0.14
137_(STM)	89.89	93.22	90.19	0.25	90.44	90.29	-0.15
142_(STM)	84.33	87.92	84.63	0.60	85.23	85.24	0.01
144_(STM)	86.49	90.32	86.79	0.45	87.24	86.95	-0.29
145_(STM)	89.65	93.28	89.95	0.38	90.33	90.06	-0.27
146_(STM)	84.38	87.82	84.68	0.75	85.43	85.68	0.25
148_(STM)	85.03	87.85	85.33	0.38	85.71	85.83	0.13
149_(STM)	85.87	88.62	86.17	0.30	86.47	86.45	-0.02
150_(STM)	87.62	90.70	87.92	0.25	88.17	88.08	-0.09
151_(STM)	84.65	87.96	84.95	0.68	85.63	85.84	0.22
152_(STM)	84.81	88.20	85.11	0.60	85.71	85.97	0.26
153_(STM)	85.23	88.52	85.53	0.45	85.98	86.11	0.13
154_(STM)	85.74	89.18	86.04	0.38	86.42	86.33	-0.09
156_(STM)	86.08	89.56	86.38	0.30	86.68	86.57	-0.11
159_(STM)	85.27	88.31	85.57	0.45	86.02	86.02	0.00
169_(STM)	84.04	87.65	84.48	0.75	85.23	85.01	-0.22
170_(STM)	83.50	86.83	83.80	1.20	85.00	83.91	-1.09
186_(STM)	83.67	88.05	83.97	1.20	85.17	84.76	-0.41
187_(STM)	83.90	86.96	83.90	1.20	85.10	84.53	-0.57
189_(STM)	85.51	88.96	85.81	0.45	86.26	86.19	-0.07
191_(STM)	85.40	88.13	85.70	0.30	86.00	86.06	0.06
193_(STM)	86.51	89.34	86.81	0.25	87.06	86.97	-0.09

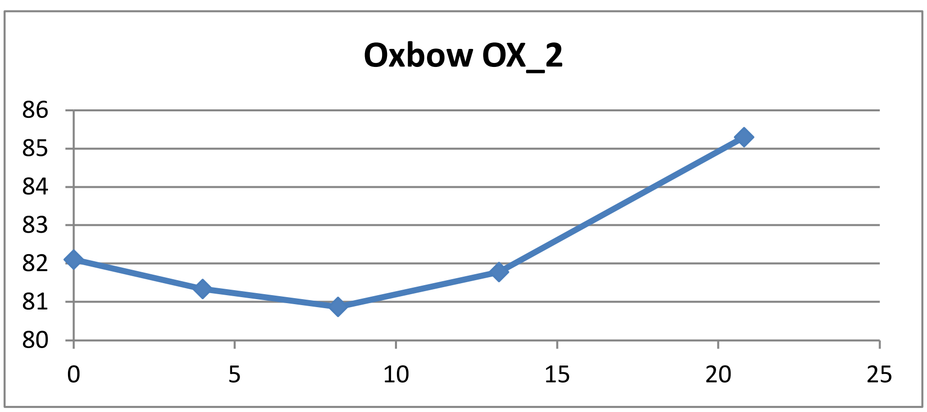
Local ROW	
0	0.35
5	0.16
6	0.15
6.01	0
10.25	0.13
14.49	0
14.5	0.15
15.5	0.16
20.5	0.35



Oxbow OX_1	
0	82.2
5.5	81.77
7.5	81.67
9.5	81.77
16	84.5



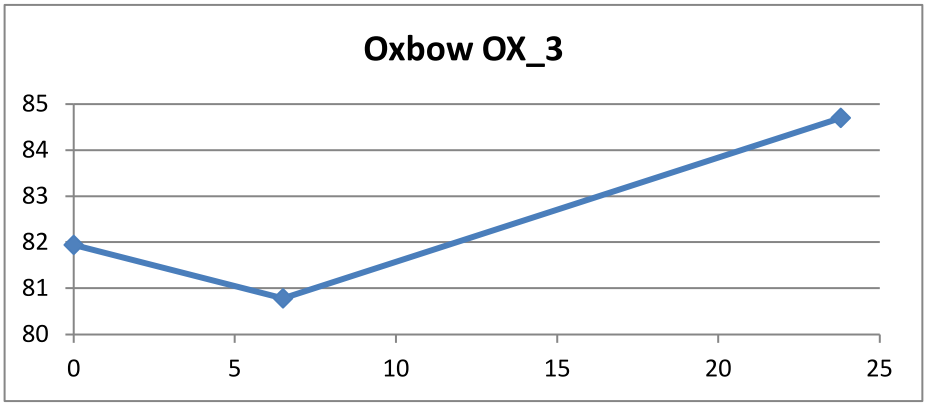
Oxbow OX_2	
0	82.1
4	81.34
8.2	80.87
13.2	81.78
20.8	85.3



Stinson Lands
Cross-Sections

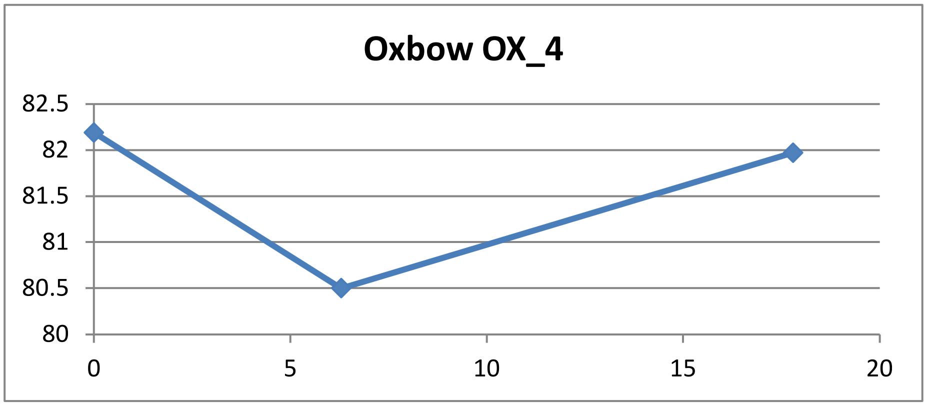
Oxbow OX_3

0	81.94
6.5	80.78
23.8	84.7



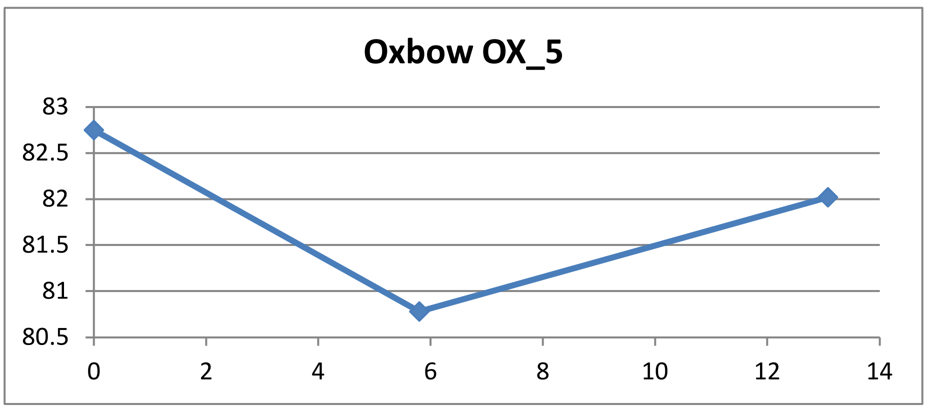
Oxbow OX_4

0	82.19
6.3	80.5
17.8	81.97



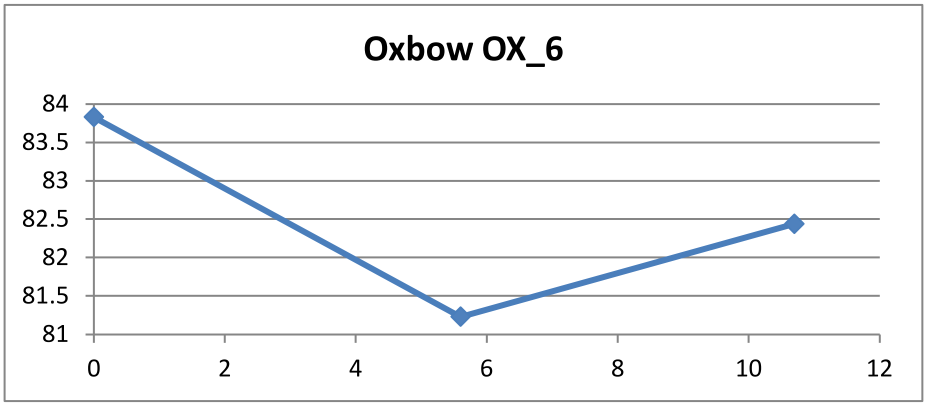
Oxbow OX_5

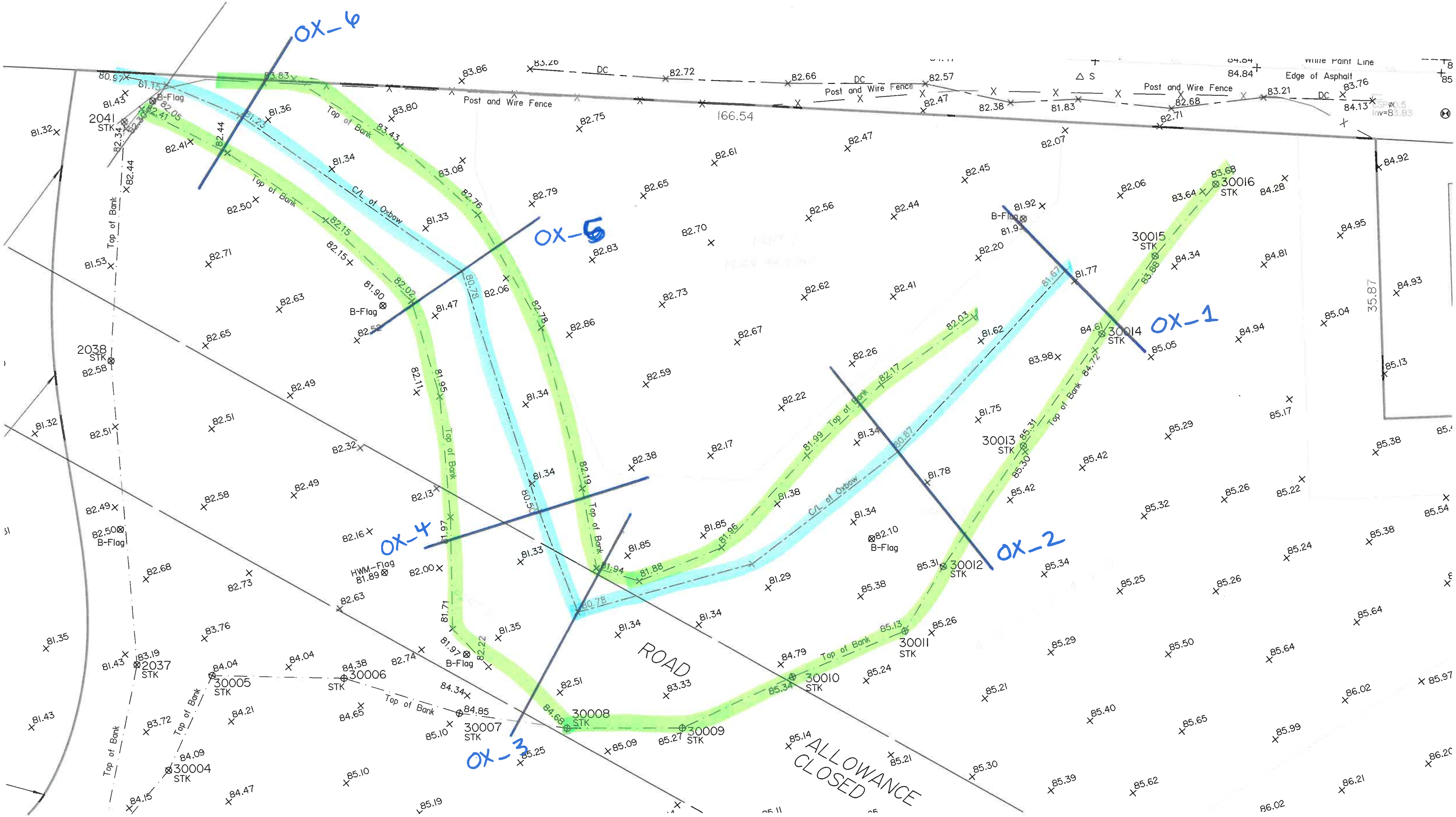
0	82.75
5.8	80.78
13.08	82.02



Oxbow OX_6

0	83.83
5.6	81.23
10.7	82.44





oxbow ditch x-sections for PCSWMM.

Stinson Lands
Design Storm Time Series Data
6-hour Chicago Design Storms

C25mm-6.stm		C2-6.stm		C5-6.stm	
Duration	Intensity	Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr	min	mm/hr
0:00	0	0:00	0	0:00	0
0:10	0.9292336	0:10	1.37	0:10	1.78
0:20	1.0106263	0:20	1.49	0:20	1.94
0:30	1.1055844	0:30	1.63	0:30	2.13
0:40	1.2344563	0:40	1.82	0:40	2.37
0:50	1.390459	0:50	2.05	0:50	2.68
1:00	1.6075062	1:00	2.37	1:00	3.1
1:10	1.9059462	1:10	2.81	1:10	3.68
1:20	2.3739543	1:20	3.5	1:20	4.58
1:30	3.1810988	1:30	4.69	1:30	6.15
1:40	4.9513905	1:40	7.3	1:40	9.61
1:50	12.351345	1:50	18.21	1:50	24.17
2:00	52.098123	2:00	76.81	2:00	104.19
2:10	16.332806	2:10	24.08	2:10	32.04
2:20	8.3834501	2:20	12.36	2:20	16.34
2:30	5.6432286	2:30	8.32	2:30	10.96
2:40	4.2731178	2:40	6.3	2:40	8.29
2:50	3.4524079	2:50	5.09	2:50	6.69
3:00	2.9097897	3:00	4.29	3:00	5.63
3:10	2.5231743	3:10	3.72	3:10	4.87
3:20	2.2315171	3:20	3.29	3:20	4.3
3:30	2.0009044	3:30	2.95	3:30	3.86
3:40	1.8177707	3:40	2.68	3:40	3.51
3:50	1.6685508	3:50	2.46	3:50	3.22
4:00	1.5464617	4:00	2.28	4:00	2.98
4:10	1.4379381	4:10	2.12	4:10	2.77
4:20	1.3497626	4:20	1.99	4:20	2.6
4:30	1.2683699	4:30	1.87	4:30	2.44
4:40	1.2005426	4:40	1.77	4:40	2.31
4:50	1.1394981	4:50	1.68	4:50	2.19
5:00	1.0852363	5:00	1.6	5:00	2.08
5:10	1.0309745	5:10	1.52	5:10	1.99
5:20	0.9902781	5:20	1.46	5:20	1.9
5:30	0.9495817	5:30	1.4	5:30	1.82
5:40	0.9088854	5:40	1.34	5:40	1.75
5:50	0.8749717	5:50	1.29	5:50	1.68
6:00	0.8410581	6:00	1.24	6:00	1.62

Stinson Lands
Design Storm Time Series Data
6-hour Chicago Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

C100-6.stm		C100-6+20%.stm	
Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr
0:00	0.00	0:00	0.00
0:10	2.90	0:10	3.48
0:50	3.16	0:50	3.79
1:30	3.48	1:30	4.18
2:10	3.88	2:10	4.66
2:50	4.39	2:50	5.27
3:30	5.07	3:30	6.08
4:10	6.05	4:10	7.26
4:50	7.54	4:50	9.05
5:30	10.16	5:30	12.19
6:10	15.97	6:10	19.16
6:50	40.65	6:50	48.78
7:30	178.56	7:30	214.27
8:10	54.05	8:10	64.86
8:50	27.32	8:50	32.78
9:30	18.24	9:30	21.89
10:10	13.74	10:10	16.49
10:50	11.06	10:50	13.27
11:30	9.29	11:30	11.15
12:10	8.02	12:10	9.62
12:50	7.08	12:50	8.50
13:30	6.35	13:30	7.62
14:10	5.76	14:10	6.91
14:50	5.28	14:50	6.34
15:30	4.88	15:30	5.86
16:10	4.54	16:10	5.45
16:50	4.25	16:50	5.10
17:30	3.99	17:30	4.79
18:10	3.77	18:10	4.52
18:50	3.57	18:50	4.28
19:30	3.40	19:30	4.08
20:10	3.24	20:10	3.89
20:50	3.10	20:50	3.72
21:30	2.97	21:30	3.56
22:10	2.85	22:10	3.42
22:50	2.74	22:50	3.29
23:30	2.64	23:30	3.17

Stinson Lands
Design Storm Time Series Data
SCS Design Storms

S2-12.stm			S5-12.stm			S100-12.stm	
Duration	Intensity		Duration	Intensity		Duration	Intensity
min	mm/hr		min	mm/hr		min	mm/hr
0:00	0.00		0:00	0		0:00	0
0:30	1.27		0:30	1.69		0:30	2.82
1:00	0.59		1:00	0.79		1:00	1.31
1:30	1.10		1:30	1.46		1:30	2.44
2:00	1.10		2:00	1.46		2:00	2.44
2:30	1.44		2:30	1.91		2:30	3.19
3:00	1.27		3:00	1.69		3:00	2.82
3:30	1.69		3:30	2.25		3:30	3.76
4:00	1.69		4:00	2.25		4:00	3.76
4:30	2.29		4:30	3.03		4:30	5.07
5:00	2.88		5:00	3.82		5:00	6.39
5:30	4.57		5:30	6.07		5:30	10.14
6:00	36.24		6:00	48.08		6:00	80.38
6:30	9.23		6:30	12.25		6:30	20.47
7:00	4.06		7:00	5.39		7:00	9.01
7:30	2.71		7:30	3.59		7:30	6.01
8:00	2.37		8:00	3.15		8:00	5.26
8:30	1.86		8:30	2.47		8:30	4.13
9:00	1.95		9:00	2.58		9:00	4.32
9:30	1.27		9:30	1.69		9:30	2.82
10:00	1.02		10:00	1.35		10:00	2.25
10:30	1.44		10:30	1.91		10:30	3.19
11:00	0.93		11:00	1.24		11:00	2.07
11:30	0.85		11:30	1.12		11:30	1.88
12:00	0.85		12:00	1.12		12:00	1.88

Stinson Lands
Design Storm Time Series Data
SCS Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

S2-24.stm		S5-24.stm		S100-24.stm	
Duration	Intensity	Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr	min	mm/hr
0:00	0.00	0:00	0	0:00	0
1:00	0.72	1:00	0.44	1:00	0.6
2:00	0.34	2:00	0.44	2:00	0.75
3:00	0.63	3:00	0.81	3:00	1.39
4:00	0.63	4:00	0.81	4:00	1.39
5:00	0.81	5:00	1.06	5:00	1.81
6:00	0.72	6:00	0.94	6:00	1.6
7:00	0.96	7:00	1.25	7:00	2.13
8:00	0.96	8:00	1.25	8:00	2.13
9:00	1.30	9:00	1.68	9:00	2.88
10:00	1.63	10:00	2.12	10:00	3.63
11:00	2.59	11:00	3.37	11:00	5.76
12:00	20.55	12:00	26.71	12:00	45.69
13:00	5.23	13:00	6.8	13:00	11.64
14:00	2.30	14:00	2.99	14:00	5.12
15:00	1.54	15:00	2	15:00	3.42
16:00	1.34	16:00	1.75	16:00	2.99
17:00	1.06	17:00	1.37	17:00	2.35
18:00	1.11	18:00	1.44	18:00	2.46
19:00	0.72	19:00	0.94	19:00	1.6
20:00	0.58	20:00	0.75	20:00	1.28
21:00	0.81	21:00	1.06	21:00	1.81
22:00	0.53	22:00	0.68	22:00	1.17
23:00	0.48	23:00	0.63	23:00	1.07
0:00	0.48	0:00	0.63	0:00	1.07

VORTECHS SYSTEM® ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS
STINSON SUBDIVISION (4386 RIDEAU VALLEY DRIVE)
OTTAWA, ON
MODEL PC1421 OFF-LINE

Design Ratio[1] = $\frac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(14.3 \text{ m2})}$ **= 0.79**

Rainfall Intensity mm/hr	Operating Rate[2] % of capacity	Flow Treated (l/s)	% Total Rainfall Volume[3]	Rmvl. Effcy[4] (%)	Rel. Effcy (%)
0.5	0.6	5.8	9.2%	100.0%	9.2%
1.0	1.2	11.5	10.6%	98.0%	10.4%
1.5	1.7	17.3	9.9%	98.0%	9.7%
2.0	2.3	23.0	8.4%	98.0%	8.2%
2.5	2.9	28.8	7.7%	98.0%	7.5%
3.0	3.5	34.5	5.9%	98.0%	5.8%
3.5	4.1	40.3	4.4%	98.0%	4.3%
4.0	4.6	46.0	4.7%	98.0%	4.6%
4.5	5.2	51.8	3.3%	98.0%	3.3%
5.0	5.8	57.6	3.0%	98.0%	3.0%
6.0	7.0	69.1	5.4%	98.0%	5.3%
7.0	8.1	80.6	4.4%	96.9%	4.2%
8.0	9.3	92.1	3.5%	96.3%	3.4%
9.0	10.5	103.6	2.8%	96.0%	2.7%
10.0	11.6	115.1	2.2%	95.3%	2.1%
15.0	17.4	172.7	7.0%	89.9%	6.3%
20.0	23.2	230.2	4.5%	85.7%	3.9%
25.0	29.0	287.8	1.4%	82.6%	1.2%
30.0	34.8	345.4	0.7%	80.0%	0.5%
35.0	40.6	402.9	0.5%	76.0%	0.4%
40.0	46.5	460.5	0.5%	69.0%	0.4%
					96.2%

Predicted Annual Runoff Volume Treated =	93.5%
Assumed Removal Efficiency of remaining % =	0.0%
Removal Efficiency Adjustment[5] =	6.5%
Predicted Net Annual Load Removal Efficiency =	**90%**

1 - Design Ratio = (Total Drainage Area) x (Runoff Coefficient) x (Rational Method Conversion) / Grit Chamber Area
- The Total Drainage Area and Runoff Coefficient are specified by the site engineer.
- The rational method conversion based on the units in the above equation is 2.775.

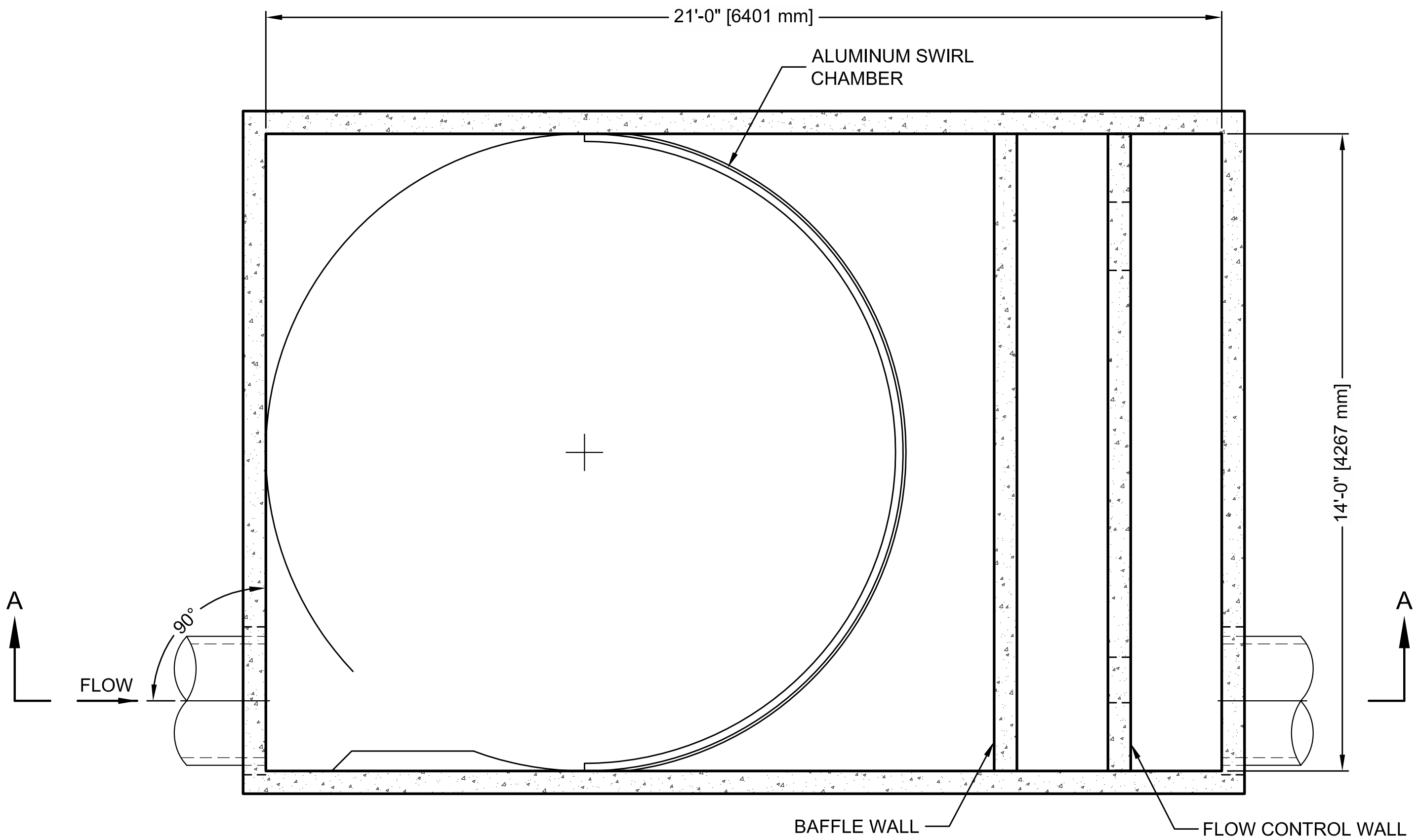
2 - Operating Rate (% of capacity) = percentage of peak operating rate of 68 l/s/m^2.
3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON
4 - Based on Contech Construction Products laboratory verified removal of an average particle size of 80 microns (see Technical Bulletin #1).
5- Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.

Calculated by:	JAK	7/26	Checked by:

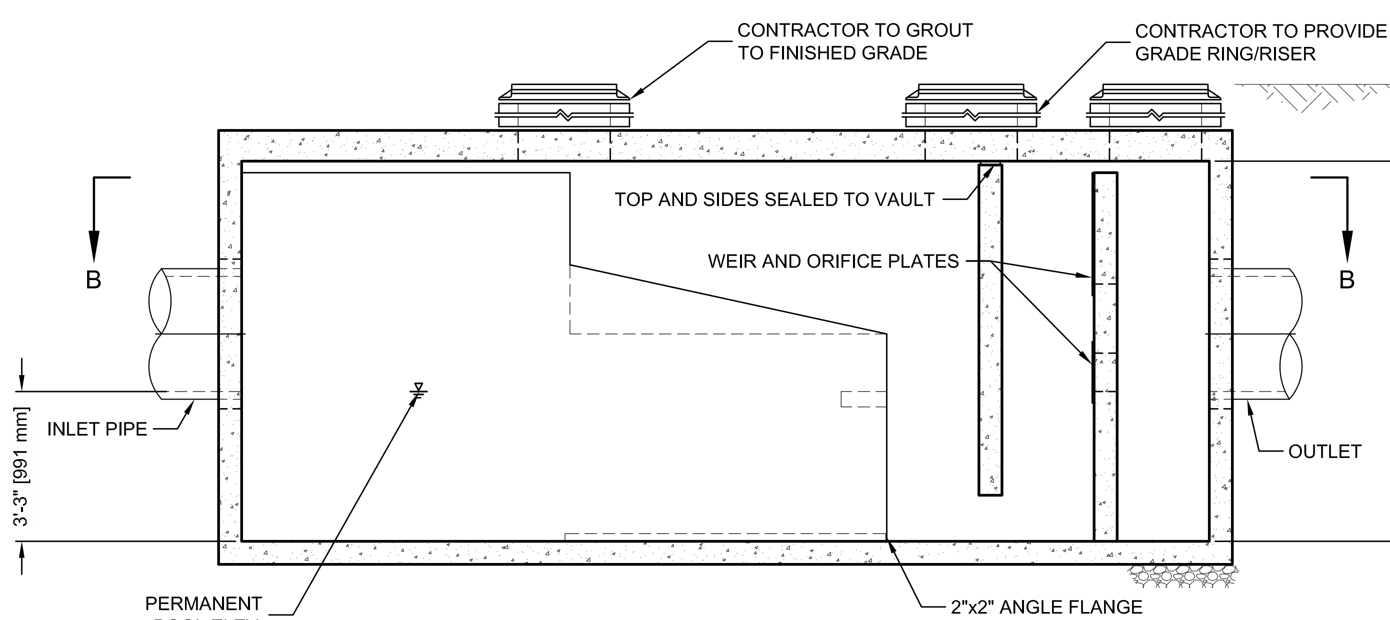
VORTECHS PC1421 DESIGN NOTES

VORTECHS PC1421 RATED TREATMENT CAPACITY IS 34 CFS, OR PER LOCAL REGULATIONS. IF THE SITE CONDITIONS EXCEED RATED TREATMENT CAPACITY, AN UPSTREAM BYPASS STRUCTURE IS REQUIRED.

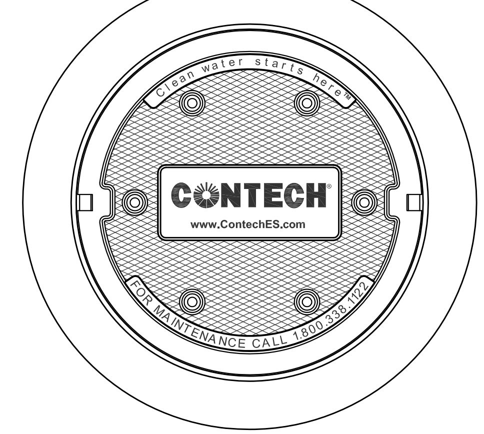
THE STANDARD INLET/OUTLET CONFIGURATION IS SHOWN. FOR OTHER CONFIGURATION OPTIONS , PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS, LLC REPRESENTATIVE. www.ContechES.com



SECTION B-B



SECTION A-A



FRAME AND COVER
(DIAMETER VARIES)
N.T.S.

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID			*
WATER QUALITY FLOW RATE (CFS)			*
PEAK FLOW RATE (CFS)			*
RETURN PERIOD OF PEAK FLOW (YRS)			*
PIPE DATA:	I.E.	MATERIAL	DIAMETER
INLET PIPE 1	*	*	*
INLET PIPE 2	*	*	*
OUTLET PIPE	*	*	*
RIM ELEVATION			*
ANTI-FLOTATION BALLAST		WIDTH	HEIGHT
		*	*
NOTES/SPECIAL REQUIREMENTS:			

* PER ENGINEER OF RECORD

GENERAL NOTES
1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHT, PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS, LLC REPRESENTATIVE. www.ContechES.com
4. VORTECHS WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.
5. STRUCTURE SHALL MEET AASHTO HS20 AND CASTINGS SHALL MEET AASHTO M306 LOAD RATING, ASSUMING GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION.
6. INLET PIPE(S) MUST BE PERPEDICULAR TO THE VAULT AND AT THE CORNER TO INTRODUCE THE FLOW TANGENTIALLY TO THE SWIRL CHAMBER. DUAL INLETS NOT TO HAVE OPPOSING TANGENTIAL FLOW DIRECTIONS.
7. OUTLET PIPE(S) MUST BE DOWN STREAM OF THE FLOW CONTROL BAFFLE AND MAY BE LOCATED ON THE SIDE OR END OF THE VAULT. THE FLOW CONTROL WALL MAY BE TURNED TO ACCOMODATE OUTLET PIPE KNOCKOUTS ON THE SIDE OF THE VAULT.

INSTALLATION NOTES
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE VORTECHS STRUCTURE (LIFTING CLUTCHES PROVIDED).
C. CONTRACTOR TO INSTALL JOINT SEALANT BETWEEN ALL STRUCTURE SECTIONS AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES. MATCH PIPE INVERTS WITH ELEVATIONS SHOWN.
E. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO ASSURE UNIT IS WATER TIGHT, HOLDING WATER TO FLOWLINE INVERT MINIMUM. IT IS SUGGESTED THAT ALL JOINTS BELOW PIPE INVERTS ARE GROUTED.

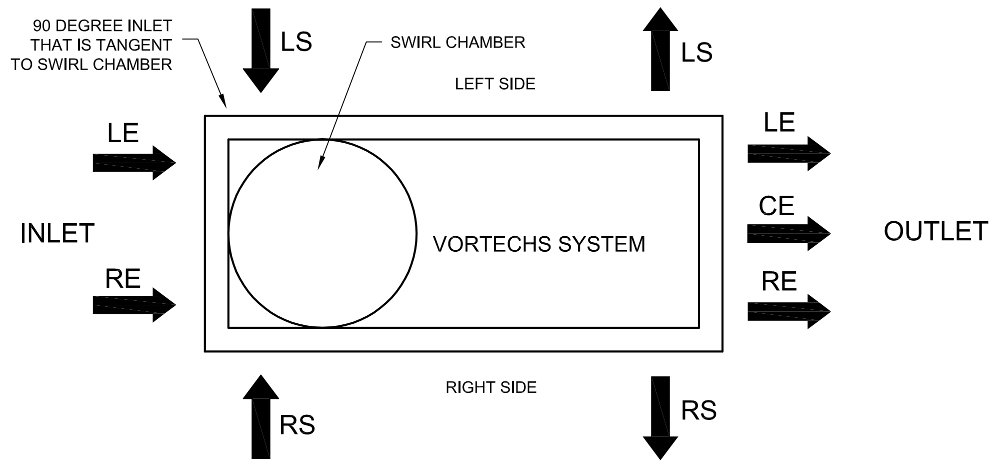
Vortechs®

THIS PRODUCT MAY BE PROTECTED BY THE FOLLOWING U.S. PATENT: 5,759,415; RELATED FOREIGN PATENTS.

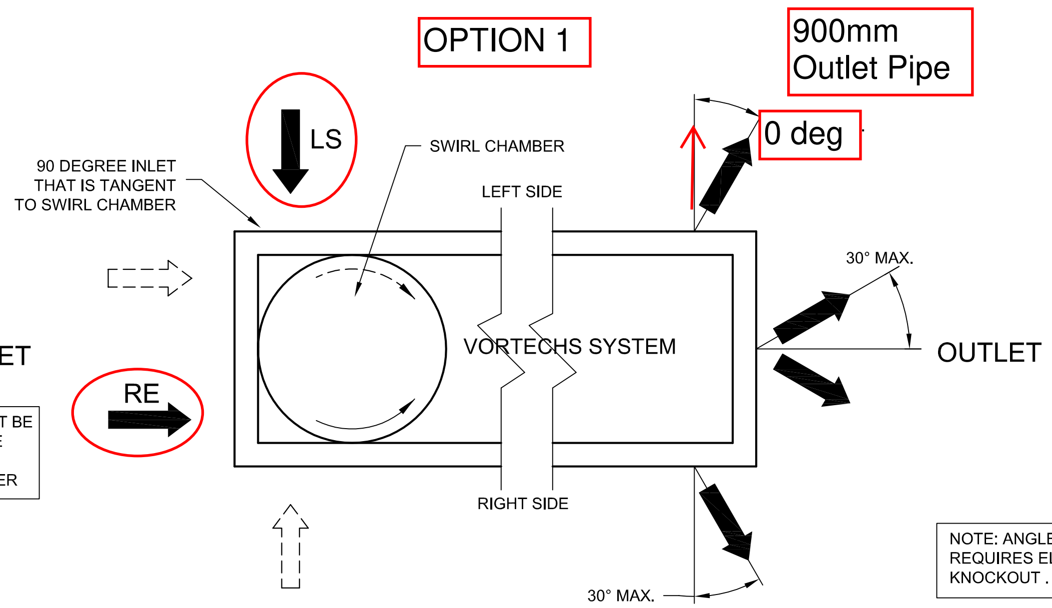
CONTECH®
ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

VORTECHS PC1421
STANDARD DETAIL

NOTE: INLET PIPE MUST BE PERPENDICULAR TO WALL IT IS ENTERING ON



TYPICAL INLET/OUTLET ORIENTATION OPTIONS



NOTE: DUAL INLET MUST BE POSITIONED TO INDUCE SAME DIRECTION OF FLOW IN SWIRL CHAMBER

NOTE: ANGLED OUTLET REQUIRES ELLIPTICAL KNOCKOUT .

DUAL INLET ORIENTATION OPTIONS

ANGLED OUTLET ORIENTATION OPTIONS

ORIENTATION KEY

LE = LEFT END
RE = RIGHT END
LS = LEFT SIDE
RS = RIGHT SIDE
CE = CENTER END

This CADD file is for the purpose of specifying stormwater treatment equipment to be furnished by CONTECH Stormwater Solutions and may only be transferred to other documents exactly as provided by CONTECH Stormwater Solutions. Title block information, **excluding** the CONTECH Stormwater Solutions logo and the Vortechs Stormwater Treatment System designation and patent number, may be deleted if necessary. Revisions to any part of this CADD file without prior coordination with CONTECH Stormwater Solutions shall be considered unauthorized use of proprietary information.

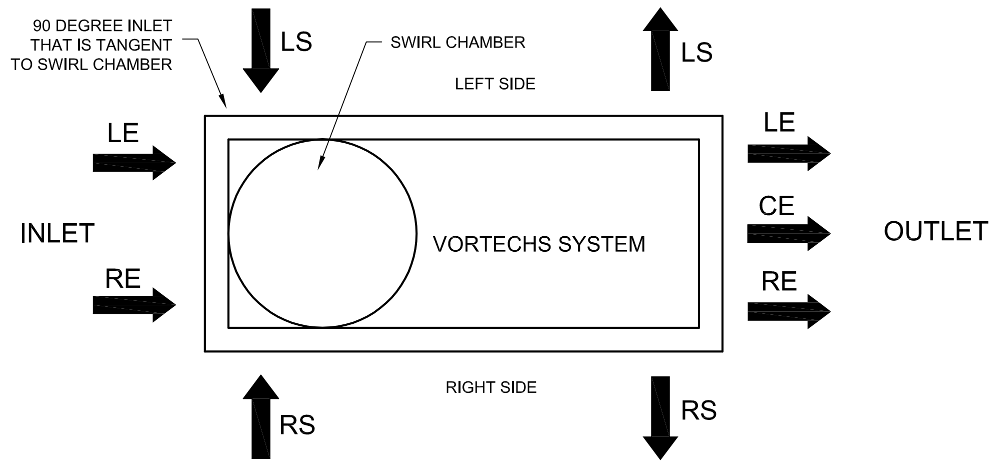
CONTECH® STORMWATER SOLUTIONS INC.

contechstormwater.com

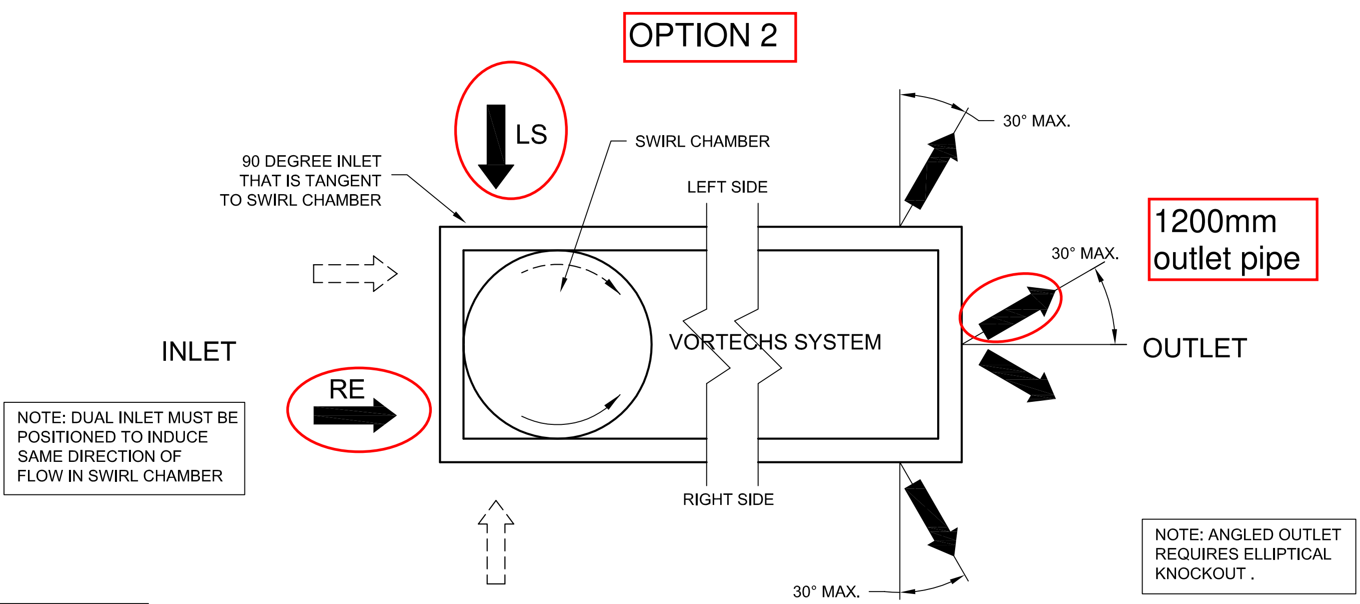
TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06	SCALE: NONE	FILE NAME: TYPVX ORIENTATION	DRAWN: GMC	CHECKED: NDG

NOTE: INLET PIPE MUST BE PERPENDICULAR TO WALL IT IS ENTERING ON



TYPICAL INLET/OUTLET ORIENTATION OPTIONS



DUAL INLET ORIENTATION OPTIONS

ANGLED OUTLET ORIENTATION OPTIONS

ORIENTATION KEY

LE = LEFT END
RE = RIGHT END
LS = LEFT SIDE
RS = RIGHT SIDE
CE = CENTER END

CONTECH® STORMWATER SOLUTIONS INC.
contechstormwater.com

TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06	SCALE: NONE	FILE NAME: TYPVX ORIENTATION	DRAWN: GMC	CHECKED: NDG

VORTECHS SYSTEM® ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS
Stinson Subdivision (4386 Rideau Valley Drive)
Ottawa, ON
Model 1522CIP In-line

CONTECH ENGINEERED SOLUTIONS

Design Ratio[1] = $\frac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(16.4 \text{ m2})}$ **= 0.69**

Rainfall Intensity mm/hr	Operating Rate[2] % of capacity	Flow Treated (l/s)	% Total Rainfall Volume[3]	Rmvl. Effcy[4] (%)	Rel. Effcy (%)
0.5	0.5	5.6	9.2%	98.0%	9.0%
1.0	1.0	11.2	10.6%	98.0%	10.4%
1.5	1.5	16.8	9.9%	98.0%	9.7%
2.0	2.0	22.4	8.4%	98.0%	8.2%
2.5	2.5	27.9	7.7%	98.0%	7.5%
3.0	3.0	33.5	5.9%	97.9%	5.8%
3.5	3.5	39.1	4.4%	97.9%	4.3%
4.0	4.0	44.7	4.7%	97.1%	4.5%
4.5	4.6	50.3	3.3%	97.1%	3.2%
5.0	5.1	55.9	3.0%	96.3%	2.9%
6.0	6.1	67.1	5.4%	95.6%	5.1%
7.0	7.1	78.2	4.4%	95.0%	4.1%
8.0	8.1	89.4	3.5%	93.7%	3.3%
9.0	9.1	100.6	2.8%	92.6%	2.6%
10.0	10.1	111.8	2.2%	91.9%	2.0%
15.0	15.2	167.6	7.0%	86.7%	6.1%
20.0	20.2	223.5	4.5%	81.4%	3.7%
25.0	25.3	279.4	1.4%	77.0%	1.1%
30.0	30.4	335.3	0.7%	73.1%	0.5%
35.0	35.4	391.1	0.5%	69.7%	0.3%
					94.5%
				Predicted Annual Runoff Volume Treated =	99.5%
				Assumed Removal Efficiency of remaining % =	0.0%
				Removal Efficiency Adjustment[5] =	0.0%
				Predicted Net Annual Load Removal Efficiency =	**94%**

1 - Design Ratio = (Total Drainage Area) x (Runoff Coefficient) x (Rational Method Conversion) / Grit Chamber Area
- The Total Drainage Area and Runoff Coefficient are specified by the site engineer.
- The rational method conversion based on the units in the above equation is 2.775.

2 - Operating Rate (% of capacity) = percentage of peak operating rate of 68 l/s/m^2.

3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON

4 - Based on Contech Stormwater Solutions laboratory verified removal of an average particle size of 80 microns (see Technical Bulletin #1).

5- Increase due to comparison of flows based on historical rational rainfall method and actual modeled by specifying engineer.

Calculated by: JAK 8/1/2022	Checked by:

Stinson Lands
Design Storm Time Series Data
6-hour Chicago Design Storms

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C25mm-6.stm		C2-6.stm		C5-6.stm	
Duration min	Intensity mm/hr	Duration min	Intensity mm/hr	Duration min	Intensity mm/hr
0:00	0	0:00	0	0:00	0
0:10	0.9292336	0:10	1.37	0:10	1.78
0:20	1.0106263	0:20	1.49	0:20	1.94
0:30	1.1055844	0:30	1.63	0:30	2.13
0:40	1.2344563	0:40	1.82	0:40	2.37
0:50	1.390459	0:50	2.05	0:50	2.68
1:00	1.6075062	1:00	2.37	1:00	3.1
1:10	1.9059462	1:10	2.81	1:10	3.68
1:20	2.3739543	1:20	3.5	1:20	4.58
1:30	3.1810988	1:30	4.69	1:30	6.15
1:40	4.9513905	1:40	7.3	1:40	9.61
1:50	12.351345	1:50	18.21	1:50	24.17
2:00	52.098123	2:00	76.81	2:00	104.19
2:10	16.332806	2:10	24.08	2:10	32.04
2:20	8.3834501	2:20	12.36	2:20	16.34
2:30	5.6432286	2:30	8.32	2:30	10.96
2:40	4.2731178	2:40	6.3	2:40	8.29
2:50	3.4524079	2:50	5.09	2:50	6.69
3:00	2.9097897	3:00	4.29	3:00	5.63
3:10	2.5231743	3:10	3.72	3:10	4.87
3:20	2.2315171	3:20	3.29	3:20	4.3
3:30	2.0009044	3:30	2.95	3:30	3.86
3:40	1.8177707	3:40	2.68	3:40	3.51
3:50	1.6685508	3:50	2.46	3:50	3.22
4:00	1.5464617	4:00	2.28	4:00	2.98
4:10	1.4379381	4:10	2.12	4:10	2.77
4:20	1.3497626	4:20	1.99	4:20	2.6
4:30	1.2683699	4:30	1.87	4:30	2.44
4:40	1.2005426	4:40	1.77	4:40	2.31
4:50	1.1394981	4:50	1.68	4:50	2.19
5:00	1.0852363	5:00	1.6	5:00	2.08
5:10	1.0309745	5:10	1.52	5:10	1.99
5:20	0.9902781	5:20	1.46	5:20	1.9
5:30	0.9495817	5:30	1.4	5:30	1.82
5:40	0.9088854	5:40	1.34	5:40	1.75
5:50	0.8749717	5:50	1.29	5:50	1.68
6:00	0.8410581	6:00	1.24	6:00	1.62

Stinson Lands
Design Storm Time Series Data
6-hour Chicago Design Storms

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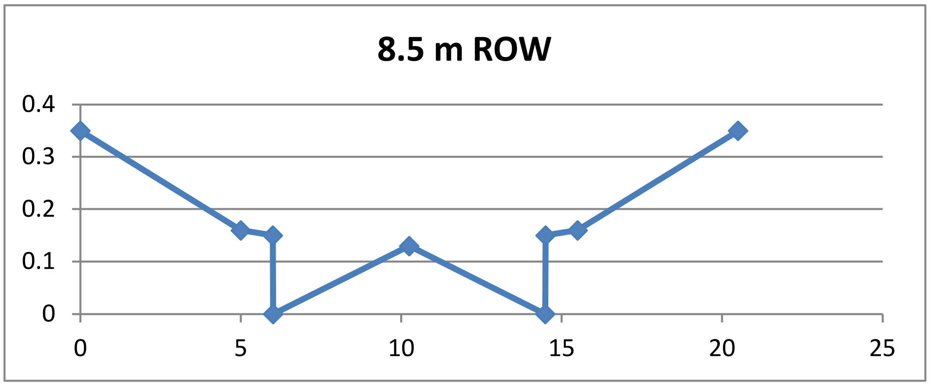
C100-6.stm			C100-6+20%.stm	
Duration	Intensity		Duration	Intensity
min	mm/hr		min	mm/hr
0:00	0.00		0:00	0.00
0:10	2.90		0:10	3.48
0:50	3.16		0:50	3.79
1:30	3.48		1:30	4.18
2:10	3.88		2:10	4.66
2:50	4.39		2:50	5.27
3:30	5.07		3:30	6.08
4:10	6.05		4:10	7.26
4:50	7.54		4:50	9.05
5:30	10.16		5:30	12.19
6:10	15.97		6:10	19.16
6:50	40.65		6:50	48.78
7:30	178.56		7:30	214.27
8:10	54.05		8:10	64.86
8:50	27.32		8:50	32.78
9:30	18.24		9:30	21.89
10:10	13.74		10:10	16.49
10:50	11.06		10:50	13.27
11:30	9.29		11:30	11.15
12:10	8.02		12:10	9.62
12:50	7.08		12:50	8.50
13:30	6.35		13:30	7.62
14:10	5.76		14:10	6.91
14:50	5.28		14:50	6.34
15:30	4.88		15:30	5.86
16:10	4.54		16:10	5.45
16:50	4.25		16:50	5.10
17:30	3.99		17:30	4.79
18:10	3.77		18:10	4.52
18:50	3.57		18:50	4.28
19:30	3.40		19:30	4.08
20:10	3.24		20:10	3.89
20:50	3.10		20:50	3.72
21:30	2.97		21:30	3.56
22:10	2.85		22:10	3.42
22:50	2.74		22:50	3.29
23:30	2.64		23:30	3.17

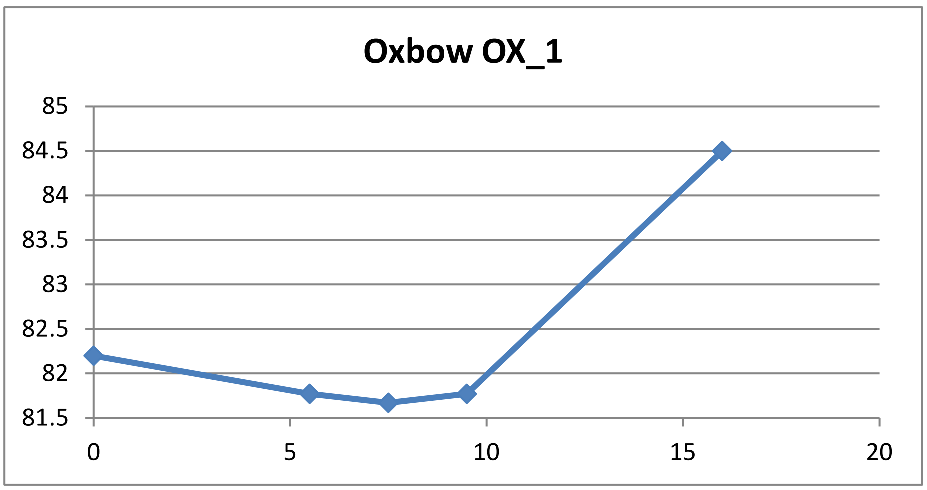
Local ROW

0	0.35
5	0.16
6	0.15
6.01	0
10.25	0.13
14.49	0
14.5	0.15
15.5	0.16
20.5	0.35



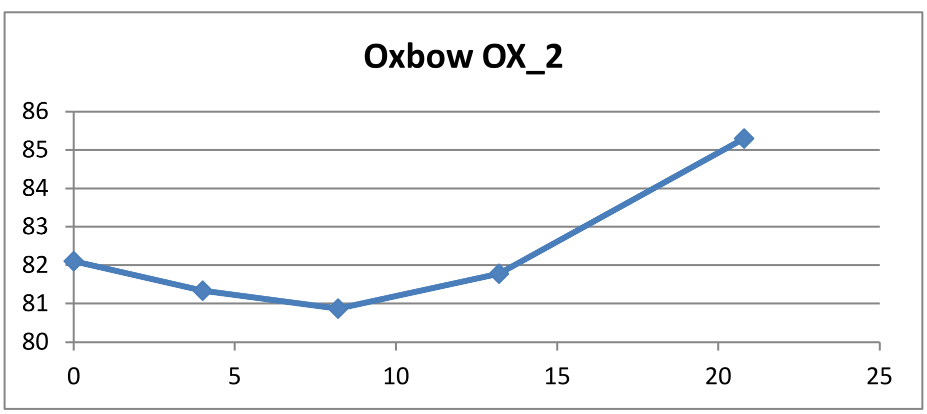
Oxbow OX_1

0	82.2
5.5	81.77
7.5	81.67
9.5	81.77
16	84.5



Oxbow OX_2

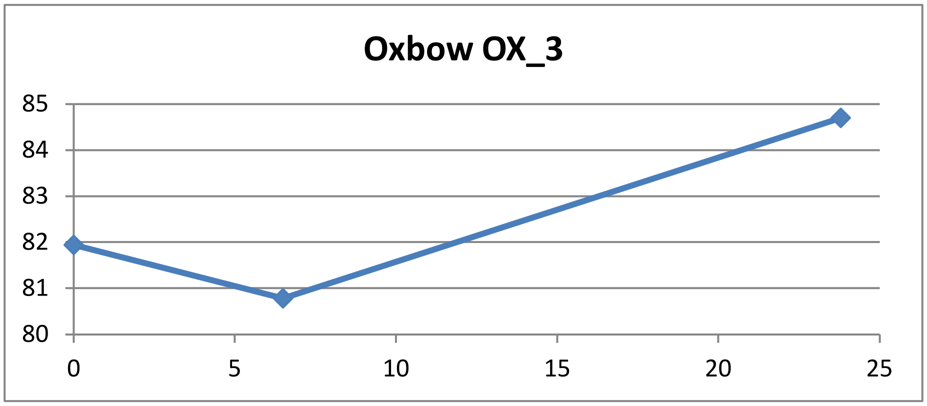
0	82.1
4	81.34
8.2	80.87
13.2	81.78
20.8	85.3



Stinson Lands
Cross-Sections

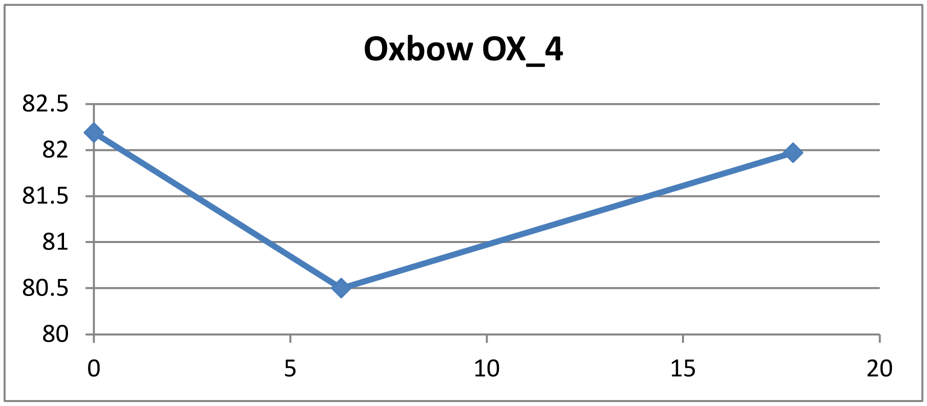
Oxbow OX_3

0	81.94
6.5	80.78
23.8	84.7



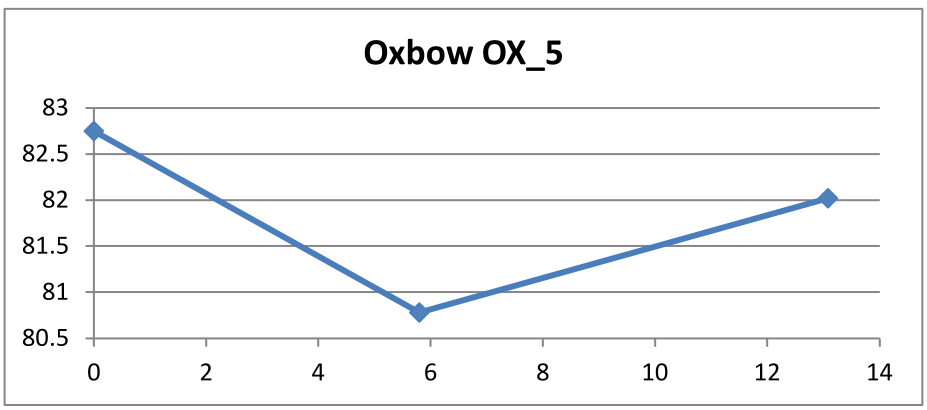
Oxbow OX_4

0	82.19
6.3	80.5
17.8	81.97

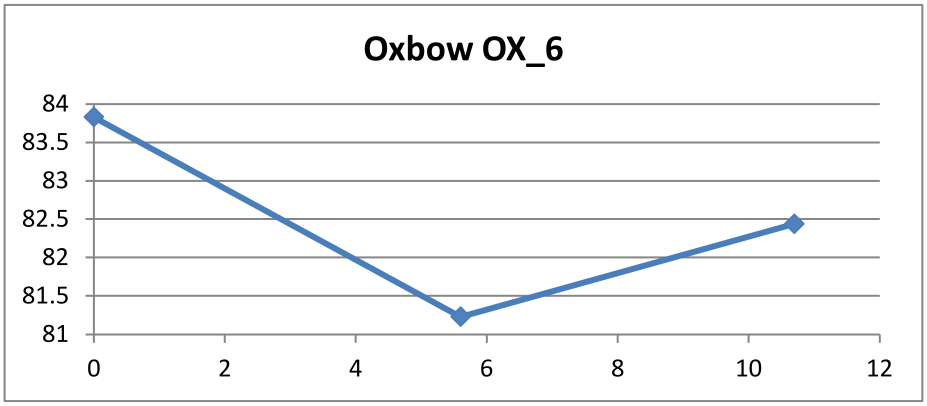


Oxbow OX_5

0	82.75
5.8	80.78
13.08	82.02



Oxbow OX_6	
0	83.83
5.6	81.23
10.7	82.44



Stinson Lands
HGL Elevations

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Manhole ID	MH Invert Elevation (m)	T/G Elevation (m)	Outlet pipe invert (m)	Outlet Pipe Diameter (m)	Outlet Pipe Obvert (m)	HGL Elevation (Chicago) (m)	WL Above Obvert (Chicago) (m)
135_(STM)	84.54	87.82	84.84	0.53	85.37	85.41	0.04
136_(STM)	86.44	89.77	86.74	0.30	87.04	86.90	-0.14
137_(STM)	89.89	93.22	90.19	0.25	90.44	90.29	-0.15
142_(STM)	84.33	87.92	84.63	0.60	85.23	85.24	0.01
144_(STM)	86.49	90.32	86.79	0.45	87.24	86.95	-0.29
145_(STM)	89.65	93.28	89.95	0.38	90.33	90.06	-0.27
146_(STM)	84.38	87.82	84.68	0.75	85.43	85.68	0.25
148_(STM)	85.03	87.85	85.33	0.38	85.71	85.83	0.13
149_(STM)	85.87	88.62	86.17	0.30	86.47	86.45	-0.02
150_(STM)	87.62	90.70	87.92	0.25	88.17	88.08	-0.09
151_(STM)	84.65	87.96	84.95	0.68	85.63	85.84	0.22
152_(STM)	84.81	88.20	85.11	0.60	85.71	85.97	0.26
153_(STM)	85.23	88.52	85.53	0.45	85.98	86.11	0.13
154_(STM)	85.74	89.18	86.04	0.38	86.42	86.33	-0.09
156_(STM)	86.08	89.56	86.38	0.30	86.68	86.57	-0.11
159_(STM)	85.27	88.31	85.57	0.45	86.02	86.02	0.00
169_(STM)	84.04	87.65	84.48	0.75	85.23	85.01	-0.22
170_(STM)	83.50	86.83	83.80	1.20	85.00	83.91	-1.09
186_(STM)	83.67	88.05	83.97	1.20	85.17	84.76	-0.41
187_(STM)	83.90	86.96	83.90	1.20	85.10	84.53	-0.57
189_(STM)	85.51	88.96	85.81	0.45	86.26	86.19	-0.07
191_(STM)	85.40	88.13	85.70	0.30	86.00	86.06	0.06
193_(STM)	86.51	89.34	86.81	0.25	87.06	86.97	-0.09

CHAPTER 10: RIPRAP BASINS AND APRONS

Riprap is a material that has long been used to protect against the forces of water. The material can be pit-run (as provided by the supplier) or specified (standard or special). State DOTs have standard specifications for a number of classes (sizes or gradations) of riprap. Suppliers maintain an inventory of frequently used classes. Special gradations of riprap are produced on-demand and are therefore more expensive than both pit-run and standard classes.

This chapter includes discussion of both riprap aprons and riprap basin energy dissipators. Both can be used at the outlet of a culvert or chute (channel) by themselves or at the exit of a stilling basin or other energy dissipator to protect against erosion downstream. Section 10.1 provides a design procedure for the riprap basin energy dissipator that is based on armoring a pre-formed scour hole. The riprap for this basin is a special gradation. Section 10.2 includes discussion of riprap aprons that provide a flat armored surface as the only dissipator or as additional protection at the exit of other dissipators. The riprap for these aprons is generally from State DOT standard classes. Section 10.3 provides additional discussion of riprap placement downstream of energy dissipators.

10.1 RIPRAP BASIN

The design procedure for the riprap basin is based on research conducted at Colorado State University (Simons, et al., 1970; Stevens and Simons, 1971) that was sponsored by the Wyoming Highway Department. The recommended riprap basin that is shown on Figure 10.1 and Figure 10.2 has the following features:

- The basin is pre-shaped and lined with riprap that is at least $2D_{50}$ thick.
- The riprap floor is constructed at the approximate depth of scour, h_s, that would occur in a thick pad of riprap. The h_s/D_{50} of the material should be greater than 2.
- The length of the energy dissipating pool, L_s, is $10h_s$, but no less than $3W_o$; the length of the apron, L_A, is $5h_s$, but no less than W_o. The overall length of the basin (pool plus apron), L_B, is $15h_s$, but no less than $4W_o$.
- A riprap cutoff wall or sloping apron can be constructed if downstream channel degradation is anticipated as shown in Figure 10.1.

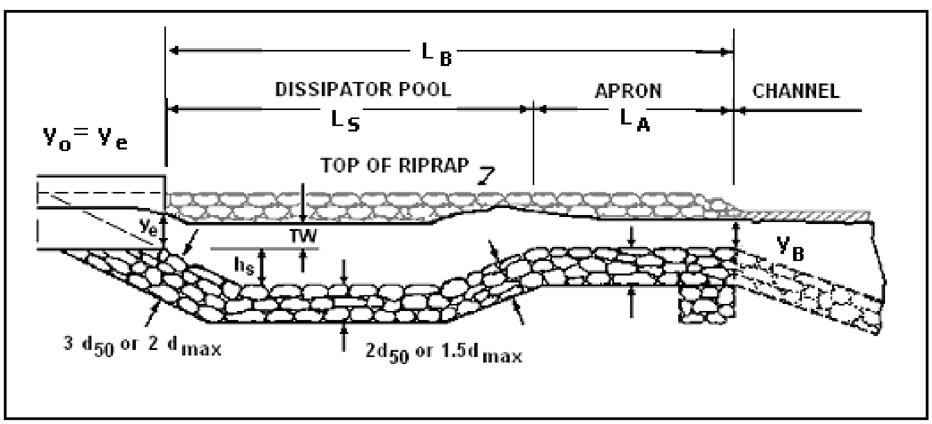


Figure 10.1. Profile of Riprap Basin

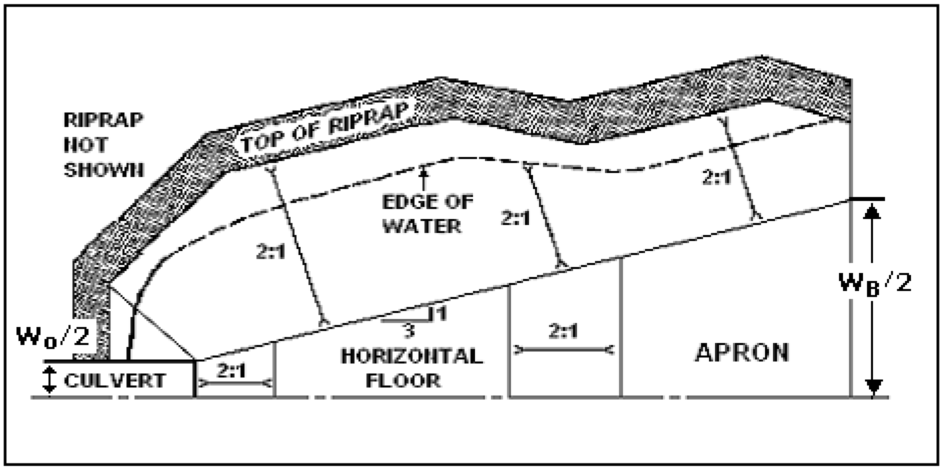


Figure 10.2. Half Plan of Riprap Basin

10.1.1 Design Development

Tests were conducted with pipes from 152 mm (6 in) to 914 mm (24 in) and 152 mm (6 in) high model box culverts from 305 mm (12 in) to 610 mm (24 in) in width. Discharges ranged from 0.003 to 2.8 m^3/s (0.1 to 100 ft^3/s). Both angular and rounded rock with an average size, D_{50}, ranging from 6 mm (1.4 in) to 177 mm (7 in) and gradation coefficients ranging from 1.05 to 2.66 were tested. Two pipe slopes were considered, 0 and 3.75%. In all, 459 model basins were studied. The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics:

- The scour hole depth, h_s; length, L_s; and width, W_s, are related to the size of riprap, D_{50}; discharge, Q; brink depth, y_o; and tailwater depth, TW.
- Rounded material performs approximately the same as angular rock.
- For low tailwater ($TW/y_o < 0.75$), the scour hole functions well as an energy dissipator if $h_s/D_{50} > 2$. The flow at the culvert brink plunges into the hole, a jump forms and flow is generally well dispersed.
- For high tailwater ($TW/y_o > 0.75$), the high velocity core of water passes through the basin and diffuses downstream. As a result, the scour hole is shallower and longer.
- The mound of material that forms downstream contributes to the dissipation of energy and reduces the size of the scour hole. If the mound is removed, the scour hole enlarges somewhat.

Plots were constructed of h_s/y_e versus $V_o/ (gy_e)^{1/2}$ with D_{50}/y_e as the third variable. Equivalent brink depth, y_e, is defined to permit use of the same design relationships for rectangular and circular culverts. For rectangular culverts, $y_e = y_o$ (culvert brink depth). For circular culverts, $y_e = (A/2)^{1/2}$, where A is the brink area.

Anticipating that standard or modified end sections would not likely be used when a riprap basin is located at a culvert outlet, the data with these configurations were not used to develop the design relationships. This assumption reduced the number of applicable runs to 346. A total of 128 runs had a D_{50}/y_e of less than 0.1. These data did not exhibit relationships that appeared useful for design and were eliminated. An additional 69 runs where $h_s/D_{50}<2$ were also eliminated by the authors of this edition of HEC 14. These runs were not considered reliable for design, especially those with $h_s = 0$. Therefore, the final design development used 149 runs from the study. Of these, 106 were for pipe culverts and 43 were for box culverts. Based on these data, two design relationships are presented here: an envelope design and a best fit design.

To balance the need for avoiding an underdesigned basin against the costs of oversizing a basin, an envelope design relationship in the form of Equation 10.1 and Equation 10.2 was developed. These equations provide a design envelope for the experimental data equivalent to the design figure (Figure XI-2) provided in the previous edition of HEC 14 (Corry, et al., 1983). Equations 10.1 and 10.2, however, improve the fit to the experimental data reducing the root-mean-square (RMS) error from 1.24 to 0.83.

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{g y_e}}\right) - C_o \tag{10.1}$$

where,

- h_s = dissipator pool depth, m (ft)
- y_e = equivalent brink (outlet) depth, m (ft)
- D_{50} = median rock size by weight, m (ft)
- C_o = tailwater parameter

The tailwater parameter, C_o, is defined as:

$$\begin{array}{ll} C_o = 1.4 & TW/y_e < 0.75 \\ C_o = 4.0(TW/y_e) - 1.6 & 0.75 < TW/y_e < 1.0 \\ C_o = 2.4 & 1.0 < TW/y_e \end{array} \tag{10.2}$$

A best fit design relationship that minimizes the RMS error when applied to the experimental data was also developed. Equation 10.1 still applies, but the description of the tailwater parameter, C_o, is defined in Equation 10.3. The best fit relationship for Equations 10.1 and 10.3 exhibits a RMS error on the experimental data of 0.56.

$$\begin{array}{ll} C_o = 2.0 & TW/y_e < 0.75 \\ C_o = 4.0(TW/y_e) - 1.0 & 0.75 < TW/y_e < 1.0 \\ C_o = 3.0 & 1.0 < TW/y_e \end{array} \tag{10.3}$$

Use of the envelope design relationship (Equations 10.1 and 10.2) is recommended when the consequences of failure at or near the design flow are severe. Use of the best fit design relationship (Equations 10.1 and 10.3) is recommended when basin failure may easily be addressed as part of routine maintenance. Intermediate risk levels can be adopted by the use of intermediate values of C_o.

10.1.2 Basin Length

Frequency tables for both box culvert data and pipe culvert data of relative length of scour hole ($L_s/h_s < 6$, $6 < L_s/h_s < 7$, $7 < L_s/h_s < 8$ . . . $25 < L_s/h_s < 30$), with relative tailwater depth TW/y_e in increments of 0.03 m (0.1 ft) as a third variable, were constructed using data from 346 experimental runs. For box culvert runs L_s/h_s was less than 10 for 78% of the data and L_s/h_s was less than 15 for 98% of the data. For pipe culverts, L_s/h_s was less than 10 for 91% of the data and, L_s/h_s was less than 15 for all data. A 3:1 flare angle is recommended for the basins walls. This angle will provide a sufficiently wide energy dissipating pool for good basin operation.

10.1.3 High Tailwater

Tailwater influenced formation of the scour hole and performance of the dissipator. For tailwater depths less than 0.75 times the brink depth, scour hole dimensions were unaffected by tailwater. Above this the scour hole became longer and narrower. The tailwater parameter defined in Equations 10.2 and 10.3 captures this observation. In addition, under high tailwater conditions, it is appropriate to estimate the attenuation of the flow velocity downstream of the culvert outlet using Figure 10.3. This attenuation can be used to determine the extent of riprap protection required. HEC 11 (Brown and Clyde, 1989) or the method provided in Section 10.3 can be used for sizing riprap.

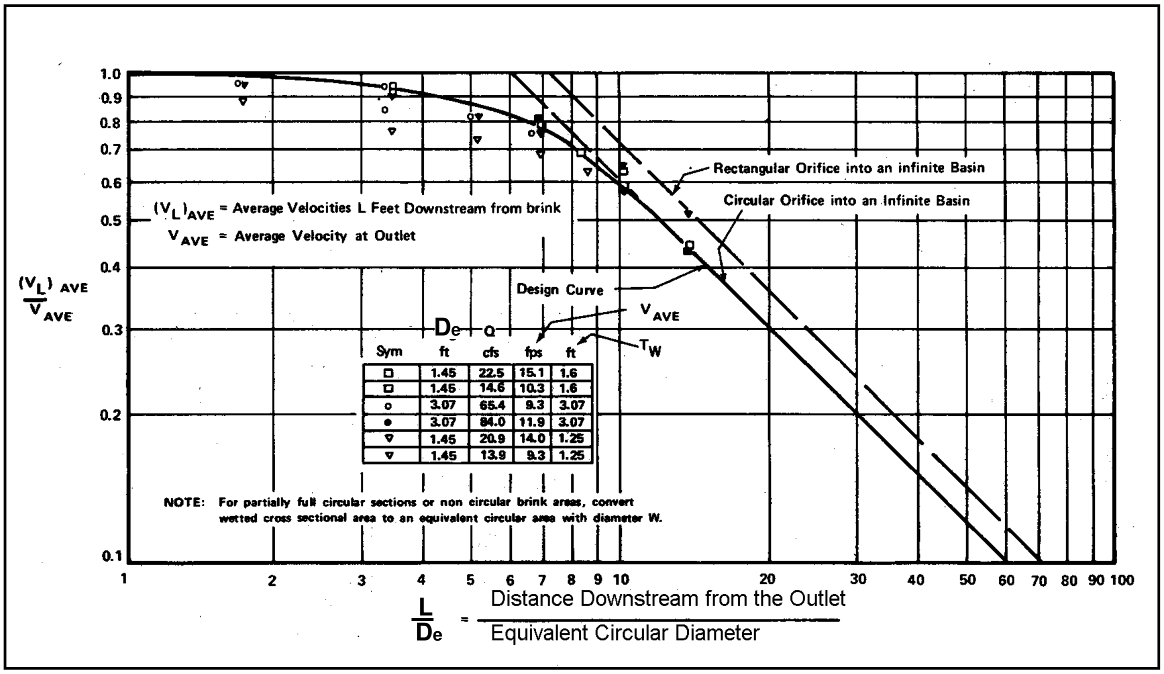


Figure 10.3. Distribution of Centerline Velocity for Flow from Submerged Outlets

10.1.4 Riprap Details

Based on experience with conventional riprap design, the recommended thickness of riprap for the floor and sides of the basin is $2D_{50}$ or $1.50D_{max}$, where D_{max} is the maximum size of rock in the riprap mixture. Thickening of the riprap layer to $3D_{50}$ or $2D_{max}$ on the foreslope of the roadway culvert outlet is warranted because of the severity of attack in the area and the necessity for preventing undermining and consequent collapse of the culvert. Figure 10.1 illustrates these riprap details. The mixture of stone used for riprap and need for a filter should meet the specifications described in HEC 11 (Brown and Clyde, 1989).

10.1.5 Design Procedure

The design procedure for a riprap basin is as follows:

Step 1. Compute the culvert outlet velocity, V_o, and depth, y_o.

For subcritical flow (culvert on mild or horizontal slope), use Figure 3.3 or Figure 3.4 to obtain y_o/D, then obtain V_o by dividing Q by the wetted area associated with y_o. D is the height of a box culvert or diameter of a circular culvert.

For supercritical flow (culvert on a steep slope), V_o will be the normal velocity obtained by using the Manning's Equation for appropriate slope, section, and discharge.

Compute the Froude number, Fr, for brink conditions using brink depth for box culverts ($y_e=y_o$) and equivalent depth ($y_e = (A/2)^{1/2}$) for non-rectangular sections.

Step 2. Select D_{50} appropriate for locally available riprap. Determine C_o from Equation 10.2 or 10.3 and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$. If h_s/D_{50} or D_{50}/y_e is out of this range, try a different riprap size. (Basins sized where h_s/D_{50} is greater than, but close to, 2 are often the most economical choice.)

Step 3. Determine the length of the dissipation pool (scour hole), L_s, total basin length, L_B, and basin width at the basin exit, W_B, as shown in Figures 10.1 and 10.2. The walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

Step 4. Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$ and compare with the allowable exit velocity, V_{allow}. The allowable exit velocity may be taken as the estimated normal velocity in the tailwater channel or a velocity specified based on stability criteria, whichever is larger. Critical depth at the basin exit may be determined iteratively using Equation 7.14:

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$ by trial and success to determine y_B.

$V_c = Q/A_c$

z = basin side slope, z:1 (H:V)

If $V_c \leq V_{allow}$, the basin dimensions developed in step 3 are acceptable. However, it may be possible to reduce the size of the dissipator pool and/or the apron with a larger riprap size. It may also be possible to maintain the dissipator pool, but reduce the flare on the apron to reduce the exit width to better fit the downstream channel. Steps 2 through 4 are repeated to evaluate alternative dissipator designs.

Step 5. Assess need for additional riprap downstream of the dissipator exit. If $TW/y_o \leq 0.75$, no additional riprap is needed. With high tailwater ($TW/y_o \geq 0.75$), estimate centerline velocity at a series of downstream cross sections using Figure 10.3 to determine the size and extent of additional protection. The riprap design details should be in accordance with specifications in HEC 11 (Brown and Clyde, 1989) or similar highway department specifications.

Two design examples are provided. The first features a box culvert on a steep slope while the second shows a pipe culvert on a mild slope.

Design Example: Riprap Basin (Culvert on a Steep Slope) (SI)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and 10.2) for a 2440 mm by 1830 mm reinforced concrete box (RCB) culvert that is in inlet control with supercritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 2.1 m/s. Riprap is available with a D_{50} of 0.50, 0.55, and 0.75 m. Consider two tailwater conditions: 1) TW = 0.85 m and 2) TW = 1.28 m. Given:

Q = 22.7 m^3/s

y_o = 1.22 m (normal flow depth) = brink depth

Solution

Step 1. Compute the culvert outlet velocity, V_o, depth, y_o, and Froude number for brink conditions. For supercritical flow (culvert on a steep slope), V_o will be V_n

$y_o = y_e = 1.22$ m

$V_o = Q/A = 22.7/ [1.22 (2.44)] = 7.63$ m/s

$Fr = V_o / (9.81y_e)^{1/2} = 7.63/ [9.81(1.22)]^{1/2} = 2.21$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 0.55$ m; $D_{50}/y_e = 0.55/1.22 = 0.45$ (≥ 0.1 OK)

Two tailwater elevations are given; use the lowest to determine the basin size that will serve the tailwater range, that is, TW = 0.85 m.

$TW/y_e = 0.85/1.22 = 0.7$, which is less than 0.75. Therefore, from Equation 10.2, $C_o = 1.4$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.45)^{-0.55}(2.21) - 1.4 = 1.55$$

$h_S = (h_S /y_e)y_e = 1.55 (1.22) = 1.89$ m

$h_S/D_{50} = 1.89/0.55 = 3.4$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(1.89) = 18.9$ m

L_S min $= 3W_o = 3(2.44) = 7.3$ m, use $L_S = 18.9$ m

$L_B = 15h_S = 15(1.89) = 28.4$ m

L_B min $= 4W_o = 4(2.44) = 9.8$ m, use $L_B = 28.4$ m

$W_B = W_o + 2(L_B/3) = 2.44 + 2(28.4/3) = 21.4$ m

Step 4. Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$22.7^2/9.81 = 52.5 = [y_c(21.4 + 2y_c)]^3/ (21.4 + 4y_c)$

By trial and success, $y_c = 0.48$ m, $T_c = 23.3$ m, $A_c = 10.7$ m^2

$V_B = V_c = Q/A_c = 22.7/10.7 = 2.1$ m/s (acceptable)

The initial trial of riprap ($D_{50} = 0.55$ m) results in a 28.4 m basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 0.75$ m; $D_{50}/y_e = 0.75/1.22 = 0.61$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.61)^{-0.55}(2.21) - 1.4 = 1.09$$

$h_S = (h_S/y_e)y_e = 1.09\,(1.22) = 1.34$ m

$h_S/D_{50} = 1.34/0.75 = 1.8$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than, 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 0.71$ m; $D_{50}/y_e = 0.71/1.22 = 0.58$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.58)^{-0.55}(2.21) - 1.4 = 1.16$$

$h_S = (h_S/y_e)y_e = 1.16\,(1.22) = 1.42$ m

$h_S/D_{50} = 1.42/0.71 = 2.0$ and $h_S/D_{50} \geq 2$ is satisfied.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(1.42) = 14.2$ m

L_S min $= 3W_o = 3(2.44) = 7.3$ m, use $L_S = 14.2$ m

$L_B = 15h_S = 15(1.42) = 21.3$ m

L_B min $= 4W_o = 4(2.44) = 9.8$ m, use $L_B = 21.3$ m

$W_B = W_o + 2(L_B/3) = 2.44 + 2(21.3/3) = 16.6$ m

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 0.75$ m) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$22.7^2/9.81 = 52.5 = [y_c(16.6 + 2y_c)]^3/ (16.6 + 4y_c)$

By trial and success, $y_c = 0.56$ m, $T_c = 18.8$ m, $A_c = 9.9$ m^2

$V_B = V_c = Q/A_c = 22.7/9.9 = 2.3$ m/s (greater than 2.1 m/s; not acceptable). If the apron were extended (with a continued flare) such that the total basin length was 28.4 m, the velocity would be reduced to the allowable level.

Two feasible options have been identified. First, a 1.89 m deep, 18.9 m long pool, with a 9.5 m apron using $D_{50} = 0.55$ m. Second, a 1.42 m deep, 14.2 m long pool, with a 14.2 m apron using $D_{50} = 0.75$ m. Because the overall length is the same, the first option is likely to be more economical.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$.

For the first tailwater condition, $TW/y_o = 0.85/1.22 = 0.70$, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed downstream.

For the second tailwater condition, $TW/y_o = 1.28/1.22 = 1.05$, which does not satisfy $TW/y_o \leq 0.75$. To determine required riprap, estimate centerline velocity at a series of downstream cross sections using Figure 10.3.

Compute equivalent circular diameter, D_e, for brink area:

$A = \pi D_e^2 /4 = (y_o)(W_o) = (1.22)(2.44) = 3.00 \text{ m}^2$

$D_e = [3.00(4)/ \pi]^{1/2} = 1.95$ m

Rock size can be determined using the procedures in Section 10.3 (Equation 10.6) or other suitable method. The computations are summarized below.

L/D_e	L (m)	V_L/V_o (Figure 10.3)	V_L (m/s)	Rock size, D_{50} (m)
10	19.5	0.59	4.50	0.43
15	29.3	0.42	3.20	0.22
20	39.0	0.30	2.29	0.11
21	41.0	0.28	2.13	0.10

The calculations above continue until $V_L \leq V_{allow}$. Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 41.0 m downstream from the culvert brink, which is 12.6 m beyond the basin exit. Riprap should be installed in accordance with details shown in HEC 11.

Design Example: Riprap Basin (Culvert on a Steep Slope) (CU)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and10.2) for an 8 ft by 6 ft reinforced concrete box (RCB) culvert that is in inlet control with supercritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 7 ft/s. Riprap is available with a D_{50} of 1.67, 1.83, and 2.5 ft. Consider two tailwater conditions: 1) TW = 2.8 ft and 2) TW = 4.2 ft. Given:

Q = 800 ft^3/s

y_o = 4 ft (normal flow depth) = brink depth

Solution

Step 1. Compute the culvert outlet velocity, V_o, depth, y_o, and Froude number for brink conditions. For supercritical flow (culvert on a steep slope), V_o will be V_n.

$y_o = y_e = 4$ ft

$V_o = Q/A = 800/ [4 (8)] = 25$ ft/s

$Fr = V_o / (32.2y_e)^{1/2} = 25/ [32.2(4)]^{1/2} = 2.2$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 1.83$ ft; $D_{50}/y_e = 1.83/4 = 0.46$ (≥ 0.1 OK)

Two tailwater elevations are given; use the lowest to determine the basin size that will serve the tailwater range, that is, TW = 2.8 ft.

$TW/y_e = 2.8/4 = 0.7$, which is less than 0.75. From Equation 10.2, $C_o = 1.4$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.46)^{-0.55}(2.2) - 1.4 = 1.50$$

$h_S = (h_S /y_e)y_e = 1.50\ (4) = 6.0$ ft

$h_S/D_{50} = 6.0/1.83 = 3.3$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(6.0) = 60$ ft

L_S min $= 3W_o = 3(8) = 24$ ft, use $L_S = 60$ ft

$L_B = 15h_S = 15(6.0) = 90$ ft

L_B min $= 4W_o = 4(8) = 32$ ft, use $L_B = 90$ ft

$W_B = W_o + 2(L_B/3) = 8 + 2(90/3) = 68$ ft

Step 4. Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$800^2/32.2 = 19{,}876 = [y_c(68 + 2y_c)]^3/ (68 + 4y_c)$

By trial and success, $y_c = 1.60$ ft, $T_c = 74.4$ ft, $A_c = 113.9$ ft^2

$V_B = V_c = Q/A_c = 800/113.9 = 7.0$ ft/s (acceptable)

The initial trial of riprap ($D_{50} = 1.83$ ft) results in a 90 ft basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 2.5$ ft; $D_{50}/y_e = 2.5/4 = 0.63$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.63)^{-0.55}(2.2) - 1.4 = 1.04$$

$h_S = (h_S / y_e) y_e = 1.04\ (4) = 4.2$ ft

$h_S/D_{50} = 4.2/2.5 = 1.7$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than, 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select riprap size and compute basin depth.

Try $D_{50} = 2.3$ ft; $D_{50}/y_e = 2.3/4 = 0.58$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.58)^{-0.55}(2.2) - 1.4 = 1.15$$

$h_S = (h_S / y_e) y_e = 1.15\ (4) = 4.6$ ft

$h_S/D_{50} = 4.6/2.3 = 2.0$ and $h_S/D_{50} \geq 2$ is satisfied.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(4.6) = 46$ ft

L_S min $= 3W_o = 3(8) = 24$ ft, use $L_S = 46$ ft

$L_B = 15h_S = 15(4.6) = 69$ ft

L_B min $= 4W_o = 4(8) = 32$ ft, use $L_B = 69$ ft

$W_B = W_o + 2(L_B/3) = 8 + 2(69/3) = 54$ ft

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 2.5$ ft) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$, and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$800^2/32.2 = 19{,}876 = [y_c(54 + 2y_c)]^3/ (54 + 4y_c)$

By trial and success, $y_c = 1.85$ ft, $T_c = 61.4$ ft, $A_c = 106.9$ ft^2

$V_B = V_c = Q/A_c = 800/106.9 = 7.5$ ft/s (not acceptable). If the apron were extended (with a continued flare) such that the total basin length was 90 ft, the velocity would be reduced to the allowable level.

Two feasible options have been identified. First, a 6-ft-deep, 60-ft-long pool, with a 30-ft-apron using $D_{50} = 1.83$ ft. Second, a 4.6-ft-deep, 46-ft-long pool, with a 44-ft-apron using $D_{50} = 2.5$ ft. Because the overall length is the same, the first option is likely to be more economical.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$.

For the first tailwater condition, $TW/y_o = 2.8/4.0 = 0.70$, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed downstream.

For the second tailwater condition, $TW/y_o = 4.2/4.0 = 1.05$, which does not satisfy $TW/y_o \leq 0.75$. To determine required riprap, estimate centerline velocity at a series of downstream cross sections using Figure 10.3.

Compute equivalent circular diameter, D_e, for brink area:

$A = \pi D_e^2 /4 = (y_o)(W_o) = (4)(8) = 32\ ft^2$

$D_e = [32(4)/ \pi]^{1/2} = 6.4$ ft

Rock size can be determined using the procedures in Section 10.3 (Equation 10.6) or other suitable method. The computations are summarized below.

L/D_e	L (ft)	V_L/V_o (Figure 10.3)	V_L (ft/s)	Rock size, D_{50} (ft)
10	64	0.59	14.7	1.42
15	96	0.42	10.5	0.72
20	128	0.30	7.5	0.37
21	135	0.28	7.0	0.32

The calculations above continue until $V_L \leq V_{allow}$. Riprap should be at least the size shown. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink, which is 45 ft beyond the basin exit. Riprap should be installed in accordance with details shown in HEC 11.

Design Example: Riprap Basin (Culvert on a Mild Slope) (SI)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and 10.2) for a pipe culvert that is in outlet control with subcritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 2.1 m/s. Riprap is available with a D_{50} of 0.125, 0.150, and 0.250 m. Given:

D = 1.83 m CMP with Manning's n = 0.024
S_o = 0.004 m/m
Q = 3.82 m^3/s
y_n = 1.37 m (normal flow depth in the pipe)
V_n = 1.80 m/s (normal velocity in the pipe)
TW = 0.61 m (tailwater depth)

Solution

Step 1. Compute the culvert outlet velocity, V_o, and depth, y_o.

For subcritical flow (culvert on mild slope), use Figure 3.4 to obtain y_o/D, then calculate V_o by dividing Q by the wetted area for y_o.

$K_u\ Q/D^{2.5} = 1.81\ (3.82)/1.83^{2.5} = 1.53$

$TW/D = 0.61/1.83 = 0.33$

From Figure 3.4, $y_o/D = 0.45$

$y_o = (y_o/D)D = 0.45(1.83) = 0.823$ m (brink depth)

From Table B.2, for $y_o/D = 0.45$, the brink area ratio $A/D^2 = 0.343$

$A = (A/D^2)D^2 = 0.343(1.83)^2 = 1.15\ m^2$

$V_o = Q/A = 3.82/1.15 = 3.32$ m/s

$y_e = (A/2)^{1/2} = (1.15/2)^{1/2} = 0.76$ m

$Fr = V_o / [9.81(y_e)]^{1/2} = 3.32/ [9.81(0.76)]^{1/2} = 1.22$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 0.15$ m; $D_{50}/y_e = 0.15/0.76 = 0.20$ (≥ 0.1 OK)

$TW/y_e = 0.61/0.76 = 0.80$. Therefore, from Equation 10.2,

$C_o = 4.0(TW/y_e) -1.6 = 4.0(0.80) -1.6 = 1.61$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.20)^{-0.55}(1.22) - 1.61 = 0.933$$

$h_S = (h_S/y_e)y_e = 0.933\ (0.76) = 0.71$ m

$h_S/D_{50} = 0.71/0.15 = 4.7$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(0.71) = 7.1$ m

L_S min $= 3W_o = 3(1.83) = 5.5$ m, use $L_S = 7.1$ m

$L_B = 15h_S = 15(0.71) = 10.7$ m

L_B min $= 4W_o = 4(1.83) = 7.3$ m, use $L_B = 10.7$ m

$W_B = W_o + 2(L_B/3) = 1.83 + 2(10.7/3) = 9.0$ m

Step 4. Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$3.82^2/9.81 = 1.49 = [y_c(9.0 + 2y_c)]^3/ (9.0 + 4y_c)$

By trial and success, $y_c = 0.26$ m, $T_c = 10.0$ m, $A_c = 2.48\ m^2$

$V_c = Q/A_c = 3.82/2.48 = 1.5$ m/s (acceptable)

The initial trial of riprap ($D_{50} = 0.15$ m) results in a 10.7 m basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.25$ m; $D_{50}/y_e = 0.25/0.76 = 0.33$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.33)^{-0.55}(1.22) - 1.61 = 0.320$$

$h_S = (h_S / y_e) y_e = 0.320\ (0.76) = 0.24$ m

$h_S/D_{50} = 0.24/0.25 = 0.96$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.205$ m; $D_{50}/y_e = 0.205/0.76 = 0.27$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.27)^{-0.55}(1.22) - 1.61 = 0.545$$

$h_S = (h_S / y_e) y_e = 0.545\ (0.76) = 0.41$ m

$h_S/D_{50} = 0.41/0.205 = 2.0$ and $h_S/D_{50} \geq 2$ is satisfied. Continue to step 3.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(0.41) = 4.1$ m

L_S min $= 3W_o = 3(1.83) = 5.5$ m, use $L_S = 5.5$ m

$L_B = 15h_S = 15(0.41) = 6.2$ m

L_B min $= 4W_o = 4(1.83) = 7.3$ m, use $L_B = 7.3$ m

$W_B = W_o + 2(L_B/3) = 1.83 + 2(7.3/3) = 6.7$ m

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 0.25$ m) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/\ (W_B + 2zy_c)$

$3.82^2/9.81 = 1.49 = [y_c(6.7 + 2y_c)]^3/\ (6.7 + 4y_c)$

By trial and success, $y_c = 0.31$ m, $T_c = 7.94$ m, $A_c = 2.28\ m^2$

$V_c = Q/A_c = 3.82/2.28 = 1.7$ m/s (acceptable)

Two feasible options have been identified. First, a 0.71 m deep, 7.1 m long pool, with an 3.6 m apron using $D_{50} = 0.15$ m. Second, a 0.41 m deep, 5.5 m long pool, with a 1.8 m apron using $D_{50} = 0.25$ m. The choice between these two options will likely depend on the available space and the cost of riprap.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$

$TW/y_o = 0.61/0.823 = 0.74$, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed.

Design Example: Riprap Basin (Culvert on a Mild Slope) (CU)

Determine riprap basin dimensions using the envelope design (Equations 10.1 and 10.2) for a pipe culvert that is in outlet control with subcritical flow in the culvert. Allowable exit velocity from the riprap basin, V_{allow}, is 7.0 ft/s. Riprap is available with a D_{50} of 0.42, 0.50, and 0.83 ft. Given:

D	=	6 ft CMP with Manning's n = 0.024
S_o	=	0.004 ft/ft
Q	=	135 ft^3/s
y_n	=	4.5 ft (normal flow depth in the pipe)
V_n	=	5.9 ft/s (normal velocity in the pipe)
TW	=	2.0 ft (tailwater depth)

Solution

Step 1. Compute the culvert outlet velocity, V_o, depth, y_o and Froude number.

For subcritical flow (culvert on mild slope), use Figure 3.4 to obtain y_o/D, then calculate V_o by dividing Q by the wetted area for y_o.

$K_uQ/D^{2.5} = 1.0(135)/6^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

From Figure 3.4, $y_o/D = 0.45$

$y_o = (y_o/D)D = 0.45(6) = 2.7$ ft (brink depth)

From Table B.2 for $y_o/D = 0.45$, the brink area ratio $A/D^2 = 0.343$

$A = (A/D^2)D^2 = 0.343(6)^2 = 12.35\ ft^2$

$V_o = Q/A = 135/12.35 = 10.9$ ft/s

$y_e = (A/2)^{1/2} = (12.35/2)^{1/2} = 2.48$ ft

$Fr = V_o / [32.2(y_e)]^{1/2} = 10.9/ [32.2(2.48)]^{1/2} = 1.22$

Step 2. Select a trial D_{50} and obtain h_s/y_e from Equation 10.1. Check to see that $h_s/D_{50} \geq 2$ and $D_{50}/y_e \geq 0.1$.

Try $D_{50} = 0.5$ ft; $D_{50}/y_e = 0.5/2.48 = 0.20$ (≥ 0.1 OK)

$TW/y_e = 2.0/2.48 = 0.806$. Therefore, from Equation 10.2,

$C_o = 4.0(TW/y_e) -1.6 = 4.0(0.806) -1.6 = 1.62$

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.20)^{-0.55}(1.22) - 1.62 = 0.923$$

$h_S = (h_S/y_e)y_e = 0.923\ (2.48) = 2.3$ ft

$h_S/D_{50} = 2.3/0.5 = 4.6$ and $h_S/D_{50} \geq 2$ is satisfied

Step 3. Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(2.3) = 23$ ft

L_S min $= 3W_o = 3(6) = 18$ ft, use $L_S = 23$ ft

$L_B = 15h_S = 15(2.3) = 34.5$ ft

L_B min $= 4W_o = 4(6) = 24$ ft, use $L_B = 34.5$ ft

$W_B = W_o + 2(L_B/3) = 6 + 2(34.5/3) = 29$ ft

Step 4. Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$135^2/32.2 = 566 = [y_c(29 + 2y_c)]^3/ (29 + 4y_c)$

By trial and success, $y_c = 0.86$ ft, $T_c = 32.4$ ft, $A_c = 26.4$ ft^2

$V_c = Q/A_c = 135/26.4 = 5.1$ ft/s (acceptable)

The initial trial of riprap ($D_{50} = 0.5$ ft) results in a 34.5 ft basin that satisfies all design requirements. Try the next larger riprap size to test if a smaller basin is feasible by repeating steps 2 through 4.

Step 2 (2nd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.83$ ft; $D_{50}/y_e = 0.83/2.48 = 0.33$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.33)^{-0.55}(1.22) - 1.62 = 0.311$$

$h_S = (h_S/y_e)y_e = 0.311\ (2.48) = 0.8$ ft

$h_S/D_{50} = 0.8/0.83 = 0.96$ and $h_S/D_{50} \geq 2$ is not satisfied. Although not available, try a riprap size that will yield h_S/D_{50} close to, but greater than 2. (A basin sized for smaller riprap may be lined with larger riprap.) Repeat step 2.

Step 2 (3rd iteration). Select a trial D_{50} and obtain h_s/y_e from Equation 10.1.

Try $D_{50} = 0.65$ ft; $D_{50}/y_e = 0.65/2.48 = 0.26$ (≥ 0.1 OK)

From Equation 10.1,

$$\frac{h_s}{y_e} = 0.86\left(\frac{D_{50}}{y_e}\right)^{-0.55}\left(\frac{V_o}{\sqrt{gy_e}}\right) - C_o = 0.86(0.26)^{-0.55}(1.22) - 1.62 = 0.581$$

$h_S = (h_S/y_e)y_e = 0.581\ (2.48) = 1.4$ ft

$h_S/D_{50} = 1.4/0.65 = 2.15$ and $h_S/D_{50} \geq 2$ is satisfied. Continue to step 3.

Step 3 (3rd iteration). Size the basin as shown in Figures 10.1 and 10.2.

$L_S = 10h_S = 10(1.4) = 14$ ft

L_S min $= 3W_o = 3(6) = 18$ ft, use $L_S = 18$ ft

$L_B = 15h_S = 15(1.4) = 21$ ft

L_B min = $4W_o$ = 4(6) = 24 ft, use L_B = 24 ft

$W_B = W_o + 2(L_B/3) = 6 + 2(24/3) = 22$ ft

However, since the trial D_{50} is not available, the next larger riprap size (D_{50} = 0.83 ft) would be used to line a basin with the given dimensions.

Step 4 (3rd iteration). Determine the basin exit depth, $y_B = y_c$ and exit velocity, $V_B = V_c$.

$Q^2/g = (A_c)^3/T_c = [y_c(W_B + zy_c)]^3/ (W_B + 2zy_c)$

$135^2/32.2 = 566 = [y_c(22 + 2y_c)]^3/ (22 + 4y_c)$

By trial and success, y_c = 1.02 ft, T_c =26.1 ft, A_c = 24.5 ft^2

$V_c = Q/A_c$ = 135/24.5 = 5.5 ft/s (acceptable)

Two feasible options have been identified. First, a 2.3-ft-deep, 23-ft-long pool, with an 11.5-ft-apron using D_{50} = 0.5 ft. Second, a 1.4-ft-deep, 18-ft-long pool, with a 6-ft-apron using D_{50} = 0.83 ft. The choice between these two options will likely depend on the available space and the cost of riprap.

Step 5. For the design discharge, determine if $TW/y_o \leq 0.75$

TW/y_o = 2.0/2.7 = 0.74, which satisfies $TW/y_o \leq 0.75$. No additional riprap needed.

10.2 RIPRAP APRON

The most commonly used device for outlet protection, primarily for culverts 1500 mm (60 in) or smaller, is a riprap apron. An example schematic of an apron taken from the Federal Lands Division of the Federal Highway Administration is shown in Figure 10.4.

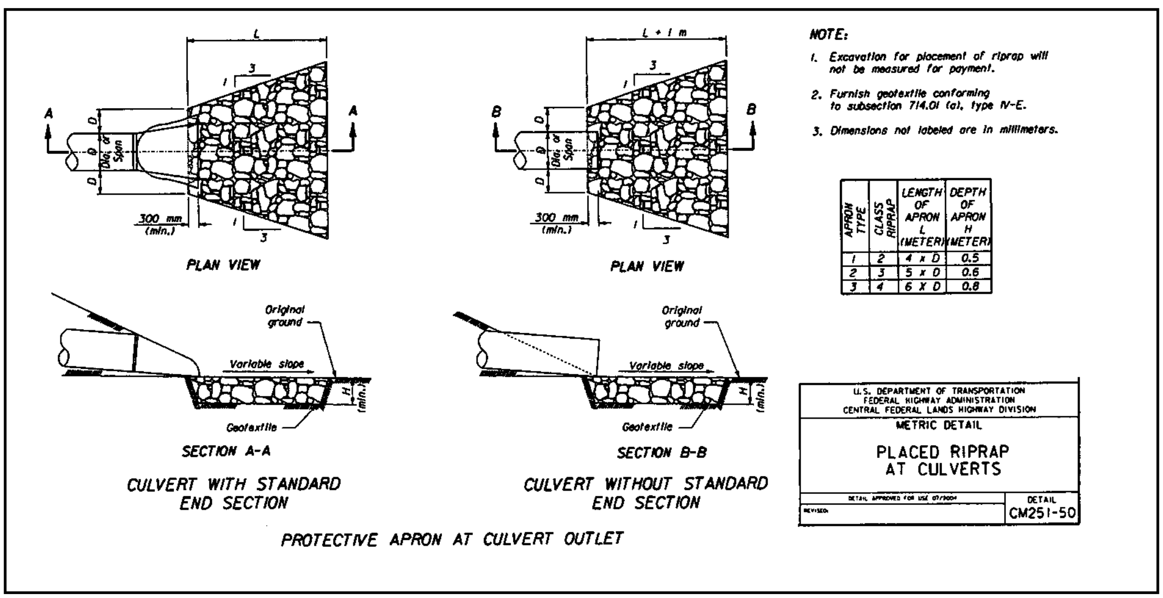


Figure 10.4. Placed Riprap at Culverts (Central Federal Lands Highway Division)

They are constructed of riprap or grouted riprap at a zero grade for a distance that is often related to the outlet pipe diameter. These aprons do not dissipate significant energy except through increased roughness for a short distance. However, they do serve to spread the flow helping to transition to the natural drainage way or to sheet flow where no natural drainage way exists. However, if they are too short, or otherwise ineffective, they simply move the location of potential erosion downstream. The key design elements of the riprap apron are the riprap size as well as the length, width, and depth of the apron.

Several relationships have been proposed for riprap sizing for culvert aprons and several of these are discussed in greater detail in Appendix D. The independent variables in these relationships include one or more of the following variables: outlet velocity, rock specific gravity, pipe dimension (e.g. diameter), outlet Froude number, and tailwater. The following equation (Fletcher and Grace, 1972) is recommended for circular culverts:

$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) \tag{10.4}$$

where,

- D_{50} = riprap size, m (ft)
- Q = design discharge, m^3/s (ft^3/s)
- D = culvert diameter (circular), m (ft)
- TW = tailwater depth, m (ft)
- g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

Tailwater depth for Equation 10.4 should be limited to between 0.4D and 1.0D. If tailwater is unknown, use 0.4D.

Whenever the flow is supercritical in the culvert, the culvert diameter is adjusted as follows:

$$D' = \frac{D + y_n}{2} \tag{10.5}$$

where,

- D' = adjusted culvert rise, m (ft)
- y_n = normal (supercritical) depth in the culvert, m (ft)

Equation 10.4 assumes that the rock specific gravity is 2.65. If the actual specific gravity differs significantly from this value, the D_{50} should be adjusted inversely to specific gravity.

The designer should calculate D_{50} using Equation 10.4 and compare with available riprap classes. A project or design standard can be developed such as the example from the Federal Highway Administration Federal Lands Highway Division (FHWA, 2003) shown in Table 10.1 (first two columns). The class of riprap to be specified is that which has a D_{50} greater than or equal to the required size. For projects with several riprap aprons, it is often cost effective to use fewer riprap classes to simplify acquiring and installing the riprap at multiple locations. In such a case, the designer must evaluate the tradeoffs between over sizing riprap at some locations in order to reduce the number of classes required on a project.

Table 10.1. Example Riprap Classes and Apron Dimensions

Class	D_{50} (mm)	D_{50} (in)	Apron Length[1]	Apron Depth
1	125	5	4D	$3.5D_{50}$
2	150	6	4D	$3.3D_{50}$
3	250	10	5D	$2.4D_{50}$
4	350	14	6D	$2.2D_{50}$
5	500	20	7D	$2.0D_{50}$
6	550	22	8D	$2.0D_{50}$

[1]D is the culvert rise.

The apron dimensions must also be specified. Table 10.1 provides guidance on the apron length and depth. Apron length is given as a function of the culvert rise and the riprap size. Apron depth ranges from $3.5D_{50}$ for the smallest riprap to a limit of $2.0D_{50}$ for the larger riprap sizes. The final dimension, width, may be determined using the 1:3 flare shown in Figure 10.4 and should conform to the dimensions of the downstream channel. A filter blanket should also be provided as described in HEC 11 (Brown and Clyde, 1989).

For tailwater conditions above the acceptable range for Equation 10.4 (TW > 1.0D), Figure 10.3 should be used to determine the velocity downstream of the culvert. The guidance in Section 10.3 may be used for sizing the riprap. The apron length is determined based on the allowable velocity and the location at which it occurs based on Figure 10.3.

Over their service life, riprap aprons experience a wide variety of flow and tailwater conditions. In addition, the relations summarized in Table 10.1 do not fully account for the many variables in culvert design. To ensure continued satisfactory operation, maintenance personnel should inspect them after major flood events. If repeated severe damage occurs, the location may be a candidate for extending the apron or another type of energy dissipator.

Design Example: Riprap Apron (SI)

Design a riprap apron for the following CMP installation. Available riprap classes are provided in Table 10.1. Given:

Q = 2.33 m^3/s
D = 1.5 m
TW = 0.5 m

Solution

Step 1. Calculate D_{50} from Equation 10.4. First verify that tailwater is within range.

TW/D = 0.5/1.5 = 0.33. This is less than 0.4D, therefore,

use TW = 0.4D = 0.4(1.5) = 0.6 m

$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) = 0.2\,(1.5)\left(\frac{2.33}{\sqrt{9.81}(1.5)^{2.5}}\right)^{4/3}\left(\frac{1.5}{0.6}\right) = 0.13\text{ m}$$

Step 2. Determine riprap class. From Table 10.1, riprap class 2 (D_{50} = 0.15 m) is required.

Step 3. Estimate apron dimensions.

From Table 10.1 for riprap class 2,

Length, L = 4D = 4(1.5) = 6 m

Depth = $3.3D_{50}$ = 3.3 (0.15) = 0.50 m

Width (at apron end) = 3D + (2/3)L = 3(1.5) + (2/3)(6) = 8.5 m

<u>Design Example: Riprap Apron (CU)</u>

Design a riprap apron for the following CMP installation. Available riprap classes are provided in Table 10.1. Given:

Q = 85 ft^3/s

D = 5.0 ft

TW = 1.6 ft

<u>Solution</u>

Step 1. Calculate D_{50} from Equation 10.4. First verify that tailwater is within range.

TW/D = 1.6/5.0 = 0.32. This is less than 0.4D, therefore,

use TW = 0.4D = 0.4(5) = 2.0 ft

$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) = 0.2\,(5.0)\left(\frac{85}{\sqrt{32.2}(5.0)^{2.5}}\right)^{4/3}\left(\frac{5.0}{2.0}\right) = 0.43\text{ ft} = 5.2\text{ in}$$

Step 2. Determine riprap class. From Table 10.1, riprap class 2 (D_{50} = 6 in) is required.

Step 3. Estimate apron dimensions.

From Table 10.1 for riprap class 2,

Length, L = 4D = 4(5) = 20 ft

Depth = $3.3D_{50}$ = 3.3 (6) = 19.8 in = 1.65 ft

Width (at apron end) = 3D + (2/3)L = 3(5) + (2/3)(20) = 28.3 ft

10.3 RIPRAP APRONS AFTER ENERGY DISSIPATORS

Some energy dissipators provide exit conditions, velocity and depth, near critical. This flow condition rapidly adjusts to the downstream or natural channel regime; however, critical velocity may be sufficient to cause erosion problems requiring protection adjacent to the energy dissipator. Equation 10.6 provides the riprap size recommended for use downstream of energy dissipators. This relationship is from Searcy (1967) and is the same equation used in HEC 11 (Brown and Clyde, 1989) for riprap protection around bridge piers.

$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) \tag{10.6}$$

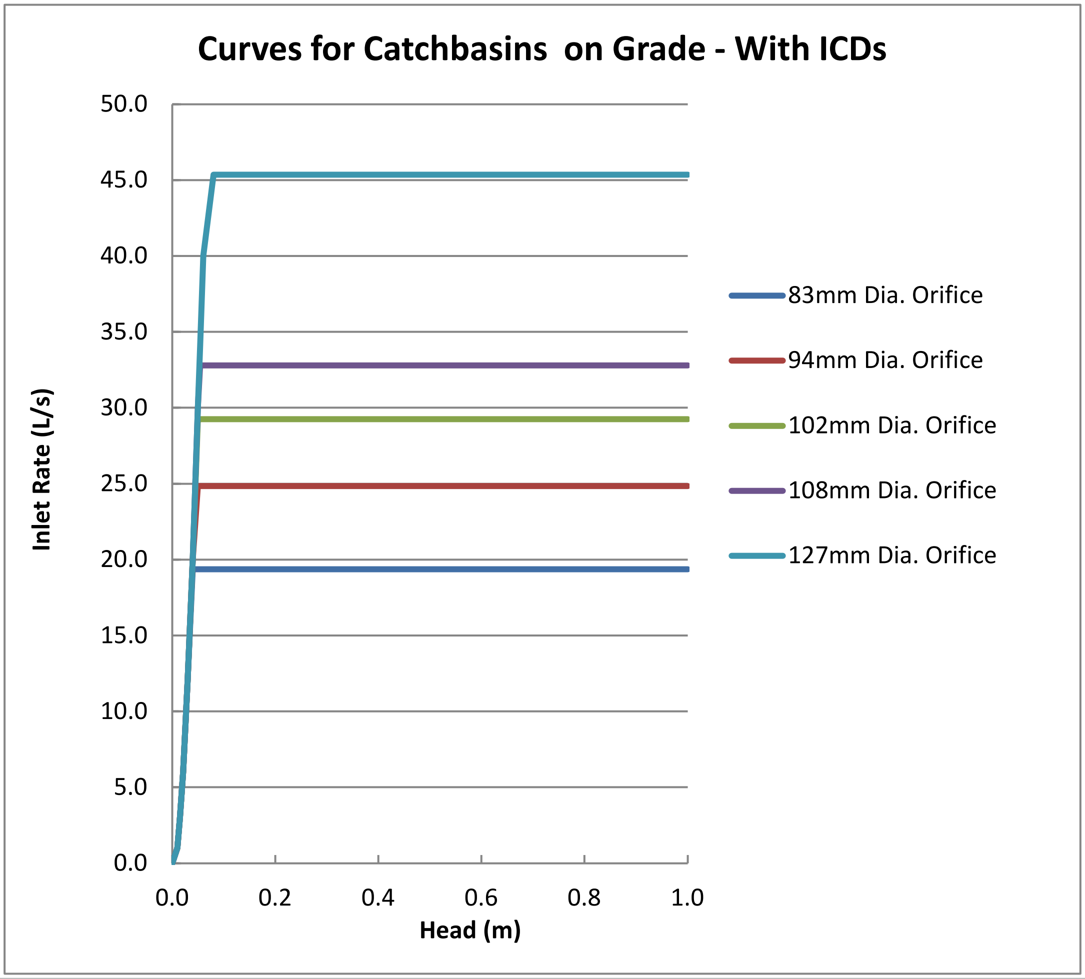
where,

D_{50} = median rock size, m (ft)

V = velocity at the exit of the dissipator, m/s (ft/s)

S = riprap specific gravity

The length of protection can be judged based on the magnitude of the exit velocity compared with the natural channel velocity. The greater this difference, the longer will be the length required for the exit flow to adjust to the natural channel condition. A filter blanket should also be provided as described in HEC 11 (Brown and Clyde, 1989).



Curb Inlet Catchbasins on Continuous Grade

Depth vs. Captured Flow Curve

A standard depth vs. captured flow curve for catch basins on a continuous grade was provided to Novatech by City staff for use in a dual-drainage model of an existing residential neighbourhood. This standard curve was derived using the inlet curves in Appendix 7A of the Ottawa Sewer Design Guidelines.

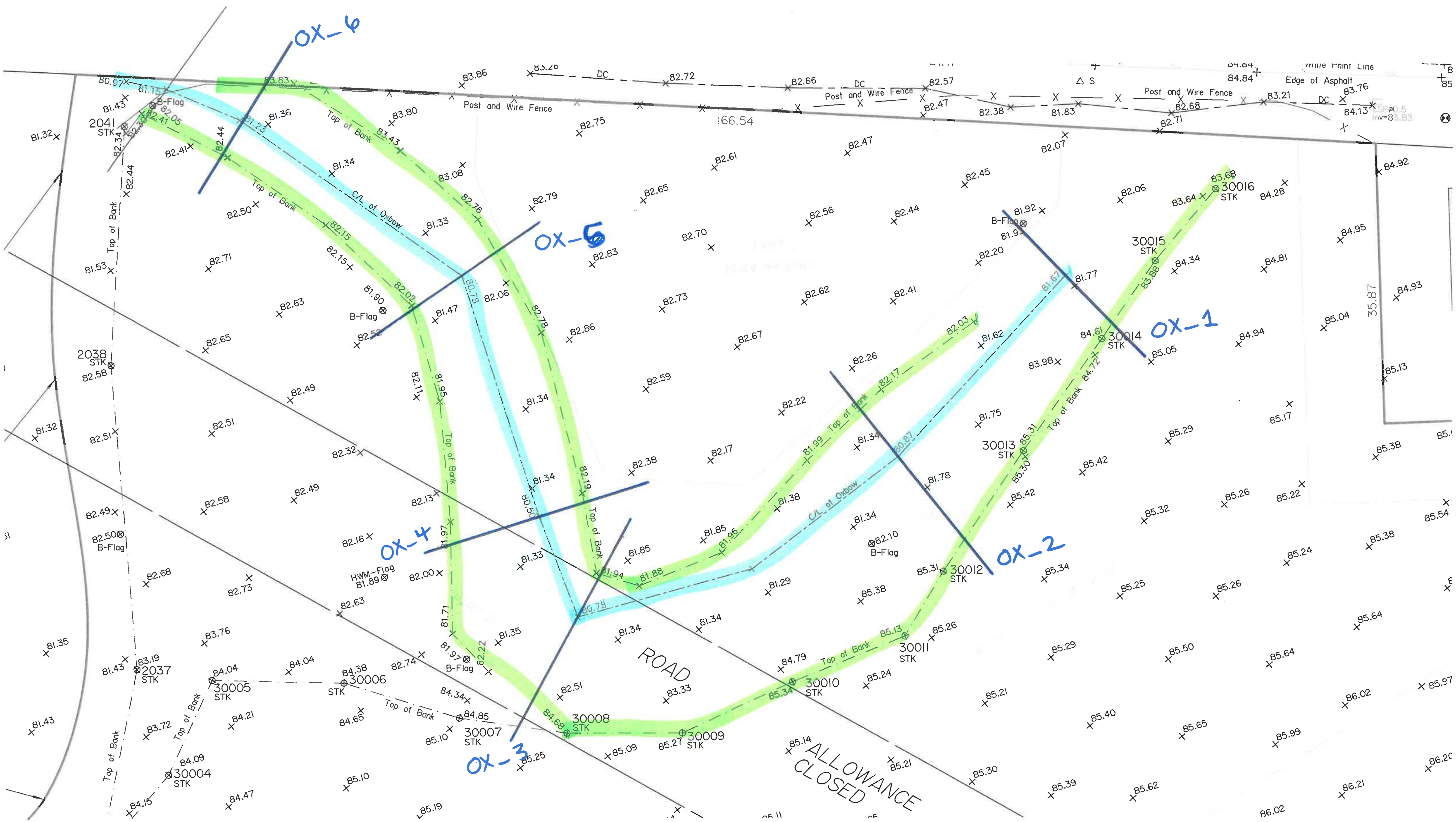
Novatech reviewed the methodology used to create this standard curve (described below) and determined that it was suitable for general use in other dual-drainage models.

- MTO Design Chart 4.04 provides the relationship between the gutter flow rate (Q_t) and flow spread (T) for Barrier Curb.
- MTO Design Chart 4.12 provides the relationship between flow spread (T) and flow depth (D).
- The relationship between the gutter flow rate (Q_t) and flow depth (D) was determined for different road slopes using the above charts and Manning's equation (refer to pages 58-60 of the MTO Drainage Management Manual – Part 2);
- The relationship between approach flow (Q_t) and captured flow (Q_c) was determined for different road slopes using the design chart for Barrier Curb with Gutter (Appendix 7-A.2).
- Using the above information, a family of curves was developed to characterize the relationship between flow depth and captured flow for curb inlet catchbasins on different road slopes. The results of this exercise can be summarized as follows:
    - For a given flow depth, the gutter flow rate (Q_t) increases as the road slope increases.
    - The capture efficiency (Q_c) of curb inlet catchbasins decrease as the road slope increases.
    - The net result is that the relationship between flow depth and capture rate is largely independent of road slope: While approach flow vs. captured flow (Q_t vs. Q_c) varies significantly with road grade, flow depth vs. captured flow (D vs. Q_c) does not.

Since there was very little difference in the flow depth vs. captured flow curves for different road slopes, this family of curves was averaged to create a single standard curve for use in dual-drainage models.

Inlet Control Devices

The standard depth vs. capture flow curve was modified to account for the installation of ICDs in curb inlet catchbasins on continuous grade. Separate inlet curves were created for each standard ICD orifice size by capping the inlet rate on the depth vs. capture flow curve at the maximum flow rate through the ICD at a head of 1.2m (depth from centerline of CB lead to top of CICB frame).



oxbow ditch x-sections for PCSWMM.

<u>Plunge Pool Calculations</u>

Reference calculations are from the FHWA Hydraulic Design of Energy Dissipators for Culverts and Channels, Chapter 10: Riprap Basins and Aprons. Section 10 has been provided following these calculations.

Preliminary calculations for the sizing of the basin follow the recommendations outlined in Section 10.1 and as referencing Figures 10.1 and 10.2 as follows:

- The basin is pre-shaped and lined with riprap approximately $2D_{50}$ thick.
  - 300mm riprap has been selected, so D_{50} is 150mm. Proposed thickness of the basin is 600mm, which exceeds this recommendation.
- The riprap floor is constricted at the approximate depth of scour, h_S, that would occur in a thick pad of riprap. The h_S/D_{50} of the material should be greater than 2.
  - Plunge pool is designed to have a depth of 350mm, this gives h_S/D_{50} of >2.
- The length of the energy dissipating pool, Ls, is $10h_S$, but no less than $3W_O$; the length of the apron, L_A, is $5h_S$, but no less than W_O. The overall length of the basin (pool plus apron), L_B, is $15h_S$, but no less than $4W_O$.
  - For the energy dissipating pool:
    - $10h_S$ = 10*0.60m = 6.0 m, or $3W_O$ = 3*1.2m = 3.6m minimum
    - Designed L_S is 5.7m, which is > $3W_O$ and just 0.3m shy of $10h_S$.
  - Length of the apron:
    - $L_A = 5h_S$ = 5*0.60m = 1.75m, which is > W_O
  - Overall length of the basin:
    - 15hS = 15*0.35m = 5.25m, which is > $4W_O$
    - Actual overall length of the basin is 7.45m
- A riprap cutoff wall or sloping apron can be constricted if downstream channel degradation is anticipated as shown in Figure 10.1.

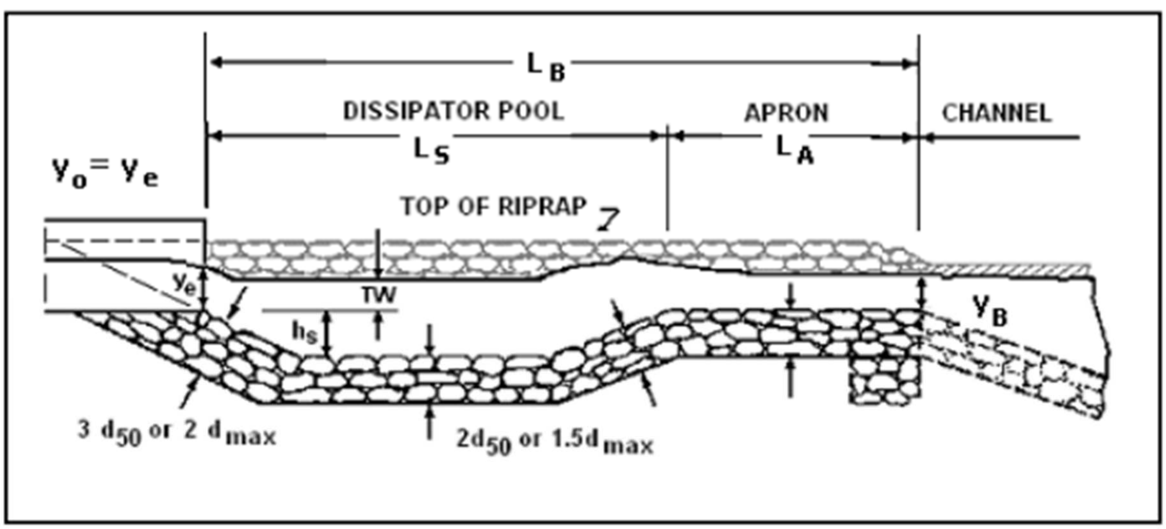


Figure 10.1. Profile of Riprap Basin

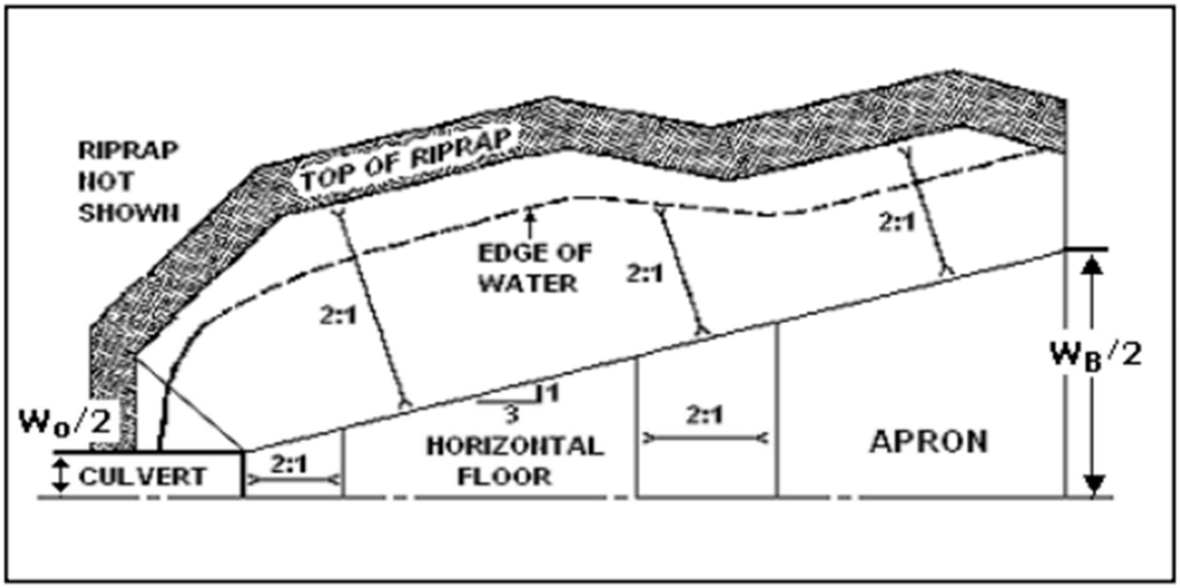


Figure 10.2. Half Plan of Riprap Basin

Using the proposed plunge pool cross-sectional dimensions, the outlet velocity from the maximum outlet peak flow (100-year) has been calculated using V=Q/A

Cross-sectional area calculated using the equation for the area of a trapezoid:

$$A = \left(\frac{W_T + W_B}{2}\right) * D$$

$$A = \left(\frac{3.87 + 10.57}{2}\right) * 0.35$$

$$A = 2.53m^3$$

Using the 100-year combined peak flow entering the plunge pool (2.3cms)

$$V = \frac{Q}{A}$$

$$V = \frac{2.3cms}{2.53m^3}$$

$$V = 0.91m/s$$

Stinson Lands

Post-Development Model Parameters

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Area ID	Catchment Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.200	0.70	71%	0%	38.18	52.38	1.0%
B-01	0.740	0.45	36%	100%	21.26	348.14	1.0%
C-01	0.280	0.70	71%	45%	23.78	117.77	1.0%
C-02	0.330	0.70	71%	45%	24.82	132.96	1.0%
C-03	0.280	0.70	71%	45%	23.62	118.54	1.0%
C-04	0.310	0.70	71%	45%	22.83	135.79	1.0%
C-05	0.240	0.70	71%	45%	23.31	102.96	1.0%
C-06	0.340	0.70	71%	45%	34.45	98.70	1.0%
C-07	0.590	0.45	36%	100%	39.87	148.00	1.0%
C-08	0.270	0.70	71%	45%	22.14	121.97	1.0%
C-09	0.230	0.70	71%	45%	22.42	102.59	1.0%
C-10	0.220	0.70	71%	45%	22.45	98.00	1.0%
C-11	0.650	0.45	36%	100%	19.05	341.19	1.0%
C-12	0.330	0.70	71%	45%	19.77	166.94	1.0%
C-13	0.290	0.70	71%	45%	23.66	122.59	1.0%
C-14	0.560	0.45	36%	100%	14.10	397.05	1.0%
C-15	0.340	0.70	71%	45%	22.21	153.06	1.0%
C-16	0.350	0.70	71%	45%	21.78	160.71	1.0%
C-17	0.320	0.70	71%	45%	22.95	139.43	1.0%
C-18	0.400	0.45	36%	100%	13.81	289.75	1.0%
C-19	0.170	0.70	71%	45%	19.18	88.63	1.0%
C-20	0.210	0.70	71%	45%	18.84	111.49	1.0%
C-21	0.040	0.45	36%	100%	45.28	8.83	1.0%
D-01	0.180	0.20	0%	0%	20.51	87.76	1.0%
TOTAL:	**7.87**						
		0.59	56%				

Project Name
Overall Model Schematic

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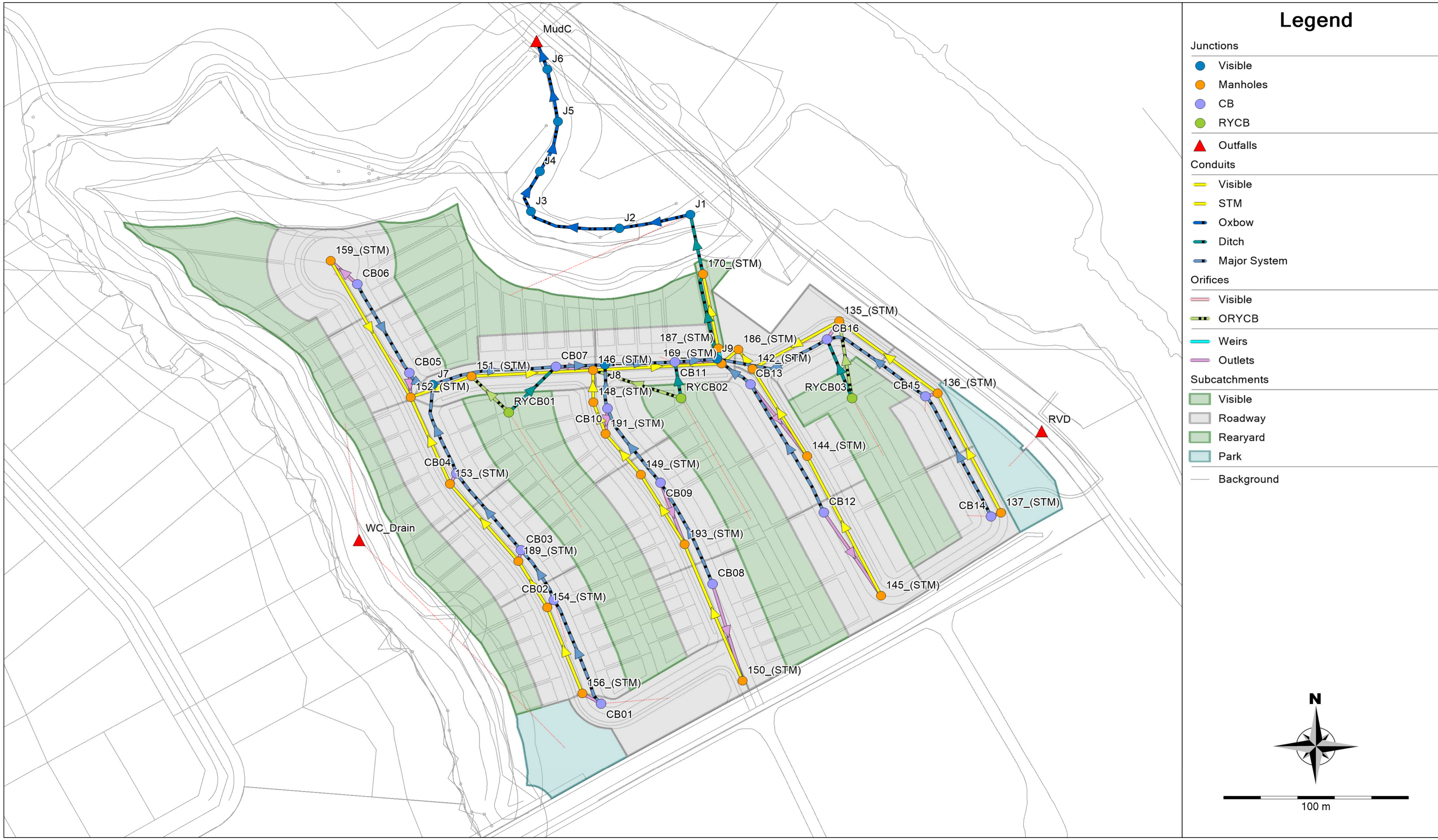

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PREPARED BY: NOVATECH

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Stinson Lands
Pre-Development Model Parameters

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Time to Peak Calculations
(Uplands Overland Flow Method)
Existing Conditions

		Overland Flow						Concentrated Overland Flow						Overall				Flow Length	Slope
Area ID	Area (ha)	Length (m)	Elevation U/S (m)	Elevation D/S (m)	Slope (%)	Velocity (m/s)	Travel Time (min)	Length (m)	Elevation U/S (m)	Elevation D/S (m)	Slope (%)	Velocity (m/s)	Travel Time (min)	Time of Concentration (min)	Time to Peak (min)	Time to Peak (min)	Time to Peak (hrs)		
A1	2.717	100	94	89	5.0%	0.33	5.05	150	89	88	0.5%	0.19	13.16	18	12	12	0.20	250	2%
A2	0.444	40	88	88	0.7%	0.14	4.76	0	0	0	0.0%	0	0.00	5	3	10	0.17	40	1%
B1	1.101	80	88	85	4.1%	0.3	4.44	0	0	0	0.0%	0	0.00	4	3	10	0.17	80	4%
C1	2.298	100	88	86	2.0%	0.21	7.94	25	86	86	2.0%	0.4	1.04	9	6	10	0.17	125	2%
D1	1.273	100	94	89	5.0%	0.33	5.05	70	89	86	4.3%	0.57	2.05	7	5	10	0.17	170	5%
TOTAL:	**7.83**																		

Weighted Curve Number Calculations
Soil type Silty Clay = D

Area ID	Land Use 1	Area	CN	Land Use 2	Area	CN	Land Use 3	Area	CN	Weighted CN
A1	Building & Road	4%	86	Tree Farm	1%	82	Row Crops	95%	89	89
A2	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
B1	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
C1	Building & Road	0%	86	Tree Farm	0%	82	Row Crops	100%	89	89
D1	Building & Road	12%	86	Tree Farm	28%	82	Row Crops	60%	89	87

Weighted IA Calculations

Area ID	Land Use 1	Area	S	IA	Land Use 2	Area	S	IA	Land Use 3	Area	S	IA	Weighted IA
A1	Building & Roads	4%	41.35	6.20	Tree Farm	1%	55.76	8.36	Row Crops	95%	31.39	6.28	6.32
A2	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
B1	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
C1	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
D1	Building & Roads	28%	41.35	6.20	Tree Farm	12%	55.76	8.36	Row Crops	60%	31.39	6.28	6.51

Project Name
Pre-Development Model Schematic

NOVATECH
Engineers, Planners & Landscape Architects

8/2/2022
PREPARED BY: NOVATECH

M:\2021\121153\DATA\Calculations\Sewer Calcs\SWM\PCSWMM Models\Draft Plan\121153-ModelParams_DP.xlsx

Stinson Lands
Design Storm Time Series Data
SCS Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

S2-12.stm		S5-12.stm		S100-12.stm	
Duration	Intensity	Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr	min	mm/hr
0:00	0.00	0:00	0	0:00	0
0:30	1.27	0:30	1.69	0:30	2.82
1:00	0.59	1:00	0.79	1:00	1.31
1:30	1.10	1:30	1.46	1:30	2.44
2:00	1.10	2:00	1.46	2:00	2.44
2:30	1.44	2:30	1.91	2:30	3.19
3:00	1.27	3:00	1.69	3:00	2.82
3:30	1.69	3:30	2.25	3:30	3.76
4:00	1.69	4:00	2.25	4:00	3.76
4:30	2.29	4:30	3.03	4:30	5.07
5:00	2.88	5:00	3.82	5:00	6.39
5:30	4.57	5:30	6.07	5:30	10.14
6:00	36.24	6:00	48.08	6:00	80.38
6:30	9.23	6:30	12.25	6:30	20.47
7:00	4.06	7:00	5.39	7:00	9.01
7:30	2.71	7:30	3.59	7:30	6.01
8:00	2.37	8:00	3.15	8:00	5.26
8:30	1.86	8:30	2.47	8:30	4.13
9:00	1.95	9:00	2.58	9:00	4.32
9:30	1.27	9:30	1.69	9:30	2.82
10:00	1.02	10:00	1.35	10:00	2.25
10:30	1.44	10:30	1.91	10:30	3.19
11:00	0.93	11:00	1.24	11:00	2.07
11:30	0.85	11:30	1.12	11:30	1.88
12:00	0.85	12:00	1.12	12:00	1.88

Stinson Lands
Design Storm Time Series Data
SCS Design Storms

NOVATECH
Engineers, Planners & Landscape Architects

S2-24.stm		S5-24.stm		S100-24.stm	
Duration	Intensity	Duration	Intensity	Duration	Intensity
min	mm/hr	min	mm/hr	min	mm/hr
0:00	0.00	0:00	0	0:00	0
1:00	0.72	1:00	0.44	1:00	0.6
2:00	0.34	2:00	0.44	2:00	0.75
3:00	0.63	3:00	0.81	3:00	1.39
4:00	0.63	4:00	0.81	4:00	1.39
5:00	0.81	5:00	1.06	5:00	1.81
6:00	0.72	6:00	0.94	6:00	1.6
7:00	0.96	7:00	1.25	7:00	2.13
8:00	0.96	8:00	1.25	8:00	2.13
9:00	1.30	9:00	1.68	9:00	2.88
10:00	1.63	10:00	2.12	10:00	3.63
11:00	2.59	11:00	3.37	11:00	5.76
12:00	20.55	12:00	26.71	12:00	45.69
13:00	5.23	13:00	6.8	13:00	11.64
14:00	2.30	14:00	2.99	14:00	5.12
15:00	1.54	15:00	2	15:00	3.42
16:00	1.34	16:00	1.75	16:00	2.99
17:00	1.06	17:00	1.37	17:00	2.35
18:00	1.11	18:00	1.44	18:00	2.46
19:00	0.72	19:00	0.94	19:00	1.6
20:00	0.58	20:00	0.75	20:00	1.28
21:00	0.81	21:00	1.06	21:00	1.81
22:00	0.53	22:00	0.68	22:00	1.17
23:00	0.48	23:00	0.63	23:00	1.07
0:00	0.48	0:00	0.63	0:00	1.07

VORTECHS SYSTEM® ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS
STINSON SUBDIVISION (4386 RIDEAU VALLEY DRIVE)
OTTAWA, ON
MODEL PC1421 OFF-LINE

CONTECH ENGINEERED SOLUTIONS

Design Ratio[1] = $\dfrac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(14.3 \text{ m2})}$ **= 0.79**

Rainfall Intensity mm/hr	Operating Rate[2] % of capacity	Flow Treated (l/s)	% Total Rainfall Volume[3]	Rmvl. Effcy[4] (%)	Rel. Effcy (%)
0.5	0.6	5.8	9.2%	100.0%	9.2%
1.0	1.2	11.5	10.6%	98.0%	10.4%
1.5	1.7	17.3	9.9%	98.0%	9.7%
2.0	2.3	23.0	8.4%	98.0%	8.2%
2.5	2.9	28.8	7.7%	98.0%	7.5%
3.0	3.5	34.5	5.9%	98.0%	5.8%
3.5	4.1	40.3	4.4%	98.0%	4.3%
4.0	4.6	46.0	4.7%	98.0%	4.6%
4.5	5.2	51.8	3.3%	98.0%	3.3%
5.0	5.8	57.6	3.0%	98.0%	3.0%
6.0	7.0	69.1	5.4%	98.0%	5.3%
7.0	8.1	80.6	4.4%	96.9%	4.2%
8.0	9.3	92.1	3.5%	96.3%	3.4%
9.0	10.5	103.6	2.8%	96.0%	2.7%
10.0	11.6	115.1	2.2%	95.3%	2.1%
15.0	17.4	172.7	7.0%	89.9%	6.3%
20.0	23.2	230.2	4.5%	85.7%	3.9%
25.0	29.0	287.8	1.4%	82.6%	1.2%
30.0	34.8	345.4	0.7%	80.0%	0.5%
35.0	40.6	402.9	0.5%	76.0%	0.4%
40.0	46.5	460.5	0.5%	69.0%	0.4%
					96.2%

Predicted Annual Runoff Volume Treated =	93.5%
Assumed Removal Efficiency of remaining % =	0.0%
Removal Efficiency Adjustment[5] =	6.5%
Predicted Net Annual Load Removal Efficiency =	**90%**

1 - Design Ratio = (Total Drainage Area) x (Runoff Coefficient) x (Rational Method Conversion) / Grit Chamber Area
- The Total Drainage Area and Runoff Coefficient are specified by the site engineer.
- The rational method conversion based on the units in the above equation is 2.775.

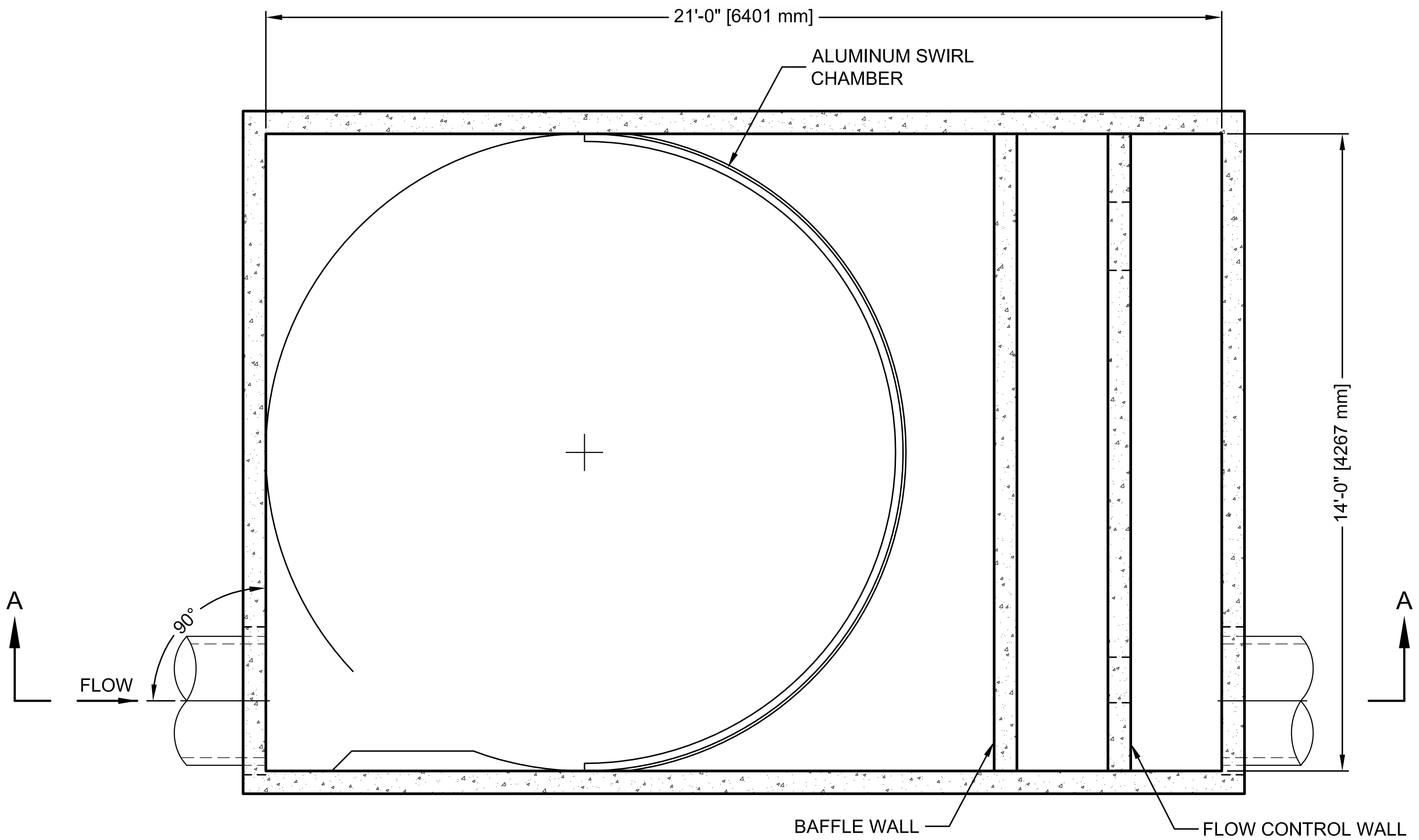
2 - Operating Rate (% of capacity) = percentage of peak operating rate of 68 l/s/m^2.
3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON
4 - Based on Contech Construction Products laboratory verified removal of an average particle size of 80 microns (see Technical Bulletin #1).
5- Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.

Calculated by:	JAK	7/26	Checked by:

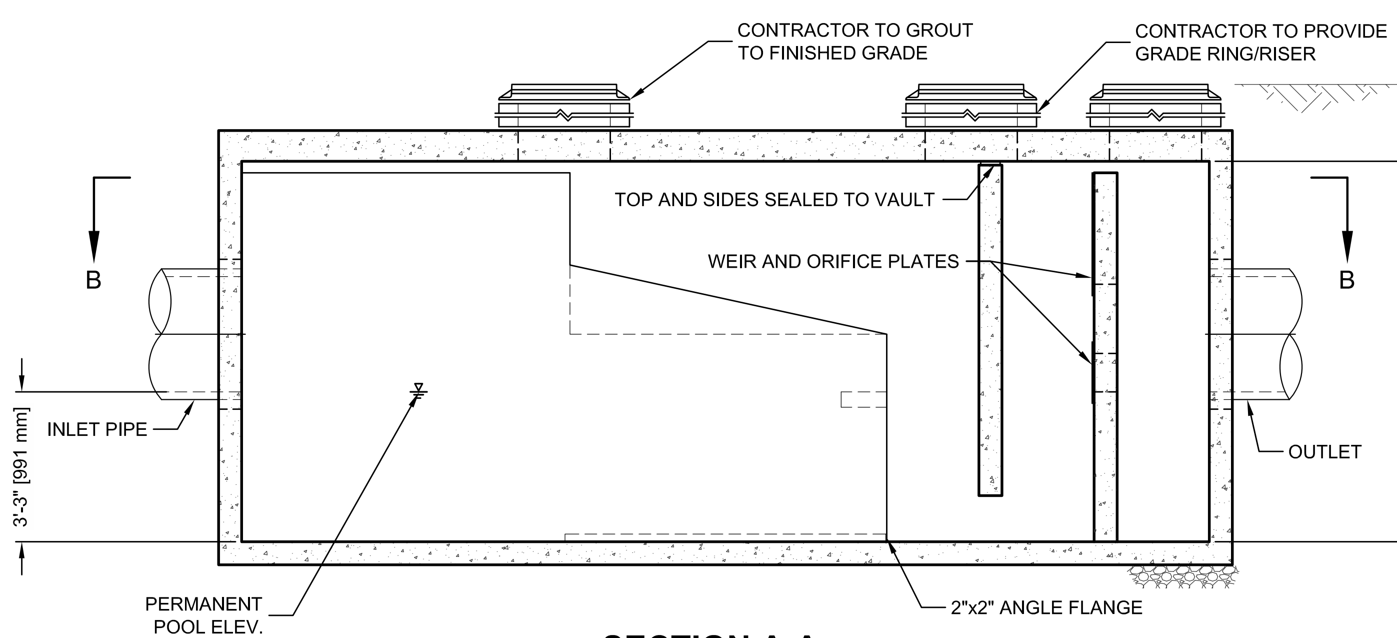
VORTECHS PC1421 DESIGN NOTES

VORTECHS PC1421 RATED TREATMENT CAPACITY IS 34 CFS, OR PER LOCAL REGULATIONS. IF THE SITE CONDITIONS EXCEED RATED TREATMENT CAPACITY, AN UPSTREAM BYPASS STRUCTURE IS REQUIRED.

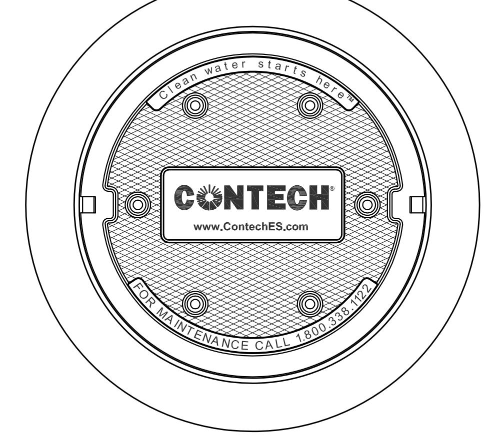
THE STANDARD INLET/OUTLET CONFIGURATION IS SHOWN. FOR OTHER CONFIGURATION OPTIONS , PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS, LLC REPRESENTATIVE. www.ContechES.com



SECTION B-B



SECTION A-A



FRAME AND COVER
(DIAMETER VARIES)
N.T.S.

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID			*
WATER QUALITY FLOW RATE (CFS)			*
PEAK FLOW RATE (CFS)			*
RETURN PERIOD OF PEAK FLOW (YRS)			*
PIPE DATA:	I.E.	MATERIAL	DIAMETER
INLET PIPE 1	*	*	*
INLET PIPE 2	*	*	*
OUTLET PIPE	*	*	*
RIM ELEVATION			*
ANTI-FLOTATION BALLAST		WIDTH	HEIGHT
		*	*
NOTES/SPECIAL REQUIREMENTS:			

* PER ENGINEER OF RECORD

GENERAL NOTES
1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHT, PLEASE CONTACT YOUR CONTECH ENGINEERED SOLUTIONS, LLC REPRESENTATIVE. www.ContechES.com
4. VORTECHS WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING.
5. STRUCTURE SHALL MEET AASHTO HS20 AND CASTINGS SHALL MEET AASHTO M306 LOAD RATING, ASSUMING GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION.
6. INLET PIPE(S) MUST BE PERPEDICULAR TO THE VAULT AND AT THE CORNER TO INTRODUCE THE FLOW TANGENTIALLY TO THE SWIRL CHAMBER. DUAL INLETS NOT TO HAVE OPPOSING TANGENTIAL FLOW DIRECTIONS.
7. OUTLET PIPE(S) MUST BE DOWN STREAM OF THE FLOW CONTROL BAFFLE AND MAY BE LOCATED ON THE SIDE OR END OF THE VAULT. THE FLOW CONTROL WALL MAY BE TURNED TO ACCOMODATE OUTLET PIPE KNOCKOUTS ON THE SIDE OF THE VAULT.

INSTALLATION NOTES
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE VORTECHS STRUCTURE (LIFTING CLUTCHES PROVIDED).
C. CONTRACTOR TO INSTALL JOINT SEALANT BETWEEN ALL STRUCTURE SECTIONS AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES. MATCH PIPE INVERTS WITH ELEVATIONS SHOWN.
E. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO ASSURE UNIT IS WATER TIGHT, HOLDING WATER TO FLOWLINE INVERT MINIMUM. IT IS SUGGESTED THAT ALL JOINTS BELOW PIPE INVERTS ARE GROUTED.

Vortechs®

THIS PRODUCT MAY BE PROTECTED BY THE FOLLOWING U.S. PATENT: 5,759,415; RELATED FOREIGN PATENTS.

CONTECH®
ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

VORTECHS PC1421
STANDARD DETAIL

VORTECHS SYSTEM® ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS
Stinson Subdivision (4386 Rideau Valley Drive)
Ottawa, ON
Model 1522CIP In-line

CONTECH ENGINEERED SOLUTIONS

Design Ratio[1] = $\dfrac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(16.4 \text{ m2})}$ **= 0.69**

Rainfall Intensity mm/hr	Operating Rate[2] % of capacity	Flow Treated (l/s)	% Total Rainfall Volume[3]	Rmvl. Effcy[4] (%)	Rel. Effcy (%)
0.5	0.5	5.6	9.2%	98.0%	9.0%
1.0	1.0	11.2	10.6%	98.0%	10.4%
1.5	1.5	16.8	9.9%	98.0%	9.7%
2.0	2.0	22.4	8.4%	98.0%	8.2%
2.5	2.5	27.9	7.7%	98.0%	7.5%
3.0	3.0	33.5	5.9%	97.9%	5.8%
3.5	3.5	39.1	4.4%	97.9%	4.3%
4.0	4.0	44.7	4.7%	97.1%	4.5%
4.5	4.6	50.3	3.3%	97.1%	3.2%
5.0	5.1	55.9	3.0%	96.3%	2.9%
6.0	6.1	67.1	5.4%	95.6%	5.1%
7.0	7.1	78.2	4.4%	95.0%	4.1%
8.0	8.1	89.4	3.5%	93.7%	3.3%
9.0	9.1	100.6	2.8%	92.6%	2.6%
10.0	10.1	111.8	2.2%	91.9%	2.0%
15.0	15.2	167.6	7.0%	86.7%	6.1%
20.0	20.2	223.5	4.5%	81.4%	3.7%
25.0	25.3	279.4	1.4%	77.0%	1.1%
30.0	30.4	335.3	0.7%	73.1%	0.5%
35.0	35.4	391.1	0.5%	69.7%	0.3%
					94.5%
				Predicted Annual Runoff Volume Treated =	99.5%
				Assumed Removal Efficiency of remaining % =	0.0%
				Removal Efficiency Adjustment[5] =	0.0%
				Predicted Net Annual Load Removal Efficiency =	**94%**

1 - Design Ratio = (Total Drainage Area) x (Runoff Coefficient) x (Rational Method Conversion) / Grit Chamber Area
- The Total Drainage Area and Runoff Coefficient are specified by the site engineer.
- The rational method conversion based on the units in the above equation is 2.775.

2 - Operating Rate (% of capacity) = percentage of peak operating rate of 68 l/s/m^2.

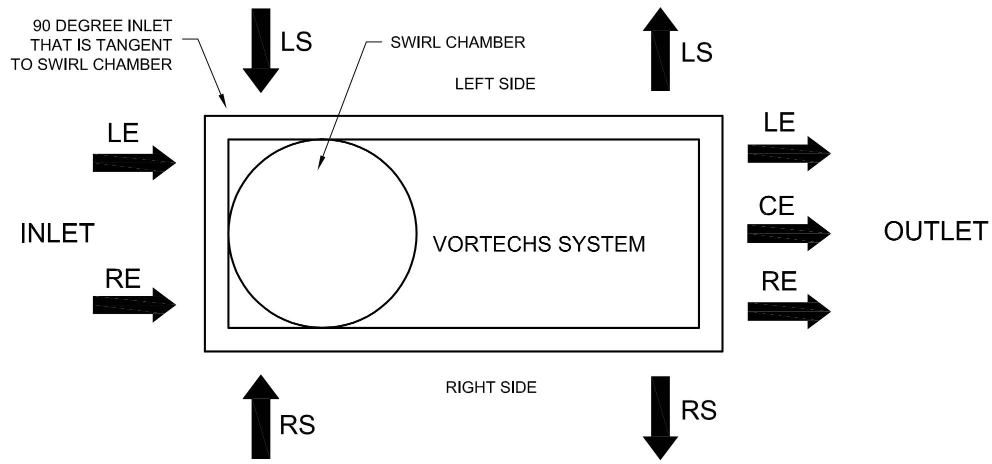
3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON

4 - Based on Contech Stormwater Solutions laboratory verified removal of an average particle size of 80 microns (see Technical Bulletin #1).

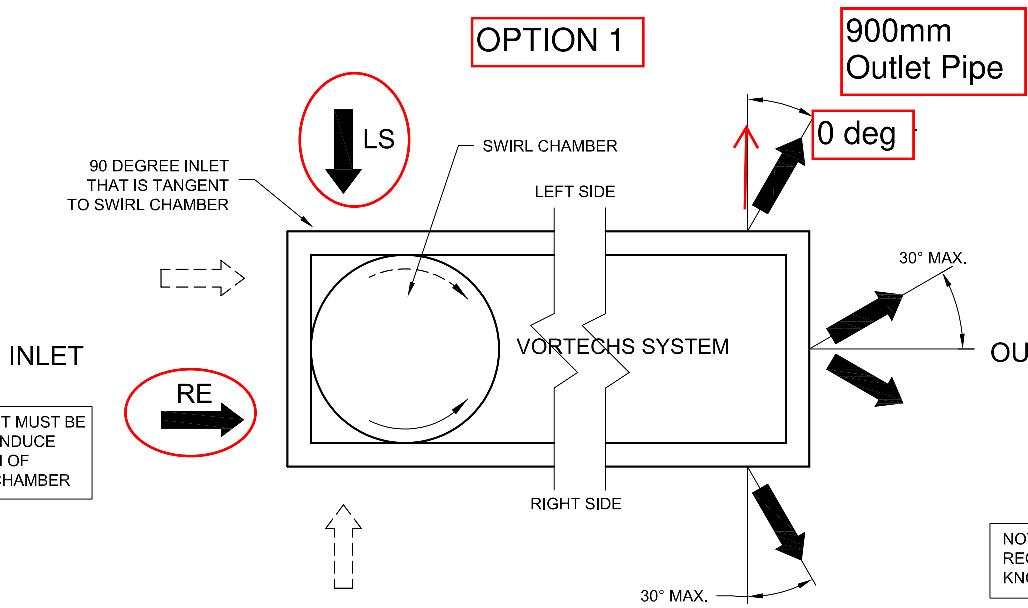
5- Increase due to comparison of flows based on historical rational rainfall method and actual modeled by specifying engineer.

Calculated by: JAK 8/1/2022	Checked by:

NOTE: INLET PIPE MUST BE PERPENDICULAR TO WALL IT IS ENTERING ON



TYPICAL INLET/OUTLET ORIENTATION OPTIONS



NOTE: DUAL INLET MUST BE POSITIONED TO INDUCE SAME DIRECTION OF FLOW IN SWIRL CHAMBER

NOTE: ANGLED OUTLET REQUIRES ELLIPTICAL KNOCKOUT .

DUAL INLET ORIENTATION OPTIONS

ANGLED OUTLET ORIENTATION OPTIONS

ORIENTATION KEY

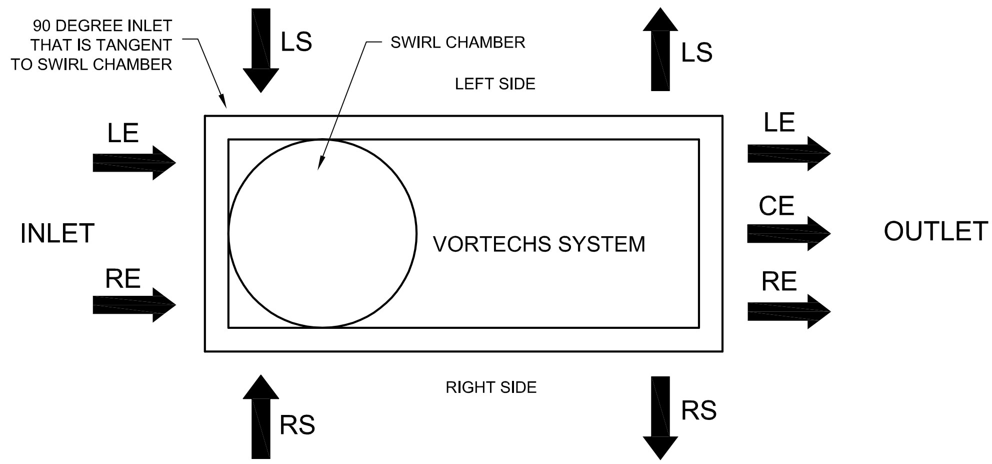
LE = LEFT END
RE = RIGHT END
LS = LEFT SIDE
RS = RIGHT SIDE
CE = CENTER END

CONTECH® STORMWATER SOLUTIONS INC.
contechstormwater.com

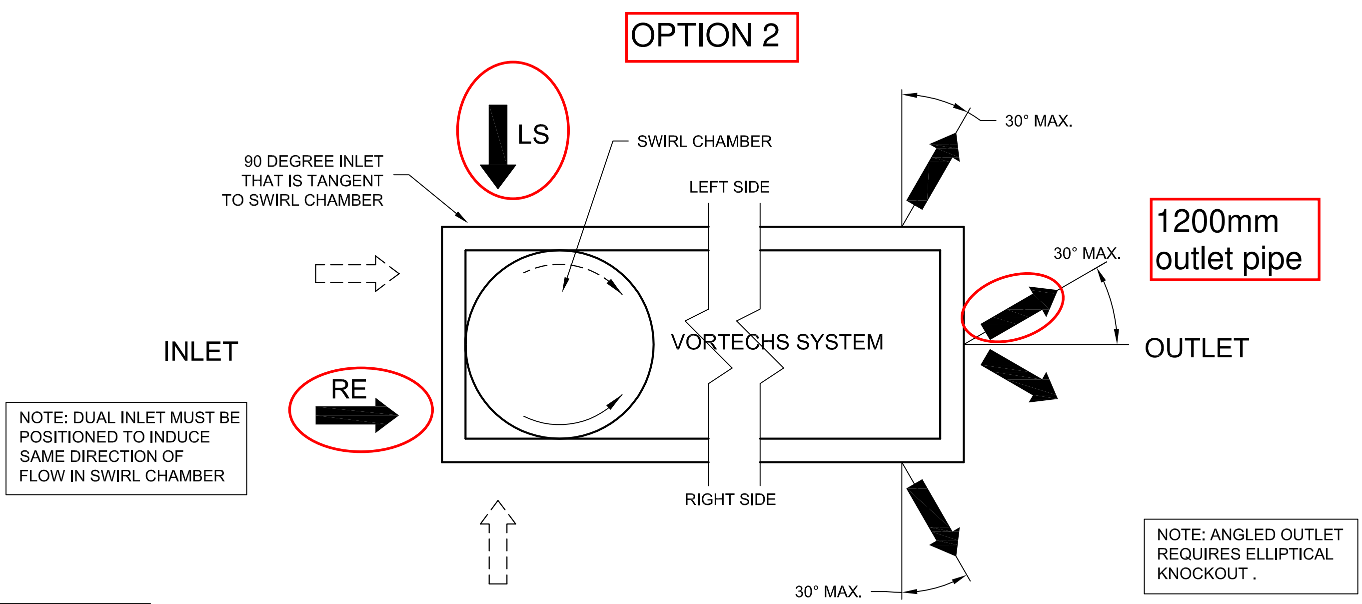
TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06	SCALE: NONE	FILE NAME: TYPVX ORIENTATION	DRAWN: GMC	CHECKED: NDG

NOTE: INLET PIPE MUST BE PERPENDICULAR TO WALL IT IS ENTERING ON



TYPICAL INLET/OUTLET ORIENTATION OPTIONS



DUAL INLET ORIENTATION OPTIONS

ANGLED OUTLET ORIENTATION OPTIONS

ORIENTATION KEY

LE = LEFT END
RE = RIGHT END
LS = LEFT SIDE
RS = RIGHT SIDE
CE = CENTER END

CONTECH® STORMWATER SOLUTIONS INC.

contechstormwater.com

TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06	SCALE: NONE	FILE NAME: TYPVX ORIENTATION	DRAWN: GMC	CHECKED: NDG

Appendix D
Sanitary Sewer Design Sheets and Sanitary Calculations

NOVATECH
Engineers, Planners & Landscape Architects

SANITARY SEWER DESIGN SHEET (FUTURE GROWTH)

Novatech Project #:	121153	Legend:	PROJECT SPECIFIC INFO
Project Name:	Stinson Lands Subdivision		USER DESIGN INPUT
Date Prepared:	1/11/2023		CUMILATIVE CELL
Date Revised:	1/11/2023		CALCULATED DESIGN CELL OUTPUT
Input By:	Brendan Rundle		CALCULATED ANNUAL CELL OUTPUT
Reviewed By:	Sam Bahia		CALCULATED RARE CELL OUTPUT
Drawing Reference:	121153-GP		USER AS-BUILT INPUT

LOCATION				DEMAND												
				RESIDENTIAL FLOW												
STREET	AREA	FROM MH	TO MH	SINGLES	SEMIS/ TOWNS	APARTS	PARK AREA (ha)	POPULATION (in 1000's)	CUMULATIVE POPULATION (in 1000's)	PEAK FACTOR M	AVG POPULATION FLOW Q(q) (L/s)	PEAKED DESIGN POP FLOW Q(p) (L/s)	PEAK ANNUAL/RARE FACTOR M	PEAKED ANNUAL/RARE POP FLOW Q(AR - Res) (L/s)	RESIDENTIAL DRAINAGE AREA (ha.)	CUMULATIVE RES DRAINAGE AREA (ha.)
Street 1	1	1	3	5	10			0.044	0.044	3.66	0.14	0.52	3.00	0.31	0.650	0.650
Street 1	2	3	5	1	11			0.033	0.077	3.62	0.25	0.90	2.96	0.53	0.370	1.020
Street 1	3	5	7	2	3			0.015	0.092	3.60	0.30	1.07	2.95	0.63	0.210	1.230
Street 1	4	7	9	3				0.010	0.102	3.59	0.33	1.19	2.94	0.70	0.200	1.430
Street 1	5	9	11	1				0.003	0.106	3.59	0.34	1.23	2.94	0.72	0.100	1.530
Street 2	6	13	15	9				0.031	0.031	3.68	0.10	0.37	3.01	0.21	0.890	0.890
Street 2	7	15	17	5				0.017	0.048	3.66	0.15	0.56	2.99	0.33	0.290	1.180
Street 2	8	17	19	8				0.027	0.075	3.62	0.24	0.88	2.97	0.51	0.440	1.620
Street 2	9	19	21	7				0.024	0.099	3.60	0.32	1.15	2.95	0.67	0.390	2.010
Street 2	10	23	21	15				0.051	0.051	3.65	0.17	0.60	2.99	0.35	1.280	1.280
Street 3	11	21	25					0.000	0.150	3.55	0.48	1.72	2.91	1.01	0.080	3.370
Street 3	12	25	11	4	2			0.019	0.169	3.54	0.55	1.93	2.90	1.13	0.370	3.740
Street 3	13	11	27	2	5			0.020	0.295	3.47	0.95	3.31	2.85	1.94	0.400	5.670
Street 3	14	27	29		1			0.003	0.297	3.46	0.96	3.34	2.85	1.96	0.120	5.790
Street 3	15	33	31		25			0.068	0.068	3.63	0.22	0.79	2.97	0.46	0.760	0.760
Street 3	16	31	29		10			0.027	0.095	3.60	0.31	1.10	2.95	0.65	0.300	1.060
Street 3	17	29	35		6			0.016	0.408	3.41	1.32	4.51	2.81	2.65	0.280	7.130
Street 3	18	35	37					0.000	0.408	3.41	1.32	4.51	2.81	2.65	0.030	7.160
Street 3	18	41	39		12			0.032	0.032	3.68	0.11	0.39	3.01	0.23	0.660	0.660
Street 3	20	39	37					0.000	0.032	3.68	0.11	0.39	3.01	0.23	0.060	0.720
Offsite		39	Ex MH					0.000	0.440	3.40	1.43	4.85	2.80	2.86	0.000	7.880
TOTALS				62	85	0	0.000	0.440	0.440	3.40	1.43	4.85	2.80	2.86	7.880	7.880

		DEMAND (cont.)													
		INDUSTRIAL / COMMERICAL / INSTITUTIONAL FLOW							EXTRANOUS FLOW AREA METHOD				TOTAL DESIGN FLOW		
STREET	AREA	COMMERICAL / INSTITUTIONAL AREA (ha.)	CUMULATIVE COMMERICAL / INSTITUTIONAL AREA (ha.)	AVG DESIGN COMMERICAL / INSTITUTIONAL FLOW Q (ci) (L/s)	COMMERICAL / INSTITUTIONAL PEAK FACTOR	CUMULATIVE ICI DRAINAGE AREA (ha.)	PEAKED DESIGN ICI FLOW Q (CI) (L/s)	PEAKED ANNUAL/RARE POP FLOW Q(AR - ICI) (L/s)	CUMULATIVE EXTRANOUS DRAINAGE AREA (ha.)	DESIGN EXTRAN. FLOW Q(e) (L/s)	ANNUAL EXTRAN. FLOW Q(e) (L/s)	RARE EXTRAN. FLOW Q(e) (L/s)	TOTAL DESIGN FLOW Q(D) (L/s)	TOTAL ANNUAL FLOW Q(A) (L/s)	TOTAL RARE FLOW Q(R) (L/s)
Street 1	1		0.000	0.00	1.00	0.000	0.00	0.00	0.650	0.21	0.20	0.36	0.74	0.50	0.663
Street 1	2		0.000	0.00	1.00	0.000	0.00	0.00	1.020	0.34	0.31	0.56	1.24	0.83	1.090
Street 1	3		0.000	0.00	1.00	0.000	0.00	0.00	1.230	0.41	0.37	0.68	1.48	1.00	1.305
Street 1	4		0.000	0.00	1.00	0.000	0.00	0.00	1.430	0.47	0.43	0.79	1.66	1.13	1.483
Street 1	5		0.000	0.00	1.00	0.000	0.00	0.00	1.530	0.50	0.46	0.84	1.73	1.18	1.561
Street 2	6		0.000	0.00	1.00	0.000	0.00	0.00	0.890	0.29	0.27	0.49	0.66	0.48	0.703
Street 2	7		0.000	0.00	1.00	0.000	0.00	0.00	1.180	0.39	0.35	0.65	0.95	0.68	0.979
Street 2	8		0.000	0.00	1.00	0.000	0.00	0.00	1.620	0.53	0.49	0.89	1.41	1.00	1.404
Street 2	9		0.000	0.00	1.00	0.000	0.00	0.00	2.010	0.66	0.60	1.11	0.66	0.48	0.703
Street 2	10		0.000	0.00	1.00	0.000	0.00	0.00	1.280	0.42	0.38	0.70	1.03	0.74	1.057
Street 3	11		0.000	0.00	1.00	0.000	0.00	0.00	3.370	1.11	1.01	1.85	2.83	2.02	2.863
Street 3	12		0.000	0.00	1.00	0.000	0.00	0.00	3.740	1.23	1.12	2.06	3.17	2.26	3.191
Street 3	13		0.000	0.00	1.00	0.000	0.00	0.00	5.670	1.87	1.70	3.12	5.18	3.64	5.061
Street 3	14		0.000	0.00	1.00	0.000	0.00	0.00	5.790	1.91	1.74	3.18	5.25	3.70	5.144
Street 3	15		0.000	0.00	1.00	0.000	0.00	0.00	0.760	0.25	0.23	0.42	1.04	0.69	0.882
Street 3	16		0.000	0.00	1.00	0.000	0.00	0.00	1.060	0.35	0.32	0.58	1.45	0.96	1.228
Street 3	17		0.000	0.00	1.00	0.000	0.00	0.00	7.130	2.35	2.14	3.92	6.87	4.79	6.576
Street 3	18		0.000	0.00	1.00	0.000	0.00	0.00	7.160	2.36	2.15	3.94	6.88	4.80	6.592
Street 3	18		0.000	0.00	1.00	0.000	0.00	0.00	0.660	0.22	0.20	0.36	0.60	0.42	0.589
Street 3	20		0.000	0.00	1.00	0.000	0.00	0.00	0.720	0.24	0.22	0.40	0.62	0.44	0.622
Offsite			0.000	0.00	1.00	0.000	0.00	0.00	7.880	2.60	2.36	4.33	7.45	5.22	7.189
TOTALS			0.000	0.00	1.00	0.000	0.00	0.00	7.880	2.60	2.36	4.33	7.45	5.22	7.189

		DESIGN CAPACITY							
		PROPOSED SEWER PIPE SIZING / DESIGN							
STREET	AREA	PIPE LENGTH (m)	PIPE SIZE (mm) AND MATERIAL	PIPE ID ACTUAL (m)	ROUGH. (n)	DESIGN GRADE (%)	CAPACITY (L/s)	FULL FLOW VELOCITY (m/s)	Qpeak Design / Qcap
Street 1	1	84.5	200 PVC	0.203	0.013	1.30	39.0	1.20	1.9%
Street 1	2	46.0	200 PVC	0.203	0.013	1.30	39.0	1.20	3.2%
Street 1	3	29.8	200 PVC	0.203	0.013	1.50	41.9	1.29	3.5%
Street 1	4	18.5	200 PVC	0.203	0.013	1.50	41.9	1.29	4.0%
Street 1	5	16.4	200 PVC	0.203	0.013	1.50	41.9	1.29	4.1%
Street 2	6	53.5	200 PVC	0.203	0.013	0.50	24.2	0.75	2.7%
Street 2	7	30.7	200 PVC	0.203	0.013	0.50	24.2	0.75	3.9%
Street 2	8	57.0	200 PVC	0.203	0.013	0.50	24.2	0.75	5.8%
Street 2	9	51.5	200 PVC	0.203	0.013	0.50	24.2	0.75	2.7%
Street 2	10	91.8	200 PVC	0.203	0.013	0.35	20.2	0.62	5.1%
Street 3	11	32.9	250 PVC	0.254	0.013	0.25	31.0	0.61	9.1%
Street 3	12	67.0	250 PVC	0.254	0.013	0.25	31.0	0.61	10.2%
Street 3	13	79.6	250 PVC	0.254	0.013	0.25	31.0	0.61	16.7%
Street 3	14	6.4	250 PVC	0.254	0.013	0.50	43.9	0.87	12.0%
Street 3	15	88.2	200 PVC	0.203	0.013	3.50	64.0	1.97	1.6%
Street 3	16	57.3	200 PVC	0.203	0.013	3.50	64.0	1.97	2.3%
Street 3	17	54.8	250 PVC	0.254	0.013	0.25	31.0	0.61	22.1%
Street 3	18	22.9	250 PVC	0.254	0.013	0.25	31.0	0.61	22.2%
Street 3	18	73.8	200 PVC	0.203	0.013	4.50	72.6	2.24	0.8%
Street 3	20	42.7	200 PVC	0.203	0.013	4.50	72.6	2.24	0.9%
Offsite		24.5	250 PVC	0.254	0.013	4.67	134.1	2.65	5.6%
TOTALS									

DEMAND EQUATION

Design Parameters:

1. Q(D), Q(A), Q(R) = Q(p) + Q(fd) + Q(ici) + Q(e)
2. Q(p) = (P x q x M x K / 86,400)
3. q Avg capita flow (L/per/day)= 280 L/per/day (design); 200 L/per/day (annual and rare)
4. M = Harmon Formula (maximum of 4.0)
5. K = 0.8 (design); 0.6 (annual and rare)
6. Park flow is considered equivalent to a single unit / ha
   Park Demand = 1 Single Unit Equivalent / Park ha
7. Foundation Drains 0.45 L/s/unit
8. Q(ici) = ICI Area x ICI Flow x ICI Peak
9 Q(e) = 0.33 L/sec/ha (design); 0.30 L/sec/ha (annual); 0.55 L/sec/ha (rare)

Definitions:

Q(D) = Peak Design Flow (L/sec)   Q(A) = Peak Annual Flow (L/sec)
Q(e) = Extraneous Flow (L/sec)   Q(R) = Peak Rare Flow (L/sec)
Q(p) = Population Flow (L/sec)
K = Harmon Correction Factor

	Singles	Semis/Towns	Apts (2-BR)
P = Residential Population	3.4	2.7	2.1
Typ Service Diameter (mm)	135		
Typ Service Length (m)	15	15	
I/I Pipe Rate (L/mm dia/m/hr) =	0.007		

Q(fd) = Foundation Flow (L/sec)
Q(ici) = Industrial / Commercial / Institutional Flow (L/sec)

Institutional / Commercial / Industrial			Industrial	Commercial / Institutional	
	Design =		35000	28000	L/gHa/d
	Annual / Rare =		10000	17000	L/gHa/d
ICI Peak *	Design =	Std ICI -->	1.0	1.5	* ICI Peak = 1.0 Default, 1.5 if ICI in contributing area is >20% (design only)
	Annual / Rare =		1.0		

CAPACITY EQUATION

Q full= (1/n) A R^(2/3)So^(1/2)

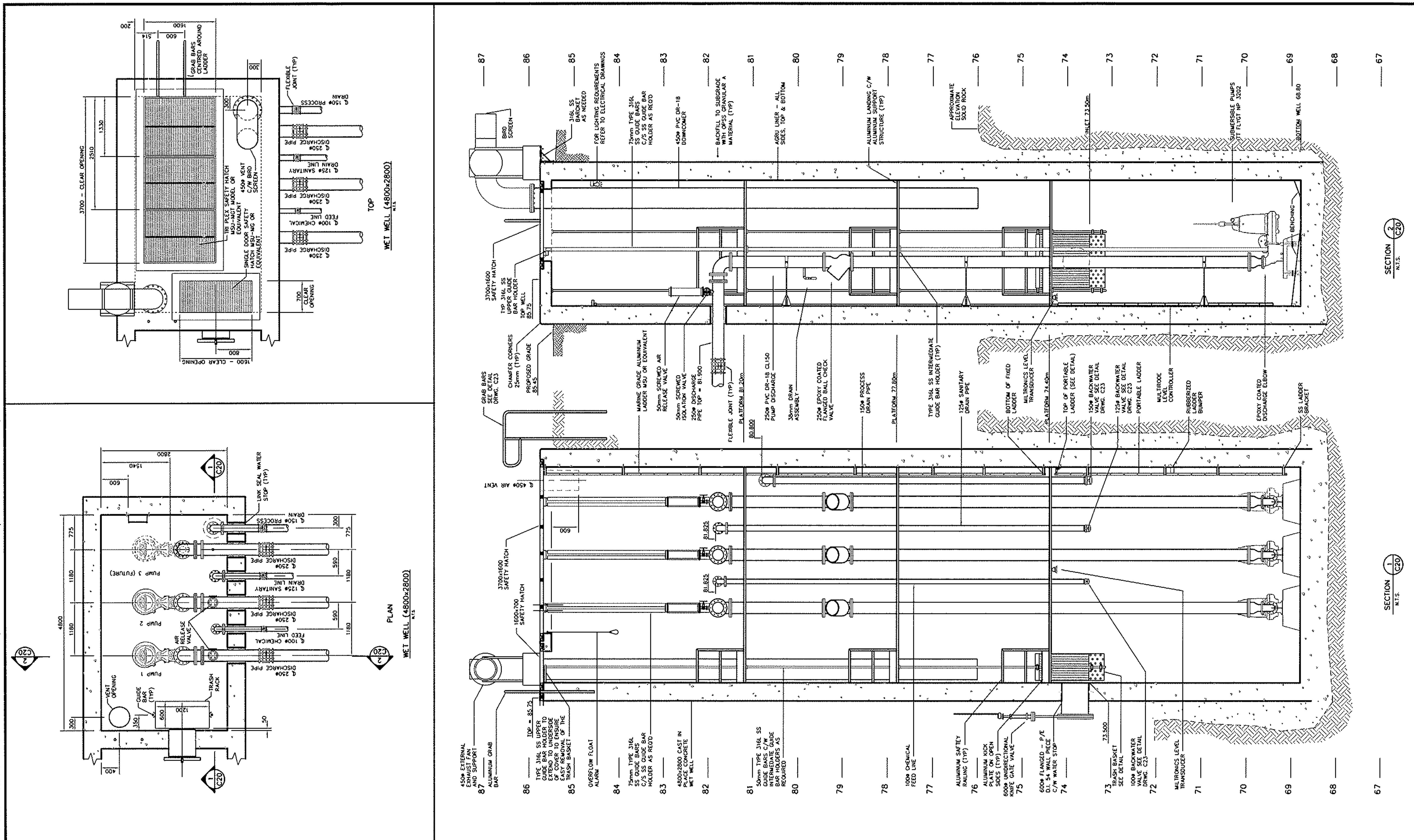
Where : Q full = Capacity (L/s)
n = Manning coefficient of roughness (0.013)
A = Flow area (m^2)
R = Wetter perimenter (m)
So = Pipe Slope/gradient

DESIGN BRIEF
VILLAGE OF MANOTICK
MUNICIPAL SERVICING
MAIN SANITARY SEWAGE PUMP STATION
CITY OF OTTAWA

11931

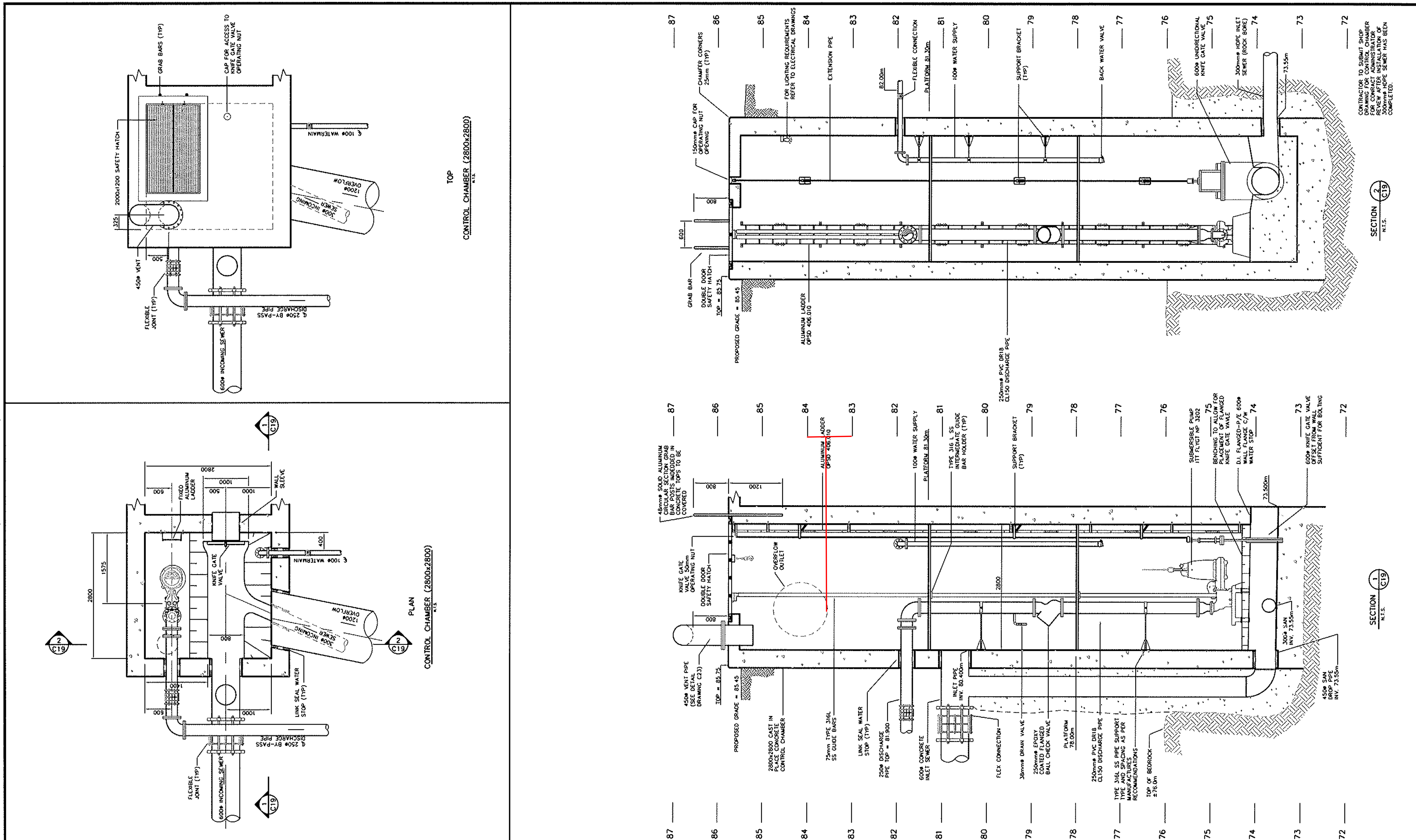
SEPTEMBER 2008

IBI GROUP



IBI GROUP

Project Title	Drawing Title	Project No. 11931
DESIGN BRIEF MANOTICK MUNICIPAL SERVICING MAIN SANITARY SEWAGE PUMP STATION CITY OF OTTAWA	WET WELL DETAILS	Date SEPT. 2008 FIGURE 7



IBI GROUP	Project Title DESIGN BRIEF MANOTICK MUNICIPAL SERVICING MAIN SANITARY SEWAGE PUMP STATION CITY OF OTTAWA	Drawing Title CONTROL CHAMBER DETAILS	Project No. 11931 Date SEPT. 2008 FIGURE 8

4.6 Emergency Overflow

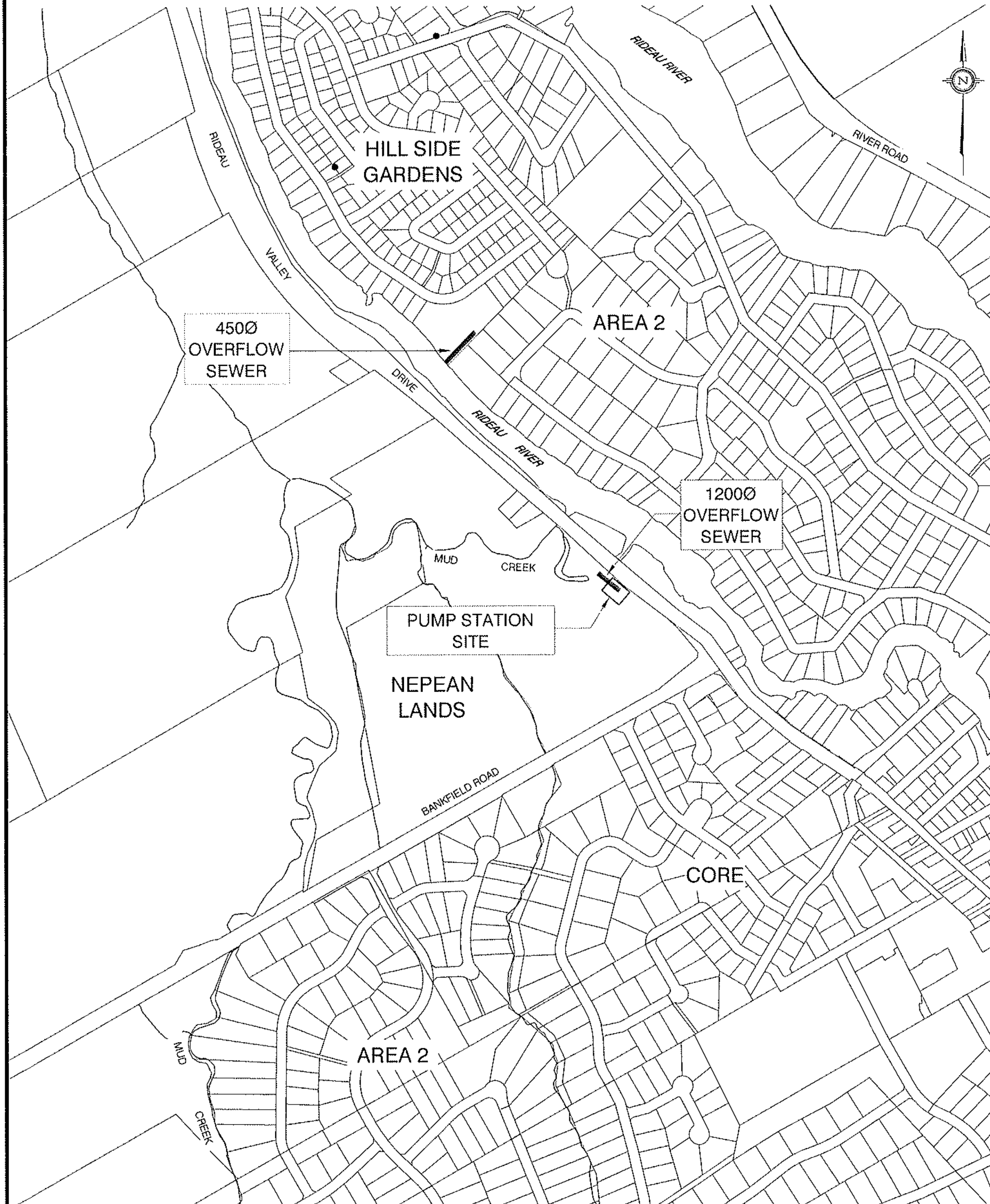
The proposed Main Sanitary Sewage Pump Station in Manotick will receive its power from the Hydro Ottawa power grid. In the event of interruption to that power source, the station will be equipped with a back-up diesel generator which automatically is put into service in the event of a grid power failure. This is a typical situation for most mid-sized sanitary pump stations.

Even with the automatically controlled back up power source, the City prefers to add a third level of operation to further ensure that sewers will not surcharge to the extent that buildings and houses connected to the system are flooded. Therefore, the potential to provide an overflow to the adjacent Rideau River has been investigated.

In order to assess the function of the proposed overflow system, the sanitary networks of the Hillside Gardens and Core areas were modelled using XPSWMM. XPSWMM is a dynamic computer model used primarily to model surcharged sewer systems. In this application, the model has quantified water levels in the sanitary sewers and computed the hydraulic grade line.

The assumed criteria are that the emergency overflow system must operate successfully during the 1:100 year storm event coincident with a peak wastewater event. Flood levels within the Rideau River for the 1:100 year event were obtained from the Rideau Valley Conservation Authority and the wastewater model, including sewer sizes, lengths and flows, were imported from the sanitary sewer design spreadsheets. Results of the predicted hydraulic grade line (HGL) elevations were compared to underside of footing (USF) elevations for each building in the service area. The USF elevations were assumed to be 0.3m below the surveyed basement floor elevations.

The proposed overflow strategy will employ two overflow locations within the sanitary sewer network. The first overflow will be a 1200mm diameter pipe and will be connected to the Control Chamber located on the pump station site, and will discharge into a backwater tributary to Mud Creek. The second overflow will be a 450mm diameter pipe and will be located in George McLean Park near Hillside Gardens, and will discharge directly to the Rideau River. The 1:100 year flood level of the Rideau River was determined to be 83.53m at the backwater tributary to Mud Creek and 83.46m adjacent to George McLean Park. The overflow sewer locations are shown in Figure 11. The performances of the results are categorized as pass, fail or pumped. A pass is assumed for any building where the predicted sanitary HGL is below the USF elevation. The tabulated results include only those areas that are marginal. All other houses and buildings are above the predicted HGL elevation and are considered passing.



IBI GROUP	Project Title DESIGN BRIEF MANOTICK MUNICIPAL SERVICING MAIN SANITARY SEWAGE PUMP STATION CITY OF OTTAWA	Drawing Title LOCATION PLAN FOR OVERFLOW SEWERS	Project No. 11931 Date SEPT. 2008 FIGURE 11

Table: XPSWMM Results

Location	Node ID	Civic Address	USF elev (m)	HGL (m)	Diff (m)	Status
Hillside Gardens	113	5254 McLean Crescent	n/a	84.92		n/a
		5257 McLean Crescent	n/a	84.82		n/a
		5258 McLean Crescent	86.29	84.78	-1.51	Pass
	112	5260 McLean Crescent	n/a	84.70		n/a
		5261 McLean Crescent	87.01	84.78	-2.23	Pass
		5263 McLean Crescent	86.58	84.72	-1.86	Pass
		5264 McLean Crescent	85.01	84.62	-0.39	Pass
		5267 McLean Crescent	86.50	84.64	-1.86	Pass
		5268 McLean Crescent	n/a	84.60		n/a
		5269 McLean Crescent	n/a	84.60		n/a
	111	5272 McLean Crescent	84.86	84.51	-0.35	Pumped
		5273 McLean Crescent	85.84	84.53	-1.31	Pass
		5274 McLean Crescent	83.38	84.49	1.11	Pumped
		5275 McLean Crescent	86.04	84.49	-1.55	Pass
		5278 McLean Crescent	n/a	84.45		n/a
		5279 McLean Crescent	86.51	84.45	-2.06	Pass
		5282 McLean Crescent	83.86	84.41	0.55	Pumped
		5283 McLean Crescent	86.34	84.42	-1.92	Pass
		5285 McLean Crescent	87.26	84.41	-2.85	Pass
	110	5286 McLean Crescent	n/a	84.40		n/a
	109	5288 McLean Crescent	83.73	84.36	0.63	Pumped
		5289 McLean Crescent	86.96	84.36	-2.6	Pass
		5290 McLean Crescent	83.73	84.34	0.61	Pumped
		5293 McLean Crescent	86.99	84.34	-2.65	Pass
		5295 McLean Crescent	85.71	84.34	-1.37	Pass
		5298 McLean Crescent	84.54	84.30	-0.24	Pass
		5299 McLean Crescent	86.44	84.29	-2.15	Pass
		5302 McLean Crescent	84.63	84.29	-0.34	Pass
	108	5303 McLean Crescent	86.32	84.28	-2.04	Pass
		5305 McLean Crescent	86.14	84.27	-1.87	Pass
	107	5306 McLean Crescent	85.17	84.25	-0.92	Pass
		5309 McLean Crescent	n/a	84.23		n/a
		5310 McLean Crescent	84.61	84.22	-0.39	Pass
		5313 McLean Crescent	86.47	84.20	-2.27	Pass
		5314 McLean Crescent	85.40	84.21	-1.19	Pass
		5315 McLean Crescent	86.75	84.19	-2.56	Pass
	106	5318 McLean Crescent	85.52	84.16	-1.36	Pass
Core	258	5497 Dickinson Circle	83.96	84.73	0.77	Pumped
		5499 Dickinson Circle	83.28	84.73	1.45	Pumped
		5501 Dickinson Circle	82.91	84.73	1.82	Pumped
	259	5503 Dickinson Circle	84.11	84.73	0.62	Pumped
	257	1129 Bridge Street	86.30	84.73	-1.57	Pass
	260	1131 Bridge Street	85.70	84.73	-0.97	Pass
	241	1118 Tighe Street	86.16	89.73	3.57	Pumped
		1119 Tighe Street	91.18	89.73	-1.45	Pass
	236B	1117 O'Grady Street	88.11	89.10	0.99	Pumped

Location	Node ID	Civic Address	USF elev (m)	HGL (m)	Diff (m)	Status
		1118 O'Grady Street	86.98	89.10	2.12	Pumped
	234B	1125 Currier Street	87.40	89.43	2.03	Pumped
	232	5583 Dickinson Street	88.97	89.83	0.86	Pumped
		5579 Dickinson Street	89.05	89.73	0.68	Pumped
	233	5573 Dickinson Street	90.14	89.65	-0.49	Pass
		5569 Dickinson Street	89.91	89.45	-0.46	Pass
	234	5565 Dickinson Street	90.41	89.35	-1.06	Pass
	221	1157 Maple Avenue	86.33	84.78	-1.55	Pass
	224 225	5514 Main Street	85.11	84.75	-0.36	Pass

The results presented in the above table indicate that under the specified criteria, the provided overflows will not negatively impact the existing or proposed development, and are therefore considered successful. The predicted HGL is below all USF elevations with the exception of those houses requiring pumping. A plan and appropriate profiles from the XPSWMM model output are included in Appendix D. For reference, the pink line illustrated on the profile drawings represents the HGL elevation, and the brown line represents the ground profile.

5.0 OTHER DESIGN ELEMENTS

5.1 Main Power Supply

The electrical power supply to the pumping station will be 600 volt, 3 phase, 60 Hertz. Major pieces of equipment will operate on 600V, 3pH, power supply. A lighting transformer and lighting panel will be provided. Power available from the lighting panel will be either 120 volt or 240 volt single phase 60 Hertz. All lighting and outlets and minor pieces of equipment will be operated from this power source.

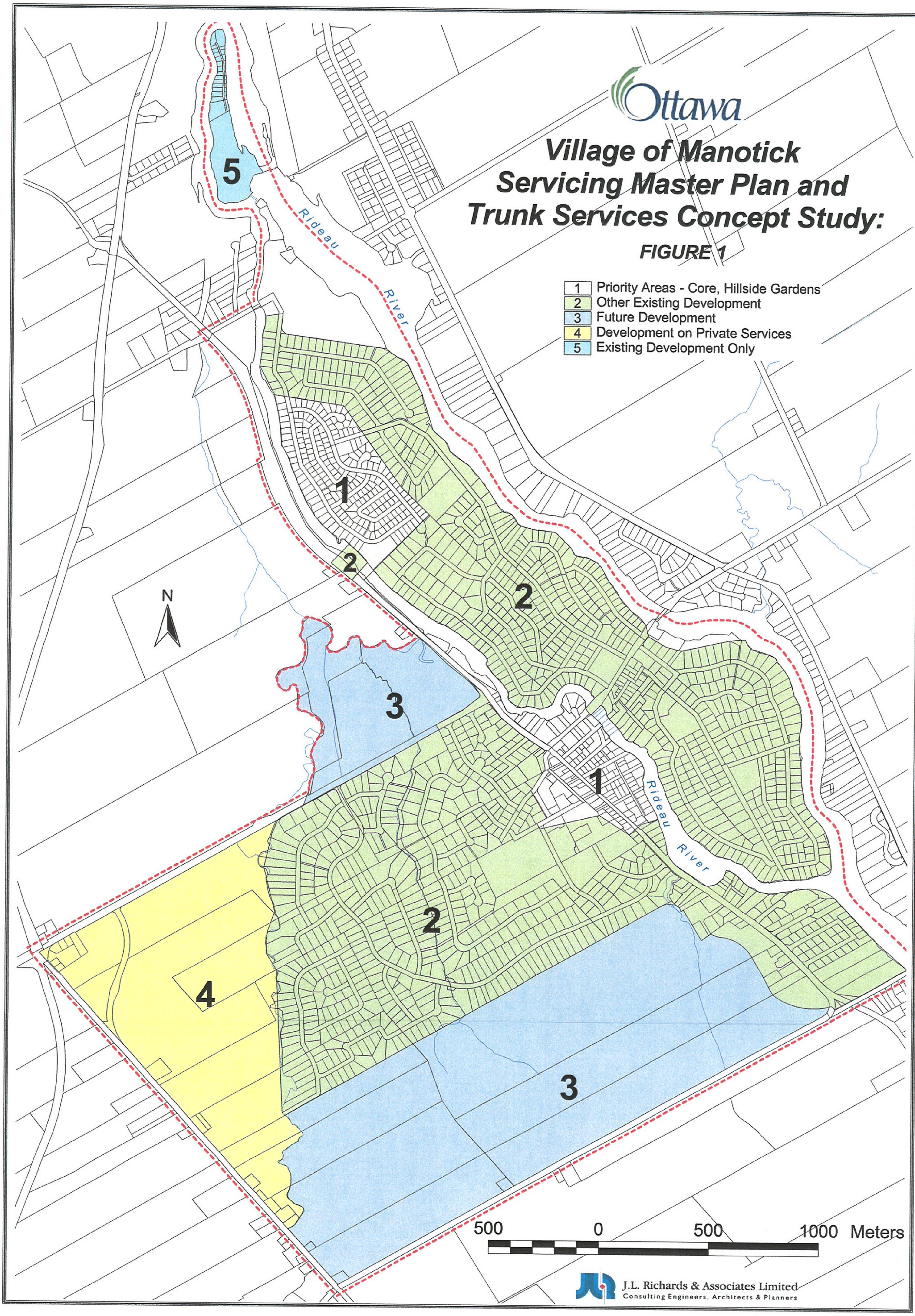
Preliminary discussions with the Hydro Ottawa, the power supply authority, indicate that a 750 KVa supply can be provided to the station. Supply to the station site will be through a pad mount transformer on site.

5.2 Electrical Systems

Motor starters and/or breakers will be contained in a modular motor control centre (MCC) with sections for incoming supply, main breakers, etc. A separate process metering control panel will be provided adjacent to the MCC section in which will be mounted the independent wet well level indicators, magnetic flow indicator readings and any other necessary process indicators. Soft Starts will be provided in order to minimize the "in-rush" or "start-up" current and thereby reduce the size of emergency generator required. Deceleration or "ramp-down" stops will also be included.

APPENDIX A

Manotick Service Areas


Ottawa
Village of Manotick
Servicing Master Plan and
Trunk Services Concept Study:
FIGURE 1
1 Priority Areas - Core, Hillside Gardens
2 Other Existing Development
3 Future Development
4 Development on Private Services
5 Existing Development Only
Rideau River
N
500 0 500 1000 Meters
J.L. Richards & Associates Limited
Consulting Engineers, Architects & Planners

APPENDIX B

Sanitary Sewer Design Sheets and Village of Manotick Sanitary Drainage Areas

IBI GROUP

SANITARY FLOW CALCULATIONS TO BUILDOUT
MANOITCK MAIN SANITARY SEWAGE PUMP STATION
DESIGN BRIEF
(MONITORED = RESIDENTIAL PEAKED WITH HARMON/NON-RESIDENTIAL PEAKED AT 1.0)

2010 (Core And Hillside Gardens partially hooked up)

	DESIGN (l/s)					MONITORED (l/s)					
	UNITS	AREA	POPN	AVG W/O I/I	PEAK	AVG DWF W/O I/I	AVG DWF	PK DWF	TYPICAL WWF	ANNUAL WWF	RARE WWF
RESIDENTIAL/PAR	175	20	595	2.41	15.08	2.07	3.07	5.06	7.06	9.66	12.06
COMM/INSTIT		15	0	8.68	13.02	2.60	3.35	3.35	4.85	6.80	8.60
TOTALS	175	35	595	11.09	28.10	4.67	6.42	8.42	11.92	16.47	20.67

2015 (Core And Hillside Gardens hooked up, Minto 25% built out)

	DESIGN (l/s)					MONITORED (l/s)					
	UNITS	AREA	POPN	AVG W/O I/I	PEAK	AVG DWF W/O I/I	AVG DWF	PK DWF	TYPICAL WWF	ANNUAL WWF	RARE WWF
RESIDENTIAL/PAR	573	81.01	1947	7.89	51.03	6.76	10.81	16.20	24.30	34.83	44.55
COMM/INSTIT		31.94		18.48	36.67	5.55	7.14	7.14	10.34	14.49	18.32
TOTALS	572.5	112.95	1947	26.37	87.70	12.30	17.95	23.34	34.64	49.32	62.87

2020 (Core And Hillside Gardens hooked up, Minto 50% built out)

	DESIGN (l/s)					MONITORED (l/s)					
	UNITS	AREA	POPN	AVG W/O I/I	PEAK	AVG DWF W/O I/I	AVG DWF	PK DWF	TYPICAL WWF	ANNUAL WWF	RARE WWF
RESIDENTIAL/PAR	855	114.81	2907	11.78	72.82	10.09	15.83	23.17	34.65	49.58	63.36
COMM/INSTIT		31.94	0	18.48	36.67	5.55	7.14	7.14	10.34	14.49	18.32
TOTALS	855	146.75	2907	30.26	109.49	15.64	22.98	30.31	44.99	64.07	81.68

2025 (Core And Hillside Gardens hooked up, Minto 75% built out)

	DESIGN (l/s)					MONITORED (l/s)					
	UNITS	AREA	POPN	AVG W/O I/I	PEAK	AVG DWF W/O I/I	AVG DWF	PK DWF	TYPICAL WWF	ANNUAL WWF	RARE WWF
RESIDENTIAL/PAR	1138	148.61	3868	15.67	94.04	13.43	20.86	29.90	44.76	64.08	81.91
COMM/INSTIT		31.94		18.48	36.67	5.55	7.14	7.14	10.34	14.49	18.32
TOTALS	1137.5	180.55	3868	34.15	130.71	18.97	28.00	37.04	55.10	78.57	100.23

2030 (Core, Hillside Gardens, Minto and 100% built out)

	DESIGN (l/s)					MONITORED (l/s)					
	UNITS	AREA	POPN	AVG W/O I/I	PEAK	AVG DWF W/O I/I	AVG DWF	PK DWF	TYPICAL WWF	ANNUAL WWF	RARE WWF
RESIDENTIAL/PAR	1440	182.41	4896	19.83	115.60	17.00	26.12	36.77	55.02	78.73	100.62
COMM/INSTIT		31.94	0	18.48	36.67	5.55	7.14	7.14	10.34	14.49	18.32
TOTALS	1440	214.35	4896	38.32	152.27	22.55	33.26	43.92	65.35	93.22	118.94

BUILDOUT

	DESIGN (l/s)					MONITORED (l/s)					
	UNITS	AREA	POPN	AVG W/O I/I	PEAK	AVG DWF W/O I/I	AVG DWF	PK DWF	TYPICAL WWF	ANNUAL WWF	RARE WWF
RESIDENTIAL/PAR	2837	574.54	9646	39.07	276.93	33.49	62.22	78.47	135.92	210.61	279.56
COMM/INSTIT		39.04		22.59	44.82	6.78	8.73	8.73	12.63	17.71	22.39
TOTALS	2837	613.58	9646	61.67	321.75	40.27	70.95	87.20	148.55	228.32	301.95

Revised April 2008

Revised Sept 2008

UNIT SANITARY FLOWS

SOURCE	Monitored		Design
Residential (Lpcd)			
Average	300		350
Peak Factor	Harmon (K)	(K= 0.40 to 0.60)	Harmon (K=1.0)
Unit Population			3.4
ICI (L/ha/d)			
Average	15000		50000
Peak Factor	1 (non-coincident peak)		1.5
Inflow/Infiltration (L/ha/s)			
Dry Weather Inflow (Low)	0.05	0.08	
Wet Weather Event (Typ)	0.15	0.20	0.28
Wet Weather Event (Annual)	0.28		
Wet Weather Event (Rare)	0.30	0.50	

Inflow/Infiltration Allowances Used For Monitored Events Are

Average Dry Weather=	0.05 l/s
Peak Dry Weather Flow=	0.05 l/s
Typical Dry Weather Flow=	0.15 l/s
Annual Wet Weather Flow=	0.28 l/s
Rare Wet Weather Flow=	0.40 l/s

K factor used for Harmon Formula for monitored events = 0.50

![IBI GROUP]

SANITARY SEWER DESIGN SHEET

Manotick Main Sanitary Sewage Pump Station
City Of Ottawa
Contract No. ISB06-2053

JOB #: 11931
DATE: 26-Sep-08
DESIGN: JIM

LOCATION			INDIVIDUAL						CUM. RES. FLOW			CUM. COM. & INST. FLOW					INFILTRATION			TOTAL	PROPOSED SEWER						
STREET	FROM MH	TO MH	RESID. UNITS Singles	Towns Semis	Condo	RES. AREA (Ha)	Parks/OS	POP.	POP.	PEAK FACT.	PEAK FLOW (l/s)	COMMER AREA (Ha)	INSTIT AREA (Ha)	TOTAL AREA (Ha)	PEAK FACT.	PEAK FLOW (l/s)	INCR. AREA (Ha)	CUM. AREA (Ha)	FLOW (l/s)	DESIGN FLOW (l/s)	CAP. l/s	PIPE (mm)	LGTH. (m)	SLOPE %	VEL. (full) m/s	AVAIL. CAP. (l/s)	AVAIL. CAP. (%)
Incoming Sewer To Station																											
Rideau Valley Drive	Stub	Wet Well				381.14		6793.2	6793.2	3.12	85.84	30.17		30.17	1.5	26.19	411.31	411.31	115.17	**227.19**	**452.97**	600	21.0	0.50	1.55	225.78	50%
Outlet Sewer																											
Jockvale Road	Chamber	MH 1				574.54		9645.0	9645.0	2.97	116.05	39.04		39.04	1.5	33.89	613.58	613.58	171.80	**321.74**	**329.71**	375	12.5	3.25	2.89	7.97	2%
Golflinks Drive	MH 1	Ex MH				574.54		9645.0	9645.0	2.97	116.05	39.04		39.04	1.5	33.89	613.58	613.58	171.80	**321.74**	**329.71**	375	22.5	3.25	2.89	7.97	2%

Where average daily per capita flow (350 l/cap.d.) or (0.0041l/sec./cap)
Unit of peak extraneous flow (0.28 l/sec/ha)
Residential Peaking factor = Harmon Peaking Factor , M = 1+(14/(4+P^0.5)) , where P = population in thousands
Commercial/Institutional Flow Rate = 50000 Peaking Factor = 1.5

Pipe Coefficient = 0.013

IBI GROUP

SANITARY FLOW PROJECTION-INTERIM AND ULTIMATE

Manotick Municipal Servicing

Main Sanitary Sewage Pump Station

City of Ottawa

LOCATION			INDIVIDUAL RESIDENTIAL				CUMULATIVE RESID'L FLOW				CUMULATIVE ICI FLOW			INFILTRATION ALLOWANCE			TOTAL	PROPOSED SEWER DESIGN					
Street/Area	From MH	To MH	Area for Pop. (Ha)	Population	ICI Area (Ha.)	Park/OS (Ha.)	Population	Avg. Flow (l/s)	Peaking Factor	Peak Flow (l/s)	Area	Pk. Fact.	Pk. Flow (l/s)	Incr. Area (Ha.)	Cum. Area (Ha.)	Flow (l/s)	FLOW (l/s)	Capacity (l/s)	Pipe Size (mm)	Length (M)	Slope (%)	Velocity(f M/sec	Avail. Cap. (%)
INTERIM																							
Hillside Gardens		1670	28.17	734	2.55		734	2.97	3.88	11.55	2.55			30.72	30.72								
Core		1670	12.53	253	26.69		253	1.02	4.00	4.10	26.69			39.22	39.22								
Area 2		1670	6.51	68	2.70		68	0.28	4.00	1.10	2.70			9.21	9.21								
Minto Lands		1670	135.20	3842	0.00		3842	15.56	3.35	52.12	0.00			135.20	135.20								
City lands (Station Site)		1670									0.00			0.00	0.00								
Total Interim Flows	1670	PS	182.41	4897	31.94	0.00	4897	19.84	3.25	64.54	31.94	1.50	27.73	0.00	214.35	60.02	152.28						
ULTIMATE																							
Hillside Gardens		1670	28.17	734	2.55		734	2.97	3.88	11.54	2.55			30.72	30.72								
Core		1670	9.78	54	29.44		54	0.22	4.00	0.88	29.44			39.22	39.22								
Minto Lands		1670	135.20	3842	0.00		3842	15.56	3.35	52.12	0.00			135.20	135.20								
Area 2		1670	364.19	4444	7.05		4444	18.00	3.29	59.26	7.05			371.24	371.24								
City lands (Station Site)		1670									0.00			0.00	0.00								
Nepean Lands		1670	37.20	571	0.00	0.00	571	2.31	3.94	9.13	0.00			37.20	37.20								
Total Ultimate Flows	1670	PS	574.54	9645	39.04	0.00	9645	39.07	2.97	116.05	39.04	1.50	33.89	0.00	613.58	171.80	321.74						

Population Per Unit: 3.4 All units

Avg. Per Capita Flow Rate: 350 l/day

Infiltration Allowance: 0.28 l/sec/Ha

Residential Peaking Factor: Harmon Formula = 1+(14/(4+P^0.5)) where P = pop'n in thousands

Avg. Commercial/Institutional: 50000 l/Ha/day

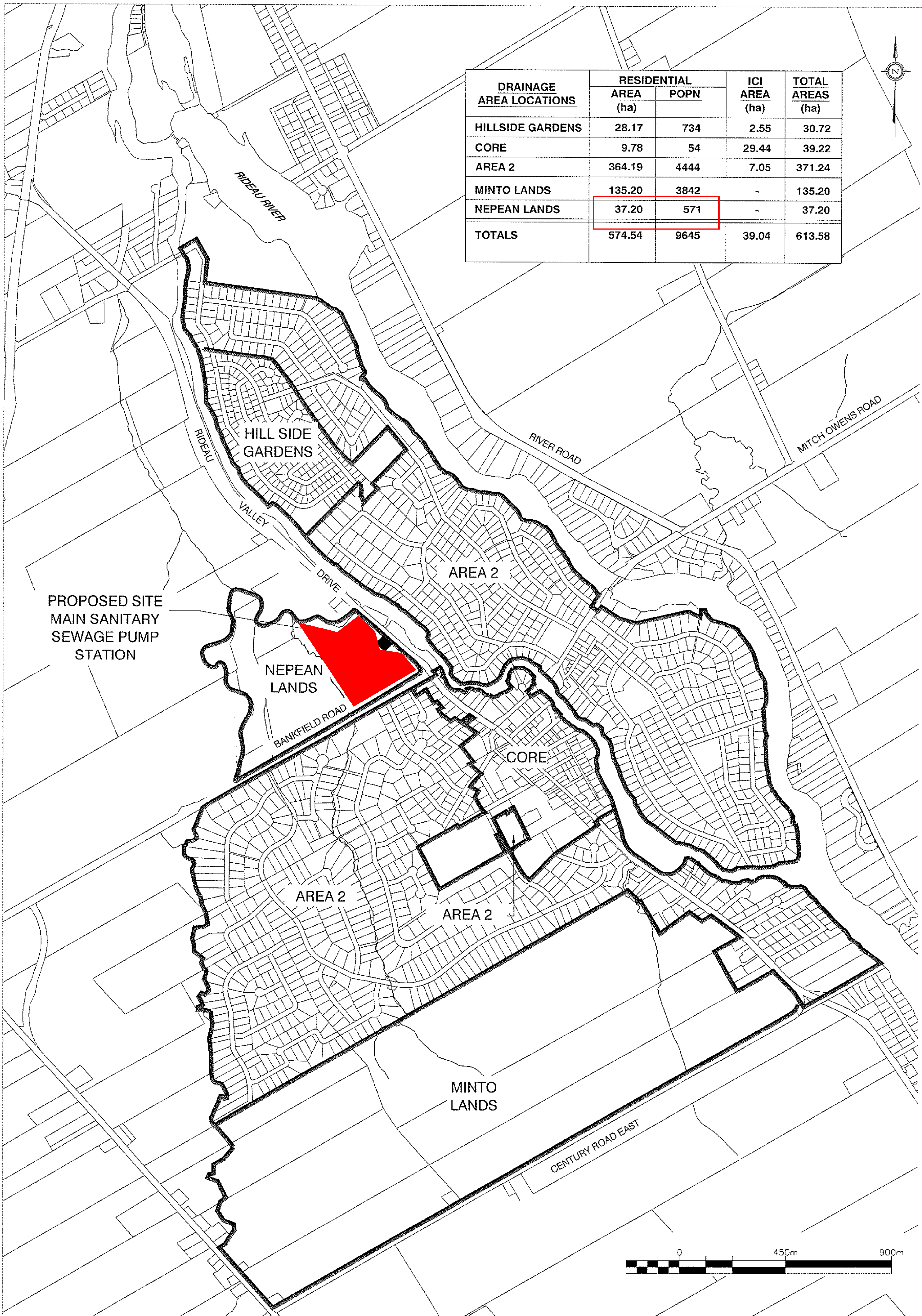
Assumed pipe loss ceofficient = 0.013

Revised: Apr-08

Revised: Sep-08

J:\11931_Manotick\5.9 Drawings\59civil\current\Design Brief-Sept. 2008\FIG 2-5.dwg Sheet Set: ####
Plot Style: AIA STANDARD COLOR-HALF.CTB Plot Scale: 1:1 Plotted At: Sep. 29, 08 12:37 PM Printed By: DON SIURNA Last Saved By: ---- Last Saved At: Sep. 23, 08

DRAINAGE AREA LOCATIONS	RESIDENTIAL AREA (ha)	RESIDENTIAL POPN	ICI AREA (ha)	TOTAL AREAS (ha)
HILLSIDE GARDENS	28.17	734	2.55	30.72
CORE	9.78	54	29.44	39.22
AREA 2	364.19	4444	7.05	371.24
MINTO LANDS	135.20	3842	-	135.20
NEPEAN LANDS	37.20	571	-	37.20
TOTALS	574.54	9645	39.04	613.58



IBI GROUP

Project Title: DESIGN BRIEF
MANOTICK MUNICIPAL SERVICING
MAIN SANITARY SEWAGE PUMP STATION
CITY OF OTTAWA

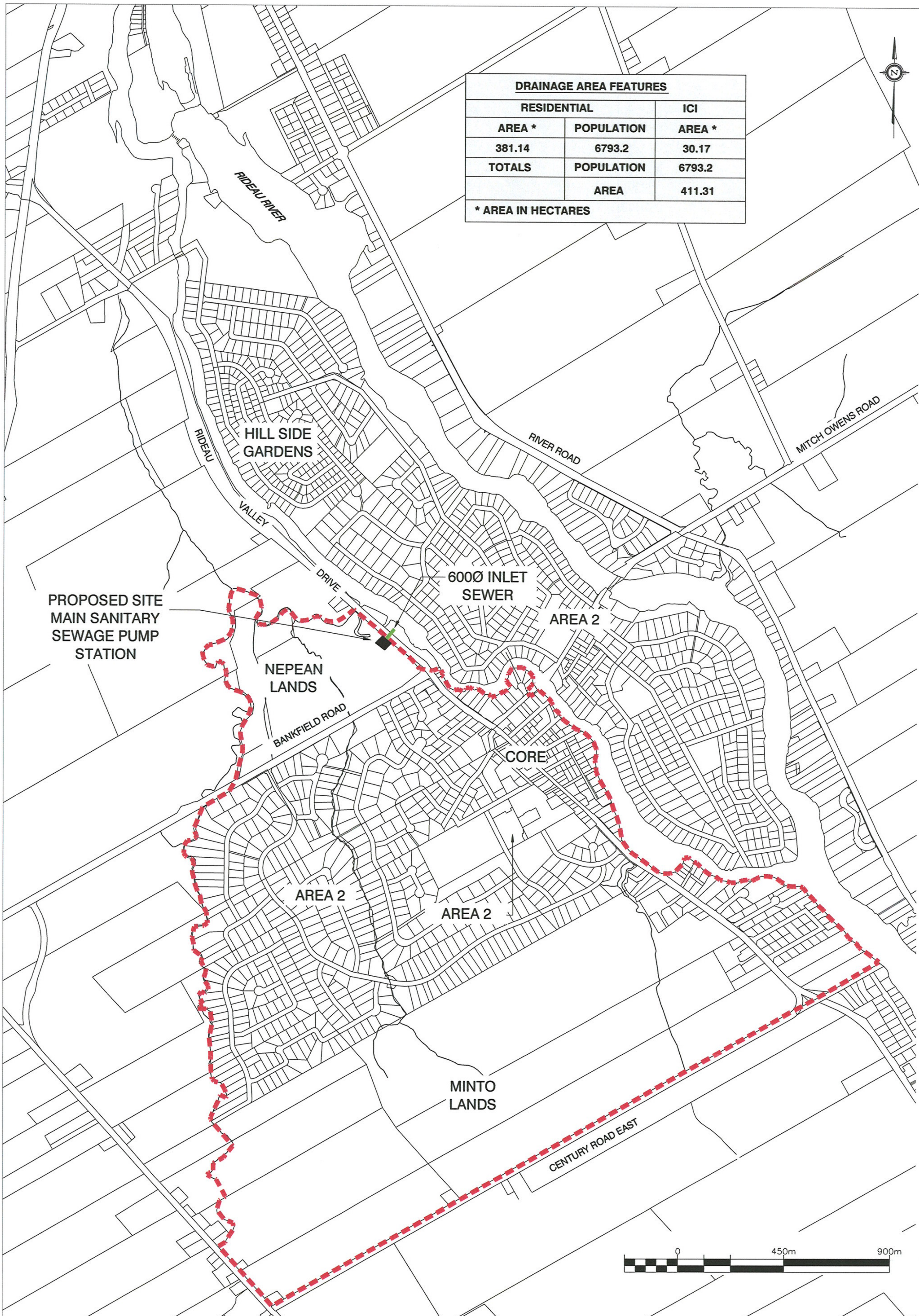
Drawing Title: SANITARY DRAINAGE AREAS

Project No. 11931

Date: SEPT. 2008

FIGURE 3

J:\11931_Manotick\5.9 Drawings\59civil\current\Design Brief-Sept. 2008\FIG 2-5.dwg Sheet Set: ####
Plot Style: AIA STANDARD COLOR-HALF.CTB Plot Scale: 1:1 Plotted At: Sep. 29, 08 2:16 PM Printed By: DON SIURNA Last Saved By: ---- Last Saved At: Sep. 29, 08



DRAINAGE AREA FEATURES		
RESIDENTIAL		ICI
AREA *	POPULATION	AREA *
381.14	6793.2	30.17
TOTALS	POPULATION	6793.2
	AREA	411.31

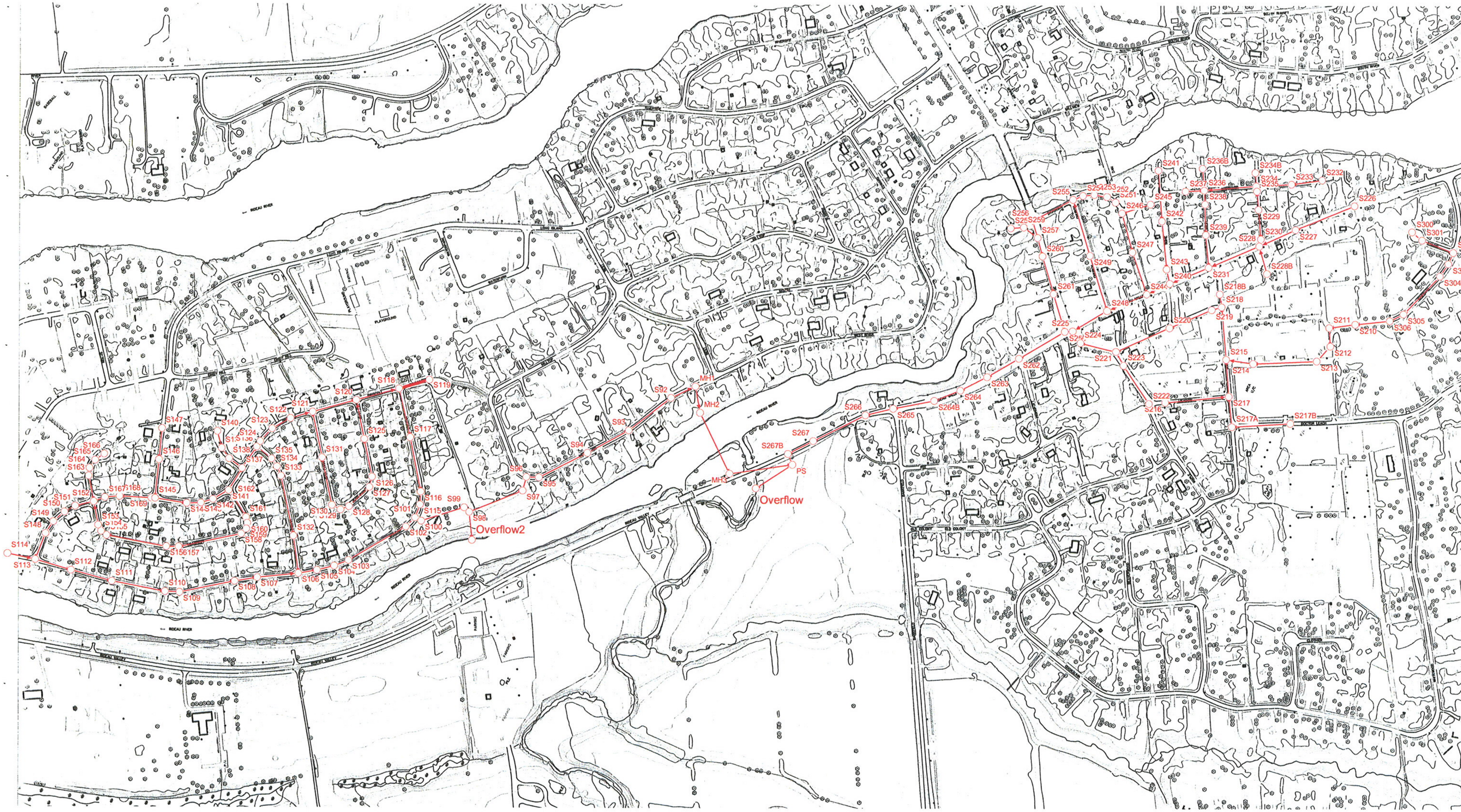
* AREA IN HECTARES

IBI GROUP

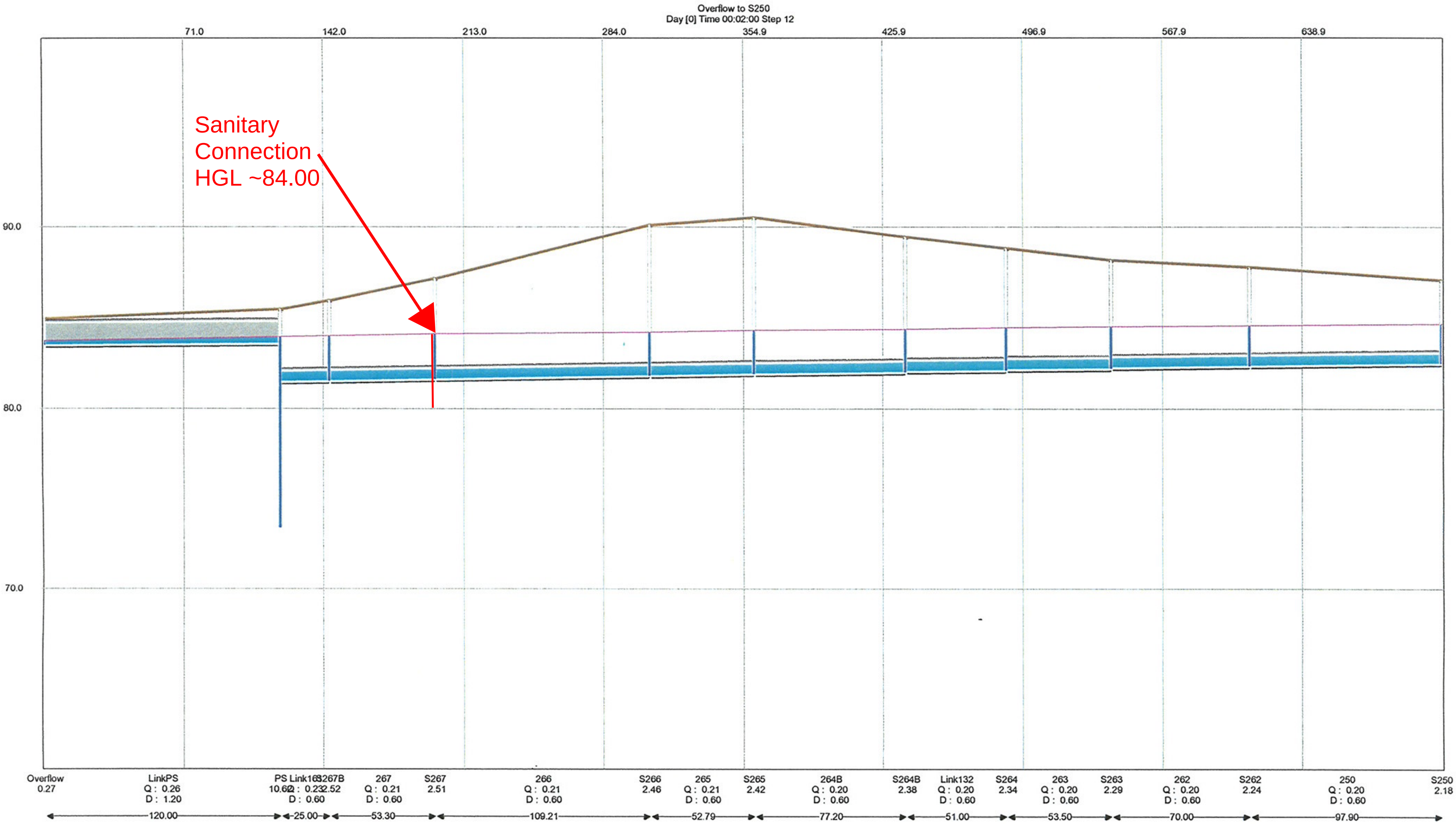
Project Title	Drawing Title	
DESIGN BRIEF MANOTICK MUNICIPAL SERVICING MAIN SANITARY SEWAGE PUMP STATION CITY OF OTTAWA	DRAINAGE AREA PLAN INLET SEWER	Project No. 11931 Date SEPT. 2008 FIGURE 6

APPENDIX D

Emergency Overflow Plan and Profiles



SWMM Version 9.12 Copyright (c) XP Software	**11931 - Manotick Pump Station** **Emergency Overflow HGL Analysis** Licenced To: Cumming Cockburn [42-1000-1763]	08/27/08 Page 1/1



Overflow to S250
Day [0] Time 00:02:00 Step 12
71.0
142.0
213.0
284.0
354.9
425.9
496.9
567.9
638.9
Sanitary
Connection
HGL ~84.00
90.0
80.0
70.0
Overflow
0.27
LinkPS
Q : 0.26
D : 1.20
PS Link16S267B
10.62Q : 0.232.52
D : 0.60
267
Q : 0.21
D : 0.60
S267
2.51
266
Q : 0.21
D : 0.60
S266
2.46
265
Q : 0.21
D : 0.60
S265
2.42
264B
Q : 0.20
D : 0.60
S264B
2.38
Link132
Q : 0.20
D : 0.60
S264
2.34
263
Q : 0.20
D : 0.60
S263
2.29
262
Q : 0.20
D : 0.60
S262
2.24
250
Q : 0.20
D : 0.60
S250
2.18
120.00
25.00
53.30
109.21
52.79
77.20
51.00
53.50
70.00
97.90

Appendix E
Water Demand Calculations and Hydraulic Modeling

Boundary Conditions
4386 Rideau Valley Drive

Provided Information

Scenario	Demand	
	L/min	L/s
Average Daily Demand	86	1.43
Maximum Daily Demand	308	5.14
Peak Hour	463	7.71
Fire Flow Demand #1	10,000	166.67
Fire Flow Demand #2	13,500	225.00

Location

Results – Existing Conditions

Connection 1 – Rideau Valley Dr.

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	156.6	100.5
Peak Hour	139.6	76.3
Max Day plus Fire 1	124.2	54.4
Max Day plus Fire 2	107.3	30.4

Ground Elevation = 85.9 m

Connection 2 – Rideau Valley Dr. / Bankfield Rd.

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	156.6	99.3
Peak Hour	139.6	75.1
Max Day plus Fire 1	123.0	51.6
Max Day plus Fire 2	105.5	26.6

Ground Elevation = 86.7 m

Results – SUC Zone Reconfiguration

Connection 1 – Rideau Valley Dr.

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	148.2	88.6
Peak Hour	141.6	79.1
Max Day plus Fire 1	119.7	48.1
Max Day plus Fire 2	104.0	25.8

Ground Elevation = 85.9 m

Connection 2 – Rideau Valley Dr. / Bankfield Rd.

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	148.2	87.4
Peak Hour	141.5	77.9
Max Day plus Fire 1	118.6	45.3
Max Day plus Fire 2	102.2	22.0

Ground Elevation = 86.7 m

Notes

1. As per the Ontario Building Code in areas that may be occupied, the static pressure at any fixture shall not exceed 552 kPa (80 psi.) Pressure control measures to be considered are as follows, in order of preference:
    a. If possible, systems to be designed to residual pressures of 345 to 552 kPa (50 to 80 psi) in all occupied areas outside of the public right-of-way without special pressure control equipment.
    b. Pressure reducing valves to be installed immediately downstream of the isolation valve in the home/ building, located downstream of the meter so it is owner maintained.

Disclaimer

The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.

FUS - Fire Flow Calculations

As per 1999 Fire Underwriter's Survey Guidelines

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands
Date: 7/20/2022
Input By: Brendan Rundle
Reviewed By: Sam Bahia / Ben Sweet

Legend: Input by User; No Information or Input Required

Building Description: Lots 1-22, 2 Storey Singles
Wood frame

Step			Input		Value Used	Total Fire Flow (L/min)
	Base Fire Flow					
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Wood frame	Yes	1.5	1.5	
		Ordinary construction		1		
		Non-combustible construction		0.8		
		Modified Fire resistive construction (2 hrs)		0.6		
		Fire resistive construction (> 3 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	3785			
		Number of Floors/Storeys	2			
		Area of structure considered (m^2)			**7,570**	
	F	**Base fire flow without reductions** $F = 220\ C\ (A)^{0.5}$				**29,000**
	Reductions or Surcharges					
3	**Occupancy hazard reduction or surcharge**			**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**24,650**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**			**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	No	-30%		**0**
		Standard Water Supply	No	-10%		
		Fully Supervised System	No	-10%		
				Cumulative Total	**0%**	
5	**Exposure Surcharge (cumulative %)**				**Surcharge**	
	(3)	North Side	> 45.1m		0%	**2,465**
		East Side	20.1 - 30 m		10%	
		South Side	> 45.1m		0%	
		West Side	> 45.1m		0%	
				Cumulative Total	**10%**	
	Results					
6	**(1) + (2) + (3)**	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**27,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**450**
				or	USGPM	7,133
7	**Storage Volume**	Required Duration of Fire Flow (hours)			Hours	6
		Required Volume of Fire Flow (m^3)			m^3	9720

FUS - Fire Flow Calculations

As per 1999 Fire Underwriter's Survey Guidelines

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #:	121153
Project Name:	Stinson Lands
Date:	7/20/2022
Input By:	Brendan Rundle
Reviewed By:	Sam Bahia / Ben Sweet

Legend

	Input by User
	No Information or Input Required

Building Description: Block 78, 2 Storey Towns
Wood frame

Step			Input		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Wood frame	Yes	1.5	1.5	
		Ordinary construction		1		
		Non-combustible construction		0.8		
		Modified Fire resistive construction (2 hrs)		0.6		
		Fire resistive construction (> 3 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	600			
		Number of Floors/Storeys	2			
		Area of structure considered (m^2)			**1,200**	
	F	**Base fire flow without reductions**				**11,000**
		$F = 220\ C\ (A)^{0.5}$				
Reductions or Surcharges						
3	**Occupancy hazard reduction or surcharge**			**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**9,350**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**			**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	No	-30%		**0**
		Standard Water Supply	No	-10%		
		Fully Supervised System	No	-10%		
				Cumulative Total	**0%**	
5	**Exposure Surcharge (cumulative %)**				**Surcharge**	
	(3)	North Side	> 45.1m		0%	**2,338**
		East Side	> 45.1m		0%	
		South Side	10.1 - 20 m		15%	
		West Side	20.1 - 30 m		10%	
				Cumulative Total	**25%**	
Results						
6	(1) + (2) + (3)	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**12,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**200**
				or	USGPM	3,170
7	**Storage Volume**	Required Duration of Fire Flow (hours)			Hours	2.5
		Required Volume of Fire Flow (m^3)			m^3	1800

FUS - Fire Flow Calculations

As per 1999 Fire Underwriter's Survey Guidelines

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands
Date: 7/20/2022
Input By: Brendan Rundle
Reviewed By: Sam Bahia / Ben Sweet

Legend: Input by User; No Information or Input Required

Building Description: Block 76, 2 Storey Towns
Wood frame

Step			Input		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Wood frame	Yes	1.5	1.5	
		Ordinary construction		1		
		Non-combustible construction		0.8		
		Modified Fire resistive construction (2 hrs)		0.6		
		Fire resistive construction (> 3 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	1408			
		Number of Floors/Storeys	2			
		Area of structure considered (m^2)			**2,816**	
	F	**Base fire flow without reductions**				**18,000**
		$F = 220\ C\ (A)^{0.5}$				
Reductions or Surcharges						
3	**Occupancy hazard reduction or surcharge**			**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**15,300**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**			**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	No	-30%		**0**
		Standard Water Supply	No	-10%		
		Fully Supervised System	No	-10%		
				Cumulative Total	**0%**	
5	**Exposure Surcharge (cumulative %)**				**Surcharge**	
	(3)	North Side	10.1 - 20 m		15%	**6,120**
		East Side	10.1 - 20 m		15%	
		South Side	> 45.1m		0%	
		West Side	20.1 - 30 m		10%	
				Cumulative Total	**40%**	
Results						
6	(1) + (2) + (3)	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**21,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**350**
				or	USGPM	5,548
7	**Storage Volume**	Required Duration of Fire Flow (hours)			Hours	4.5
		Required Volume of Fire Flow (m^3)			m^3	5670

M:\2021\121153\DATA\Calculations\Sewer Calcs\Water\20220720-FUS.xlsx

WATER DEMAND DESIGN SHEET

Novatech Project #:	121153
Project Name:	Stinson Lands
Date Prepared:	7/20/2022
Date Revised:	
Input By:	Brendan Rundle
Reviewed By:	Sam Bahia
Drawing Reference:	121153-GP

Legend:
PROJECT SPECIFIC INFO
USER DESIGN INPUT
CALCULATED AVERAGE DAY CELL OUTPUT
CALCULATED BASIC DAY CELL OUTPUT
CALCULATED MAX DAY CELL OUTPUT
CALCULATED PEAK HOUR CELL OUTPUT
CALCULATED MAX DAY + RFF CELLOUTPUT

NOVATECH
Engineers, Planners & Landscape Architects

LOCATION		TOTAL WATER DEMAND																						
		RESIDENTIAL DEMAND & AVERAGE DEMAND							INDUSTRIAL / COMMERCIAL / INSTITUTIONAL (ICI) INPUT & AVERAGE DEMAND						BASIC DAY DEMAND DAILY VOLUME (m³)	MAXIMUM DAY DEMAND			PEAK HOUR DEMAND			DESIGN FIRE DEMAND			
									INDUST. AREA													REQUIRED FIRE FLOW (RFF)			
NODE	SMALL SYSTEM *	SINGLES	SEMIS/ TOWNS	APTS (2BR)	APTS (1BR)	APTS (AVG)	POP. EQUIV.	RES. AVERAGE DAY FLOW DEMAND (L/s)	LIGHT (ha.)	HEAVY (ha.)	COMM. AREA (ha.)	INST. AREA (ha.)	OTHER AREA (m²)	ICI AVERAGE DAY FLOW DEMAND (L/s)		RES. PEAKING FACTOR	ICI PEAKING FACTOR	MAX DAY FLOW DEMAND (L/s)	RES. PEAKING FACTOR	ICI PEAKING FACTOR	PEAK HOUR FLOW DEMAND (L/s)	RFF 1 FUS (L/min)	RFF 2 FUS (L/min)	RFF 3 OBC / NFPA (L/min)	MAX DAY + GOVERNING RFF (L/s)
J1	YES	9					30.6	0.10	0.00	0.00	0.00	0.00	0.00	0.00	6.1	3.60	1.80	0.36	5.40	3.24	0.54	10,000			167.02
J2	YES	9					30.6	0.10	0.00	0.00	0.00	0.00	0.00	0.00	6.1	3.60	1.80	0.36	5.40	3.24	0.54	10,000			167.02
J3	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J4	YES	13					44.2	0.14	0.00	0.00	0.00	0.00	0.00	0.00	8.8	3.60	1.80	0.52	5.40	3.24	0.77	10,000			167.18
J5	YES	16					54.4	0.18	0.00	0.00	0.00	0.00	0.00	0.00	10.9	3.60	1.80	0.63	5.40	3.24	0.95	10,000			167.30
J6	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J7	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J8	YES	6	16				63.6	0.21	0.00	0.00	0.00	0.00	0.00	0.00	12.7	3.60	1.80	0.74	5.40	3.24	1.11	10,000			167.41
J9	YES	9	12				63.0	0.20	0.00	0.00	0.00	0.00	0.00	0.00	12.6	3.60	1.80	0.74	5.40	3.24	1.10	10,000			167.40
J10	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J11	YES		15				40.5	0.13	0.00	0.00	0.00	0.00	0.00	0.00	8.1	3.60	1.80	0.47	5.40	3.24	0.71	10,000			167.14
J12	YES		30				81.0	0.26	0.00	0.00	0.00	0.00	0.00	0.00	16.2	3.60	1.80	0.95	5.40	3.24	1.42	10,000			167.61
J13	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J14	YES		12				32.4	0.11	0.00	0.00	0.00	0.00	0.00	0.00	6.5	3.60	1.80	0.38	5.40	3.24	0.57	10,000			167.04
J15	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J16	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J17	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
J18	YES						0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.60	1.80	0.00	5.40	3.24	0.00	10,000			166.67
SUB-TOTAL	YES	62	85	0	0	0	440.3	1.43	0.00	0.00	0.00	0.00	0.00	0.00	88.1	3.60	1.80	5.14	5.40	3.24	7.71	10,000	0	0	171.80

DEMAND PARAMETERS

	Residential				
Unit Type Population Equiv.	Singles	Semis/ Towns	Apts (2-BR)	Apts (1-BR)	Apts (AVG)
	3.4	2.7	2.1	1.4	1.8
Daily Demand	L/per person/day				
Average Demand	280				
Basic Demand	200				

Res. Peaking Factors		Max Day (x Avg Day)	Peak Hour (x Avg Day)
Population > 500		2.50	5.50
Population < 500*	Pop.	Small Systems *	
Ref: MECP DWS Guideline Table 3-3	0	9.50	14.30
	30	9.50	FALSE
	150	4.90	7.40
* Note: Use Drop Down List at ea Node.	300	3.60	5.40
YES	450	3.00	4.50
NO	500	2.90	4.30

Institutional / Commercial / Industrial				
Industrial		Commercial	Institutional	Other Use**
Light	Heavy			
L/gross ha/day				L/m²/day
35,000	55,000	28,000	28,000	5
10,000	17,000	17,000	17,000	3

ICI Peaking Factors	Max Day (x Avg Day)	Peak Hour (x Avg Day)
	1.80	3.24

**Note: Custom Designer deifined input/parameter

Vulnerable Service Area (VSA) Review ***
< 50 m³/day
> 50 m³/day

***Note: Designer to review Node / Total VSA.

Quick Fire Flow Reference Guide ****			
FUS (L/min)	Comments	OBC (L/min)	Comments
> 2,000	Min FUS	< 9,000	Unsprinklered Non- Combustible
	Low Density - Singles/Towns		
10,000	Complies w/ TB2014-01 Cap. (10m rear spacing, 6 units max, <600 m²)		
13,000	Non-complying w/TB2014-01. Calculate.		
15,000	**Medium Density** Back-to-back Towns.		
	High Density		
20,000	Wood Frame 4-Storey		
5,000	Fire-Resisitve Podium/Multi-Storey		
30,000	High Contiguous / Hazard Areas		
< 45,000	Max FUS		

****Note: Designer to confirm RFF @ each node using FUS / OBC. Use Novatech FUSv2-0 and OBCv2-0 or NFPA.

MAX PRESSURE DURING AVDY CONDITIONS

Novatech Project #: 121153
Project Name: Stinson Lands
Date Prepared: 7/20/2022
Date Revised:
Input By: Brendan Rundle
Reviewed By: Sam Bahia
Drawing Reference: 121153-GP

FUTURE CONDITIONS

NODE	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)
J01	88.30	0.10	148.20	59.90	85
J02	88.25	0.10	148.20	59.95	85
J03	88.23	0.00	148.20	59.97	85
J04	88.52	0.14	148.20	59.68	85
J05	89.13	0.18	148.20	59.07	84
J06	90.75	0.00	148.20	57.45	82
J07	90.38	0.00	148.20	57.82	82
J08	89.58	0.21	148.20	58.62	83
J09	88.30	0.20	148.20	59.90	85
J10	87.86	0.00	148.20	60.34	86
J11	88.08	0.13	148.20	60.12	85
J12	92.00	0.26	148.20	56.20	80
J13	93.35	0.00	148.20	54.85	78
J14	91.52	0.11	148.20	56.68	81
J15	88.74	0.00	148.20	59.46	85
J16	88.20	0.00	148.20	60.00	85
J17	85.90	0.00	148.20	62.30	89
J18	85.90	0.00	148.20	62.30	89

EXISTING CONDITIONS

NODE	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)
J01	88.30	0.10	156.60	68.30	97
J02	88.25	0.10	156.60	68.35	97
J03	88.23	0.00	156.60	68.37	97
J04	88.52	0.14	156.60	68.08	97
J05	89.13	0.18	156.60	67.47	96
J06	90.75	0.00	156.60	65.85	94
J07	90.38	0.00	156.60	66.22	94
J08	89.58	0.21	156.60	67.02	95
J09	88.30	0.20	156.60	68.30	97
J10	87.86	0.00	156.60	68.74	98
J11	88.08	0.13	156.60	68.52	97
J12	92.00	0.26	156.60	64.60	92
J13	93.35	0.00	156.60	63.25	90
J14	91.52	0.11	156.60	65.08	93
J15	88.74	0.00	156.60	67.86	96
J16	88.20	0.00	156.60	68.40	97
J17	85.90	0.00	156.60	70.70	101
J18	85.90	0.00	156.60	70.70	101

MIN PRESSURE DURING PKHR CONDITIONS

Novatech Project #:	121153
Project Name:	Stinson Lands
Date Prepared:	7/20/2022
Date Revised:	
Input By:	Brendan Rundle
Reviewed By:	Sam Bahia
Drawing Reference:	121153-GP

FUTURE CONDITIONS

NODE	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)
J01	88.30	0.54	141.57	53.27	76
J02	88.25	0.54	141.57	53.32	76
J03	88.23	0.00	141.57	53.34	76
J04	88.52	0.77	141.57	53.05	75
J05	89.13	0.95	141.57	52.44	75
J06	90.75	0.00	141.57	50.82	72
J07	90.38	0.00	141.57	51.19	73
J08	89.58	1.11	141.57	51.99	74
J09	88.30	1.10	141.57	53.27	76
J10	87.86	0.00	141.57	53.71	76
J11	88.08	0.71	141.58	53.50	76
J12	92.00	1.42	141.58	49.58	71
J13	93.35	0.00	141.58	48.23	69
J14	91.52	0.57	141.58	50.06	71
J15	88.74	0.00	141.59	52.85	75
J16	88.20	0.00	141.59	53.39	76
J17	85.90	0.00	141.60	55.70	79
J18	85.90	0.00	141.59	55.69	79

EXISTING CONDITIONS

NODE	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)
J01	88.30	0.54	139.57	51.27	73
J02	88.25	0.54	139.57	51.32	73
J03	88.23	0.00	139.57	51.34	73
J04	88.52	0.77	139.57	51.05	73
J05	89.13	0.95	139.57	50.44	72
J06	90.75	0.00	139.57	48.82	69
J07	90.38	0.00	139.57	49.19	70
J08	89.58	1.11	139.57	49.99	71
J09	88.30	1.10	139.57	51.27	73
J10	87.86	0.00	139.57	51.71	74
J11	88.08	0.71	139.58	51.50	73
J12	92.00	1.42	139.58	47.58	68
J13	93.35	0.00	139.58	46.23	66
J14	91.52	0.57	139.58	48.06	68
J15	88.74	0.00	139.59	50.85	72
J16	88.20	0.00	139.59	51.39	73
J17	85.90	0.00	139.60	53.70	76
J18	85.90	0.00	139.59	53.69	76

AVAILABLE FLOW AT 20psi DURING MXDY + FF CONDITIONS

Novatech Project #:	121153
Project Name:	Stinson Lands
Date Prepared:	7/20/2022
Date Revised:	
Input By:	Brendan Rundle
Reviewed By:	Sam Bahia
Drawing Reference:	121153-GP

FUTURE CONDITIONS

NODE	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)	FIRE FLOW DEMAND (L/s)	FIRE FLOW DEMAND (L/min)	AVAILABLE FLOW (L/min)
J01	88.30	0.36	144.28	55.98	80	167	10000	9042
J02	88.25	0.36	144.28	56.03	80	167	10000	10368
J04	88.52	0.52	144.28	55.76	79	167	10000	10410
J05	89.13	0.63	144.28	55.15	78	167	10000	10020
J08	89.58	0.74	144.28	54.70	78	167	10000	10344
J09	88.30	0.74	144.28	55.98	80	167	10000	10662
J11	88.08	0.47	144.28	56.20	80	167	10000	11508
J12	92.00	0.95	144.28	52.28	74	167	10000	10368
J14	91.52	0.38	144.28	52.76	75	167	10000	10896

EXISTING CONDITIONS

NODE	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)	FIRE FLOW DEMAND (L/s)	FIRE FLOW DEMAND (L/min)	AVAILABLE FLOW (L/min)
J01	88.30	0.36	146.52	58.22	83	167	10000	9732
J02	88.25	0.36	146.52	58.27	83	167	10000	10944
J04	88.52	0.52	146.52	58.00	82	167	10000	10986
J05	89.13	0.63	146.52	57.39	82	167	10000	10590
J08	89.58	0.74	146.52	56.94	81	167	10000	10932
J09	88.30	0.74	146.52	58.22	83	167	10000	11244
J11	88.08	0.47	146.52	58.44	83	167	10000	12114
J12	92.00	0.95	146.52	54.52	78	167	10000	10998
J14	91.52	0.38	146.52	55.00	78	167	10000	11538

1. Background

On behalf of the City of Ottawa, the National Research Council of Canada (NRC) evaluated the City's hydrant spacing guidelines in relation to Required Fire Flow (RFF) as calculated using the Fire Underwriters Survey (FUS) methodology. This work lead to the development of a procedure to be used to establish the appropriate sizing of, and hydrant spacing on, dead-end watermains. This procedure may also be used as an optional watermain network design method to optimize watermain sizing based on RFF and standard hydrant spacing.

The procedure is partially based on the NFPA 1: Fire Code (NFPA1) and the City of Ottawa existing hydrant classification practice (refer to **Attachment A** at the end of this appendix for relevant excerpts of the Fire Code).

2. Rationale for Guideline

Given a Required Fire Flow (RFF) for a certain asset/structure/building, proper planning must ensure that there is a sufficient number of hydrants at sufficient proximities to actually provide the RFF. Both the capacity of the hydrants and their proximity to the asset/structure/building must be considered. Pressure losses (due to friction) in firehoses are proportional to the firehose length. Therefore, the actual fire flow delivered by the nozzle at the end of a very long firehose will be less compared to a short firehose connected to the same hydrant. Table 1 provides conservative values for hydrant fire flow capacity adjusted for firehose length.

3. Hydrant Capacity Requirement

For the purposes of this guidelines, the aggregate fire flow capacity of all contributing fire hydrants within 150 m of a building/asset/structure[1], measured in accordance with Table 1, shall be not less than the RFF.

4. Standard Practice

For the vast majority of developments, hydrant spacing as indicated in Section 4.5, Table 4.9, Ottawa Design Guidelines – Water Distribution, are sufficient to meet the RFF. This has been verified by evaluating approved development plans representing a

[1] Although NFPA 1 considers hydrant contribution at distances of up to 1000ft (305 m), Ottawa Fire Services (OFS) would need two pumpers to deliver flow from such a distance (one pumper midway – acting as a booster). Moreover, OFS cautioned that some redundancy is advisable to account for accessibility limitations in emergency situations, wind effects, etc. Therefore 150 m was considered as the maximum contributing distance

range of land uses and configurations. However, in some instances involving dead-end watermains, standard spacing requirements may not be sufficient to meet RFF.

Standard design practice involves systematic checking of design fire flows at every node in hydraulic models of proposed water distribution systems. Normally the entire design fire flow is applied to each node in succession. Nodes are typically at water main junctions rather than actual hydrant locations. This significantly simplifies the design process and the current software packages that are normally used for this purpose have been developed based on this practice. The "point load assumption" produces a conservative design.

Table 1. Maximum flow to be considered from a given hydrant

Hydrant Class	Distance to asset/structure/building (m)[a]	Contribution to required fire flow (L/min)[b]
AA	≤ 75	5,700
	> 75 and ≤ 150	3,800
A	≤ 75	3,800
	> 75 and ≤ 150	2,850
B	≤ 75	1,900
	> 75 and ≤ 150	1,500
C	≤ 75	800
	> 75 and ≤ 150	800

[a] Distance of contributing hydrant from the structure, measured in accordance with NFPA 1 (Appendix A).

[b] Maximum flow contribution to be considered for a given asset/structure/building, at a residual pressure of 20 psi, measured at the location of the main, at ground level.

4. Intended Application of Guideline

The intent of this procedure is to:

- Determine the appropriate sizing of dead end watermains and associated hydrant requirements.
- Provide an optional approach to local watermain network sizing that will assist the designer in determining the minimum pipe sizing needed to meet RFF.

The procedure permits the designer to: (a) reconcile available hydrant flow with computed RFFs, and (b) allow the distribution of RFFs along multiple hydrants, rather than consider RFF to be a point flow. The application of this protocol may result in reduced watermain diameters compared to those determined based on a traditional design approach. Caution is required in the application of the procedure to ensure that the transmission function of any watermains identified in a Master Servicing Study is not compromised. Normally, watermains 300mm in diameter and larger that are identified in such studies would not be considered for resizing.

5. Application Procedure

5.1 Rated hydrants

The procedure described here would apply to an existing watermain network with existing hydrants (i.e., re-development or infill in existing neighborhoods):

- Identify critical zones within the (re)development area, e.g., high RFF, dead ends, small diameter watermains, low C factor, and/or high geographic elevation zones.
- For the critical zones use Table 1 to examine if there are sufficient hydrants to deliver the RFF (following procedure described in 5.3).
- If hydrant capacity is insufficient, then consider either:
  - adding hydrants as appropriate;
  - determine if the existing hydrants can be upgraded to higher rating; or
  - upgrade existing watermains.

5.2 Un-rated hydrants

There are currently about 24,800 hydrants in the City of Ottawa, of which about 78% are rated. Of the rated hydrants, 96% are AA (Blue), 3% are A (Green). Many of the un-rated hydrants are located in old parts of the City, often installed on water mains with minimum diameter of 6" (150 mm), and would be likely to have a low rating.

Based on a review of hydrants that have been installed as part of recent urban development, approximately 99% of those which were rated are rated AA, and only 1% are rated A.

5.2.1 Un-rated Existing Hydrants

In cases where fire flow is to be evaluated in areas with an established water distribution network and with existing fire hydrants (i.e., re-development or infill in existing neighborhoods), all un-rated hydrants should be tested and rated in accordance with NFPA standard 291. The procedure described in Section 5.1 can then be followed to complete the design.

5.2.2 Planned hydrants

Planned hydrants cannot be tested for rating because they have not been installed yet. Moreover, the rating of a hydrant is an intrinsic property of the hydrant and can therefore not be directly evaluated by simulation. Based on the statistics cited previously, it can be assumed for design purposes that all planned hydrants are AA. However, there could be a situation where the proposed network might not have sufficient capacity to supply 5,700 L/min to a AA-rated hydrant in a specific area. Hydraulic analysis is required to confirm that the distribution network is capable of providing the hydrants with the fire flows in Table 1.

5.3 Hydrant Placement and Watermain Size Optimization

Ottawa design guidelines for watermain sizing and hydrant placement (Section 4) stipulate that the RFF be added to the average hourly rate of a peak day demand. This fire flow is added to hydraulic nodes in the vicinity of the planned development, while ensuring that the residual pressure is at least 140 kPa (measured at the location of the main, at ground level).[2] The following procedure is used to optimize watermain sizing and hydrant placement based on the RFF.

- Place hydrants throughout the development area according to the current Ottawa design guidelines.
- Size water mains and locate hydrants according to standard design procedures. Assume all hydrants are AA-rated.
- Identify the most critical zones in the development area, e.g. highest required fire flows, dead ends, longest distances between junctions, and/or highest elevation. Within these critical zones identify critical structures, i.e. those with highest RFF or greatest distance from proposed hydrant locations. Identify the closest hydrants to these buildings.
- For each critical structure, distribute the RFF according to Table 1 (i.e., assign a flow of 5,700 L/min to all hydrants with a distance of less or equal to 75 m from the test property and 3,800 L/min to all hydrants with a distance of more than 75 m but less or equal to 150 m from the test property) These hydrants are to be represented as hydrant-nodes in the network model, where the hydrant lateral would connect to the proposed water main.

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[2] At the time when this protocol was proposed, the City of Ottawa had in effect Technical Bulletin ISDTB 2014-02, whereby RFF may be capped at 10,000 L/min for single detached dwellings (with a minimum 10 m separation between the backs of adjacent units and for side-by-side town and row houses that comply with the OBC Div. B, subsection 3.1.10 requirement (compartments of no more than 600 m^2 area).

- For each critical structure, run a single fire flow simulation ensuring that the RFF is provided by hydrants within 150 m distance from the test property, with a minimum residual pressure of 140 kPa.
- If the required residual pressure cannot be achieved, consider either re-sizing of pipes, and/or re-spacing of hydrants.

The above procedure is optional <u>except</u> for dead-end watermains servicing cul-de-sacs because (a) based on standard spacing requirements, there would often be insufficient fire flow provided and (b) the watermain would otherwise could be sized larger than necessary and lead to excessive water age and on-going flushing requirements.

Irrespective of the above, if the RFF is equal to or less than 10,000 L/min, then:

- where the distance between two adjacent hydraulic nodes is greater than the inter-hydrant spacing allowed in the guideline, a hydraulic node should be added halfway between the two nodes, and proceed with fire flow simulations to verify watermain sizing, ensuring that the simulation considers RFF at the new hydraulic node.

Attachment A—Excerpts from NFPA 1 Fire Code (2015 Edition)

18.5 Fire Hydrants.

18.5.1 Fire Hydrant Locations and Distribution. Fire hydrants shall be provided in accordance with Section 18.5 for all new buildings, or buildings relocated into the jurisdiction unless otherwise permitted by 18.5.1.1 or 18.5.1.2.

18.5.1.4* The distances specified in Section 18.5 shall be measured along fire department access roads in accordance with 18.2.3.

18.5.1.5 Where fire department access roads are provided with median dividers incapable of being crossed by fire apparatus, or where fire department access roads have traffic counts of more than 30,000 vehicles per day, hydrants shall be placed on both sides of the fire department access road on an alternating basis, and the distances specified by Section 18.5 shall be measured independently of the hydrants on the opposite side of the fire department access road.

18.5.1.6 Fire hydrants shall be located not more than 12 ft (3.7 m) from the fire department access road.

18.5.2 Detached One- and Two-Family Dwellings. Fire hydrants shall be provided for detached one- and two-family dwellings in accordance with both of the following:

(1) The maximum distance to a fire hydrant from the closest point on the building shall not exceed 600 ft (183 m).

(2) The maximum distance between fire hydrants shall not exceed 800 ft (244 m).

18.5.3 Buildings Other than Detached One- and Two-Family Dwellings. Fire hydrants shall be provided for buildings other than detached one- and two-family dwellings in accordance with both of the following:

(1) The maximum distance to a fire hydrant from the closest point on the building shall not exceed 400 ft (122 m).

(2) The maximum distance between fire hydrants shall not exceed 500 ft (152 m).

18.5.4 Minimum Number of Fire Hydrants for Fire Flow.

18.5.4.1 The minimum number of fire hydrants needed to deliver the required fire flow for new buildings in accordance with Section 18.4 shall be determined in accordance with Section 18.5.4.

18.5.4.2 The aggregate fire flow capacity of all fire hydrants within 1000 ft (305 m) of the building, measured in accordance with 18.5.1.4 and 18.5.1.5, shall be not less than the required fire flow determined in accordance with Section 18.4.

18.5.4.3* The maximum fire flow capacity for which a fire hydrant shall be credited shall be as specified by Table 18.5.4.3. Capacities exceeding the values specified in Table 18.5.4.3 shall be permitted when local fire department operations have the ability to accommodate such values as determined by the fire department.

Table 18.5.4.3 Maximum fire flow hydrant capacity

Distance to buildings[a]		Maximum capacity[b]	
(ft)	(m)	(gpm)	(L/min)
≤ 250	≤ 76	1500	5678
> 250 and ≤ 500	> 76 and ≤ 152	1000	3785
> 500 and ≤ 1000	> 152 and ≤ 305	750	2839

[a] Measured in accordance with 18.5.1.4 and 18.5.1.5.

[b] Minimum 20 psi (139.9 kPa) residual pressure.

18.5.4.4 Fire hydrants required by **18.5.2** and **18.5.3** shall be included in the minimum number of fire hydrants for fire flow required by **18.5.4**.

The City of Ottawa design guidelines on hydrant classification conform to the NFPA Standard #291, which recommends the following:

5.1 Classification of Hydrants. Hydrants should be classified in accordance with their rated capacities [at 20 psi (1.4 bar) residual pressure or other designated value as follows:

(1) Class AA — Rated capacity of 1500 gpm (5700L/min) or greater

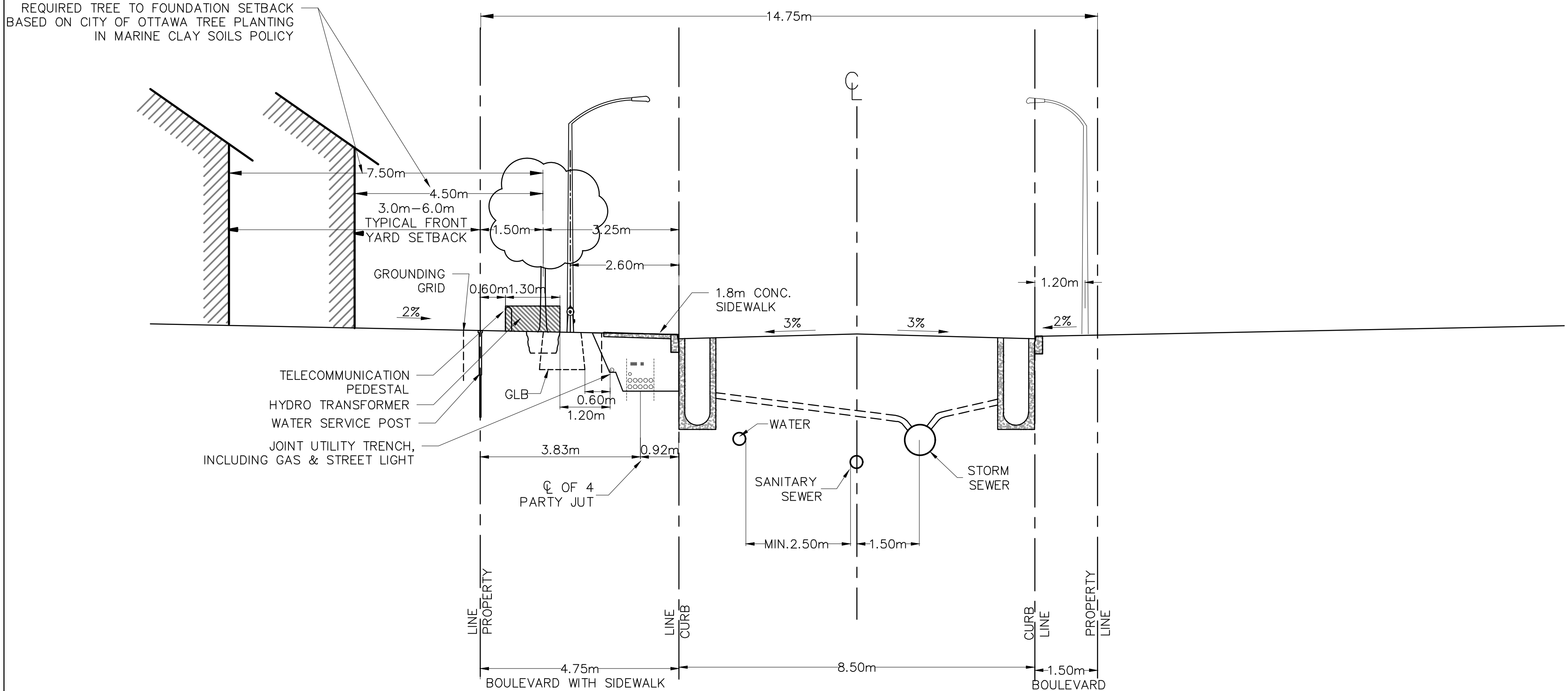
(2) Class A — Rated capacity of 1000–1499 gpm (3800– 5699 L/min)

(3) Class B — Rated capacity of 500–999 gpm (1900–3799 L/min)

(4) Class C — Rated capacity of less than 500 gpm (1900 L/min)

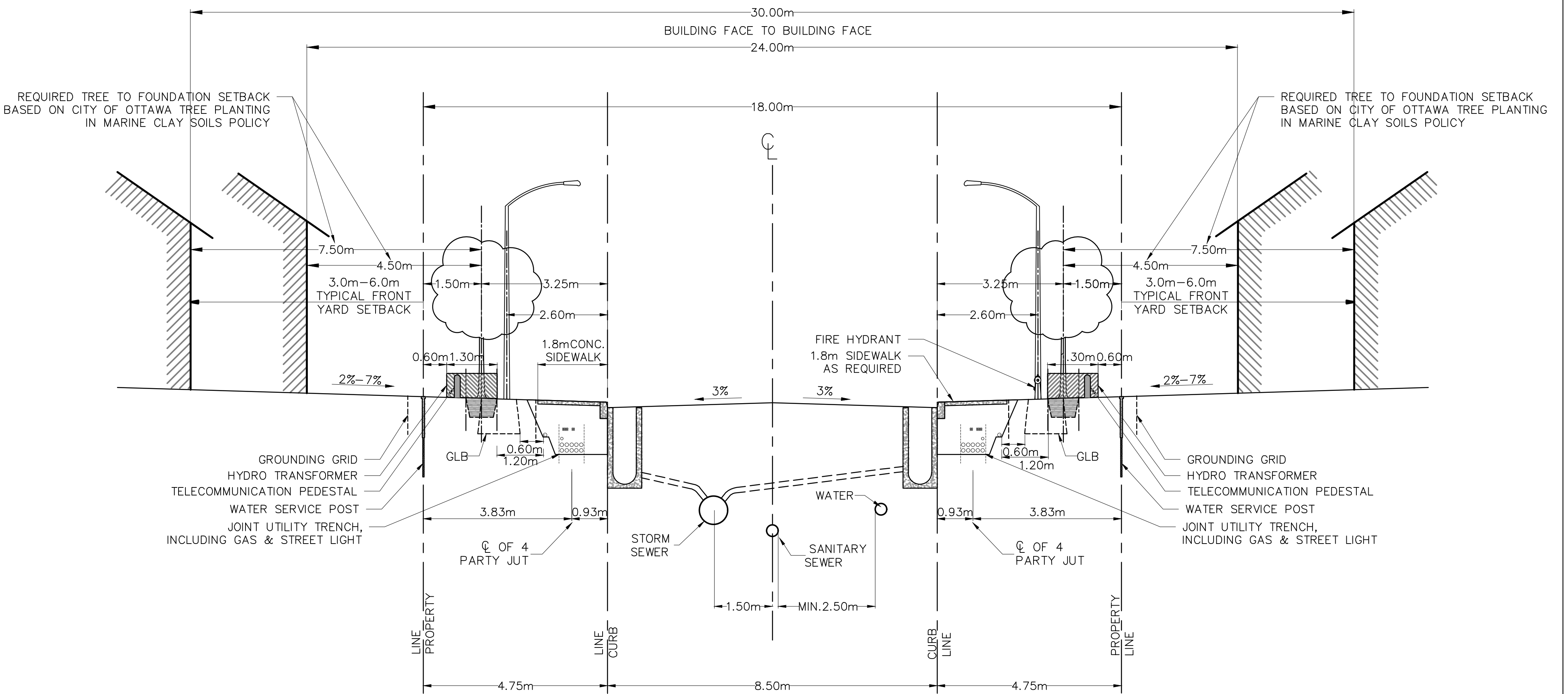
Appendix F
Geotechnical Investigation (soft copy)

Appendix G
Cross-Sections (City of Ottawa Standards)



1. REFERENCE THE GENERAL STANDARD CROSS–SECTION NOTES.
2. CONCRETE CURBS TO BE CONSTRUCTED AS PER CITY OF OTTAWA STANDARD DETAILS.
3. 3m–6m TYPICAL FRONT YARD SETBACK IS TO BE CLEAR AND UNENCUMBERED OF ANY SUBSURFACE BUILDING ENCROACHMENTS.
4. HYDRANTS TO BE LOCATED ON THE WATERMAIN SIDE OF THE STREET.
5. CATCH BASINS TO BE IN–ROAD, NOT CURB INLET TYPE.
6. 14.75m RIGHT–OF–WAY NOT TO BE USED ON STREETS WITH OC TRANSPO BUS SERVICES.
7. GAS MAIN SHALL HAVE A MINIMUM OF 0.6m CLEARANCE FROM STRUCTURES (e.g. CATCH BASINS AND HYDRANTS).
8. STREET LIGHTS CAN BE LOCATED ON EITHER SIDE OF THE RIGHT–OF–WAY.
9. 4 PARTY JOINT USE UTILITY TRENCH (JUT) UNDER SIDEWALK AS PER HYDRO OTTAWA UDS0049 CONSTRUCTION DETAIL, TOTAL WIDTH AT BASE = ±1635mm. EDGE OF HYDRO DUCTS PLACED 1.0m FROM EDGE OF TREE ROOT BALL.
10. GRADE LEVEL BOX (GLB) AS DRAWN SHOWS GLB3660 WITH THE FOLLOWING DIMENSIONS: 43" WIDTH AT BASE, 36" WIDTH AT SURFACE, 36" DEPTH. **EXACT LOCATION TO BE CONFIRMED.**
11. THIS CROSS–SECTION CANNOT BE USED WHERE A CONCRETE ENCASED HYDROELECTRIC DUCT IS REQUIRED.

14.75m ROW (WINDOW STREET)
LAST MODIFIED ON 2022-05-11



1. REFERENCE THE GENERAL STANDARD CROSS–SECTION NOTES.
2. CONCRETE CURBS TO BE CONSTRUCTED AS PER CITY OF OTTAWA STANDARD DETAILS.
3. 3m–6m TYPICAL FRONT YARD SETBACK IS TO BE CLEAR AND UNENCUMBERED OF ANY SUBSURFACE BUILDING ENCROACHMENTS.
4. HYDRANTS TO BE LOCATED ON THE WATERMAIN SIDE OF THE STREET.
5. CATCH BASINS TO BE IN–ROAD, NOT CURB INLET TYPE.
6. 18m RIGHT–OF–WAY NOT TO BE USED ON STREETS WITH OC TRANSPO BUS SERVICES.
7. GAS MAIN SHALL HAVE A MINIMUM OF 0.6m CLEARANCE FROM STRUCTURES (e.g. CATCH BASINS AND HYDRANTS).
8. STREET LIGHTS CAN BE LOCATED ON EITHER SIDE OF THE RIGHT–OF–WAY.
9. 4 PARTY JOINT USE UTILITY TRENCH (JUT) UNDER SIDEWALK AS PER HYDRO OTTAWA UDS0049 CONSTRUCTION DETAIL, TOTAL WIDTH AT BASE = ±1635mm. EDGE OF HYDRO DUCTS PLACED 1.0m FROM EDGE OF TREE ROOT BALL.
10. GRADE LEVEL BOX (GLB) AS DRAWN SHOWS GLB3660 WITH THE FOLLOWING DIMENSIONS: 43" WIDTH AT BASE, 36" WIDTH AT SURFACE, 36" DEPTH. EXACT LOCATION TO BE CONFIRMED.
11. THIS CROSS–SECTION CANNOT BE USED WHERE A CONCRETE ENCASED HYDROELECTRIC DUCT IS REQUIRED.

18m ROW (STANDARD)
LAST MODIFIED ON 22-05-11