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Stinson Lands
4386 Rideau Valley Drive

Conceptual Site Servicing & Stormwater Management Report

39 Years

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
Revised April 22, 2024
Revised December 20, 2024
Revised March 11, 2025

Novatech File: 121153

NOVATECH
Engineers, Planners & Landscape Architects

March 11, 2025

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**
Conceptual Site Servicing and Stormwater Management Report
Novatech File No.: 121153
City Planning File No.: D07-16-22-0026

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

The report has been prepared to demonstrate that the proposed Draft Plan of Subdivision can be serviced with the existing sewers, watermain, drainage outlet and utilities fronting the site. This report has been prepared based on the pre-consultation meeting and discussions with the City of Ottawa.

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

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 Background

This report assesses 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 in **Appendix H.**

The Subject Site is located at the northwest corner of Rideau Valley Drive and Bankfield Road. The Subject 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 of Subdivision also includes a parcel east of Rideau Valley Drive and bounded to the east by the Rideau River. The Subject Site's approval shall be divided into Phase 1 and Phase 2; notwithstanding this report is intended to support the Draft Plan application for both phases.

The existing land use consists of a single residential building and three barns. The land 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 in **Appendix H**. 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 Subject Site.

1.2 Development Intent

The overall Subject Site will comprise of residential dwellings, public right-of-ways (ROW), open space blocks, park blocks, servicing / road widening blocks, as shown in **Table 1.1.1.** The proposed development concept is shown on **Figure 1.3** – Site Plan in **Appendix H**. Phase 1 will consist of 41 single family dwellings, 4 semi-detached units, and 10 townhome units, and a park block. Phase 2 will consist of 21 single family dwellings, 10 semi-detached units, and 63 townhome units. The development has been phased as a result of the City's request to phase the draft approval based on sanitary capacity within the Manotick Pumping Station. The initial phase shall be limited to 55 units and the second phase shall be the remaining subject site buildout as shown in **Table 1.1.2.**

Table 1.1.1: Land Use, Development Potential, and Yield (Overall)

Unit Type	Number of Units	Area (ha)
Singles	62	3.07
Semis	14	0.36
Townhomes	73	1.67
Open Space & Park Blocks	-	3.01
Local Roads	-	2.05
Servicing and Road Widening	-	0.23
TOTAL	**149**	**10.28**

Table 1.1.2: Phased Unit Count and Land Use

Phase	Unit Count				Gross Area (ha)
	Single Family	Semi-Detached	Row Townhome	Total Unit Count	
1	41	4	10	55	3.34
Ph1 Open Space/Park/Other	-	-	-	-	3.63
2	21	10	63	94	3.15
Ph 2 Open Space/Other	-	-	-	-	0.16
Total	**62**	**14**	**73**	**149**	**10.28**

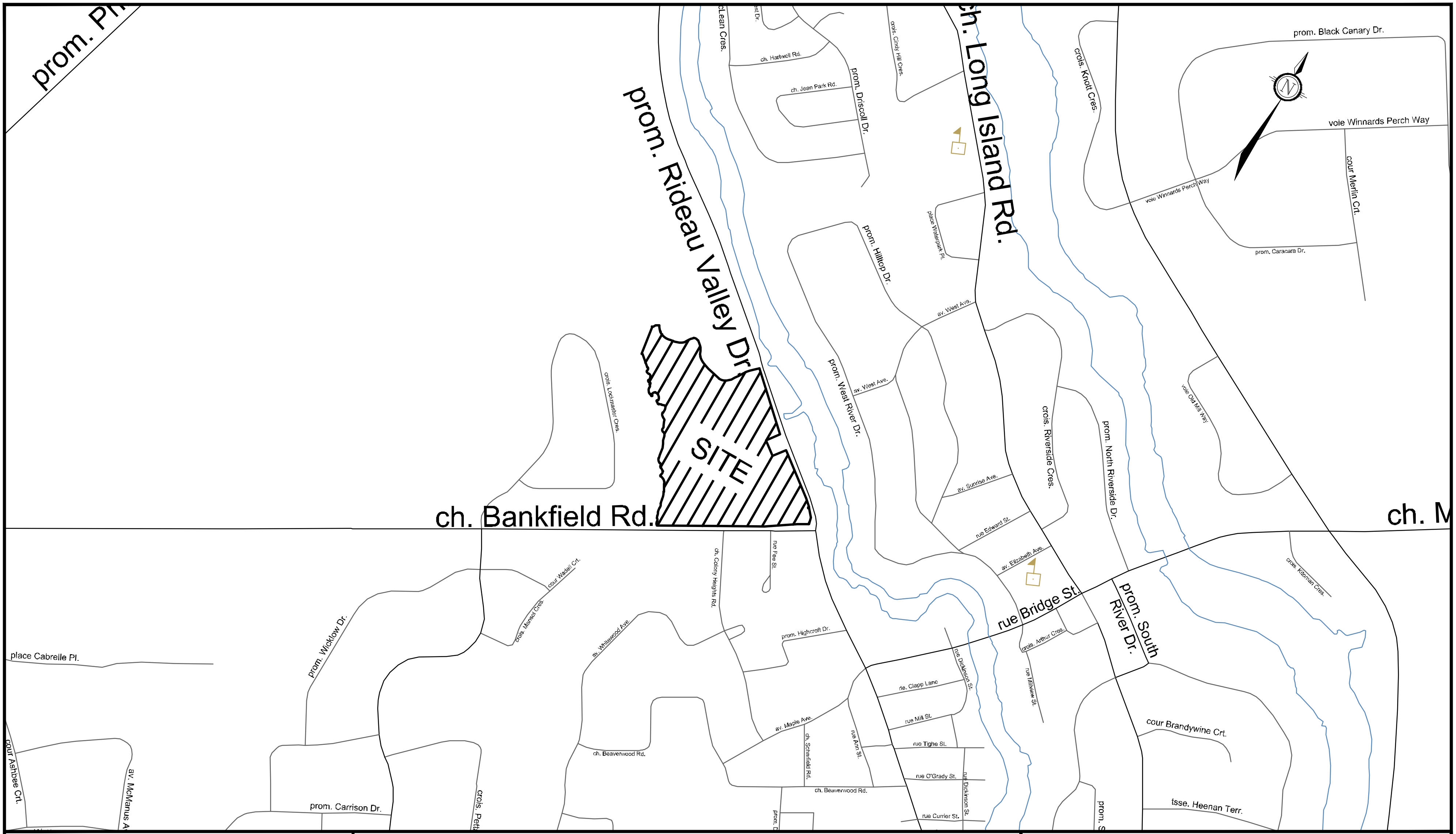
The Subject Site is located within the public service area in the Official Plan of the City of Ottawa and the Secondary Plan of the Village of Manotick; therefore, the site has been designed with municipal water and sanitary sewage collection. The development will contain City of Ottawa municipal road allowances of 14.75 and 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 of Subdivision 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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Telephone	(613) 254-9643
Facsimile	(613) 254-5867
Website	www.novatech-eng.com

CITY OF OTTAWA
STINSON LANDS
4386 RIVER ROAD

KEY PLAN

SCALE	NOT TO SCALE	
DATE MAR 2025	JOB 121153	FIGURE 1.1

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

SHT8X11.DWG - 216mmx279mm

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Website	www.novatech-eng.com

LEGEND

🟩	DEVELOPMENT LIMIT
🟪	BOUNDARY
—	WATERCOURSE

CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

EXISTING CONDITIONS

SCALE
NOT TO SCALE

DATE	JOB	FIGURE
MAR 2025	121153	1.2

M:\2021\121153\CAD\Design\121153-EX.dwg, Fig 1.2, Mar 21, 2024 - 4:21pm, brundle

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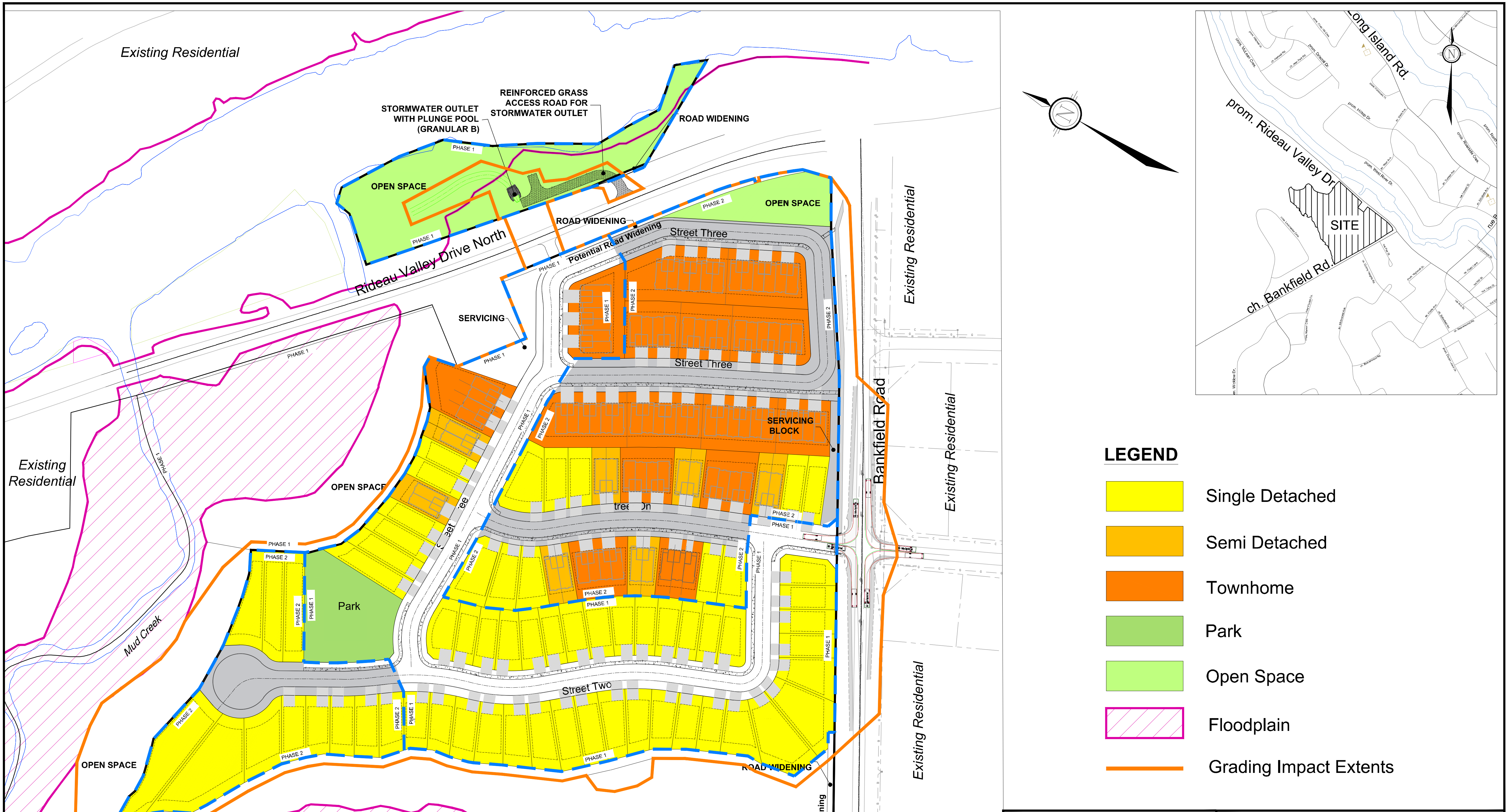

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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

SITE PLAN WITH GRADING IMPACT EXTENTS

SCALE 1:2000

DATE MAR 2025

JOB 121153

FIGURE 1.3

M:\2021\121153\CAD\Design\121153-BP.dwg, FIG 1.3 Site Plan, Dec 06, 2024 - 3:58pm, pkanani

SHT11X17.DWG - 279mmX432mm

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, & 2019-02)
  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 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 April 4, 2024.

Based on the geotechnical study, it is not anticipated that there will be any significant geotechnical concerns with respect to servicing and developing the Subject Site. Refer to **Figure 2.1** for the test hole locations and **Figure 2.2** for the permissible grade raise restrictions, both located in **Appendix H**. A summary of the geotechnical report findings is provided in **Table 2.1** below.

Table 2.1: Summary of Geotechnical Servicing and Grading Considerations

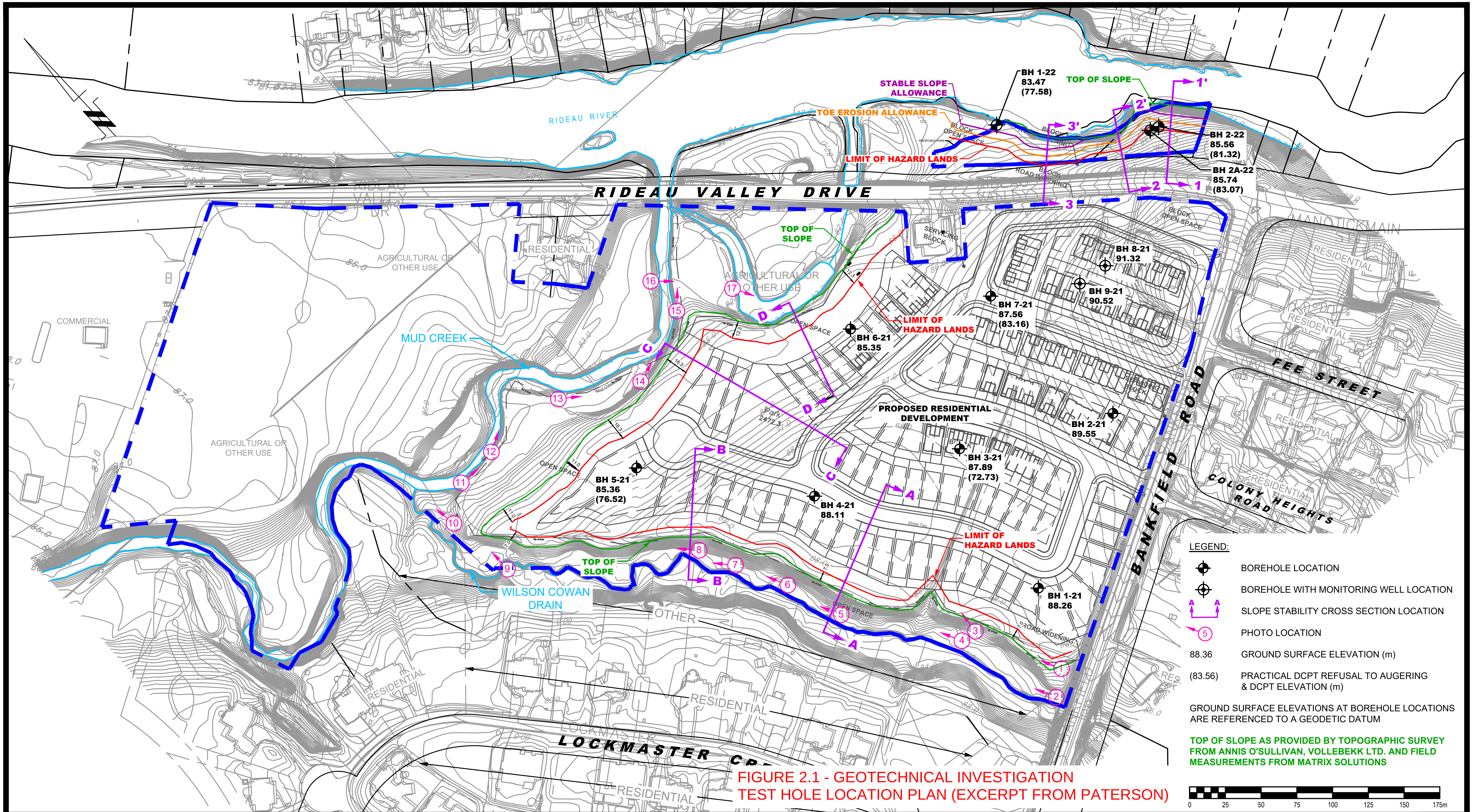
Parameter	Summary
Sub-Soil Conditions	Topsoil underlain by a deposit of silty clay (hard to stiff weathered crust) and glacial till
Grade Raise Restrictions	Refer to Figure 2.2 Alternate methods of increasing the permissible grade raise could include preloading/surcharging the areas where required or lightweight fill.
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 completed 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 was completed 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. Report No. 35268-504, April 22, 2024.

The above report findings and recommendations have been considered in establishing the development limits of the Draft Plan of Subdivision and to address erosion potential due to increased stormwater flows as a result of the development.



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

0 25 50 75 100 125 150 175m

FIGURE 2.1 - GEOTECHNICAL INVESTIGATION
TEST HOLE LOCATION PLAN (EXCERPT FROM PATERSON)

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

NO.	REVISIONS	DATE	INITIAL
4	SITE PLAN UPDATED	04/04/2024	SK
3	WATER COURSES NAME UPDATED	20/10/2023	MS
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: KP	Dwg No.: PG5828-1
Approved by: DJG	Revision No.: 4

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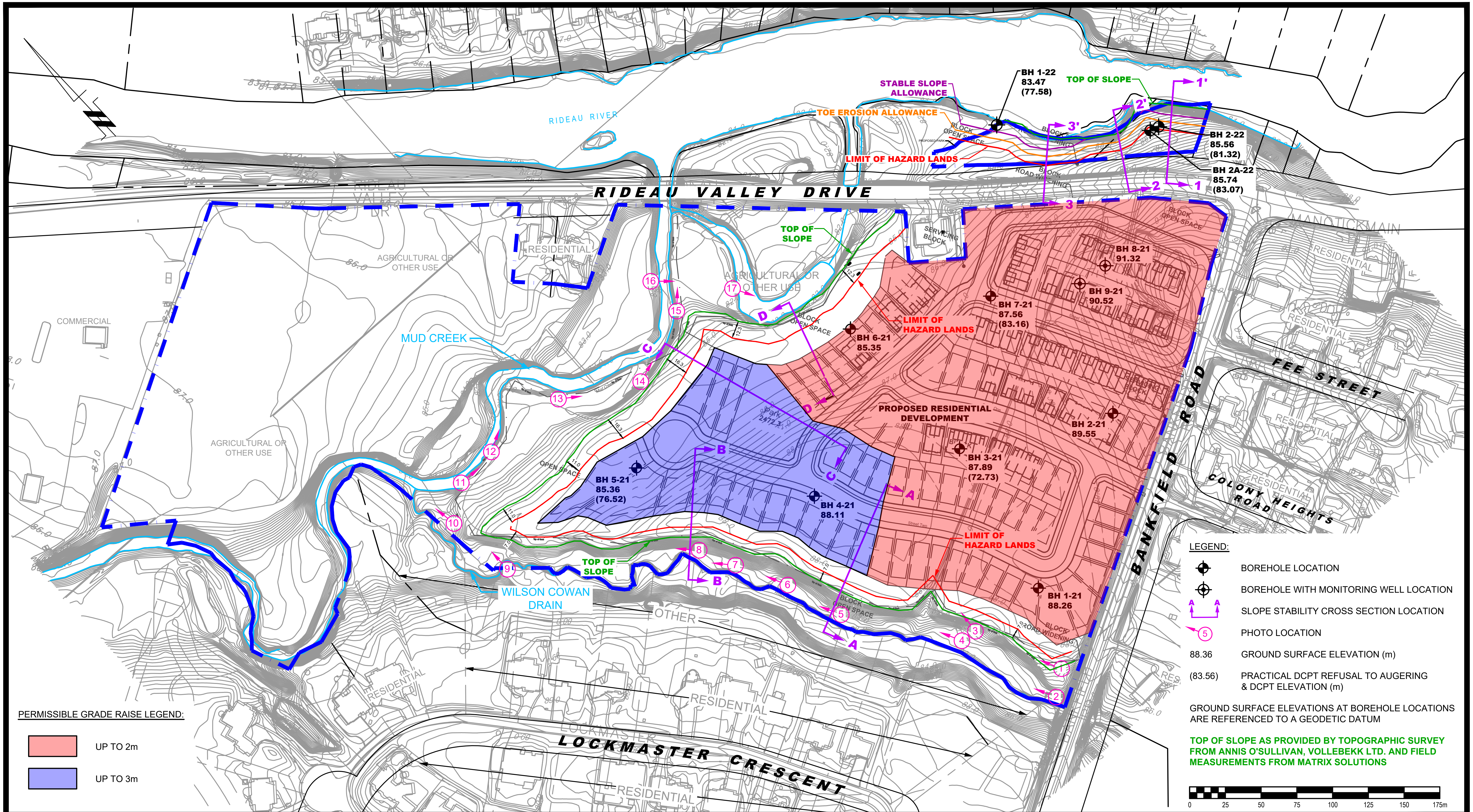

PERMISSIBLE GRADE RAISE LEGEND:

🟥	UP TO 2m
🟦	UP TO 3m

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
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TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:2500
Date: 04/2024
Drawn by: NFRV
Report No.: PG5828-1
Checked by: KP
Dwg No.: PG5828-3
Approved by: DJG
Revision No.:

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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. In order to service the Subject Site, the local sanitary sewers and watermain will need to connect to existing infrastructure along Rideau Valley Drive. The local storm sewers will connect to the proposed stormwater outlet that will cross Rideau Valley Drive to convey flows from the Subject Site to the Rideau River.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services for the off-site servicing located in **Appendix H**.

3.2 General Servicing

The Subject Site will be serviced using local storm and sanitary sewers, and watermains. As per the above, to service the Subject Site the local sanitary sewers and watermain will need to connect to existing infrastructure along Rideau Valley Drive. Local storm sewers will connect to the proposed stormwater outlet that will cross Rideau Valley Drive.

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

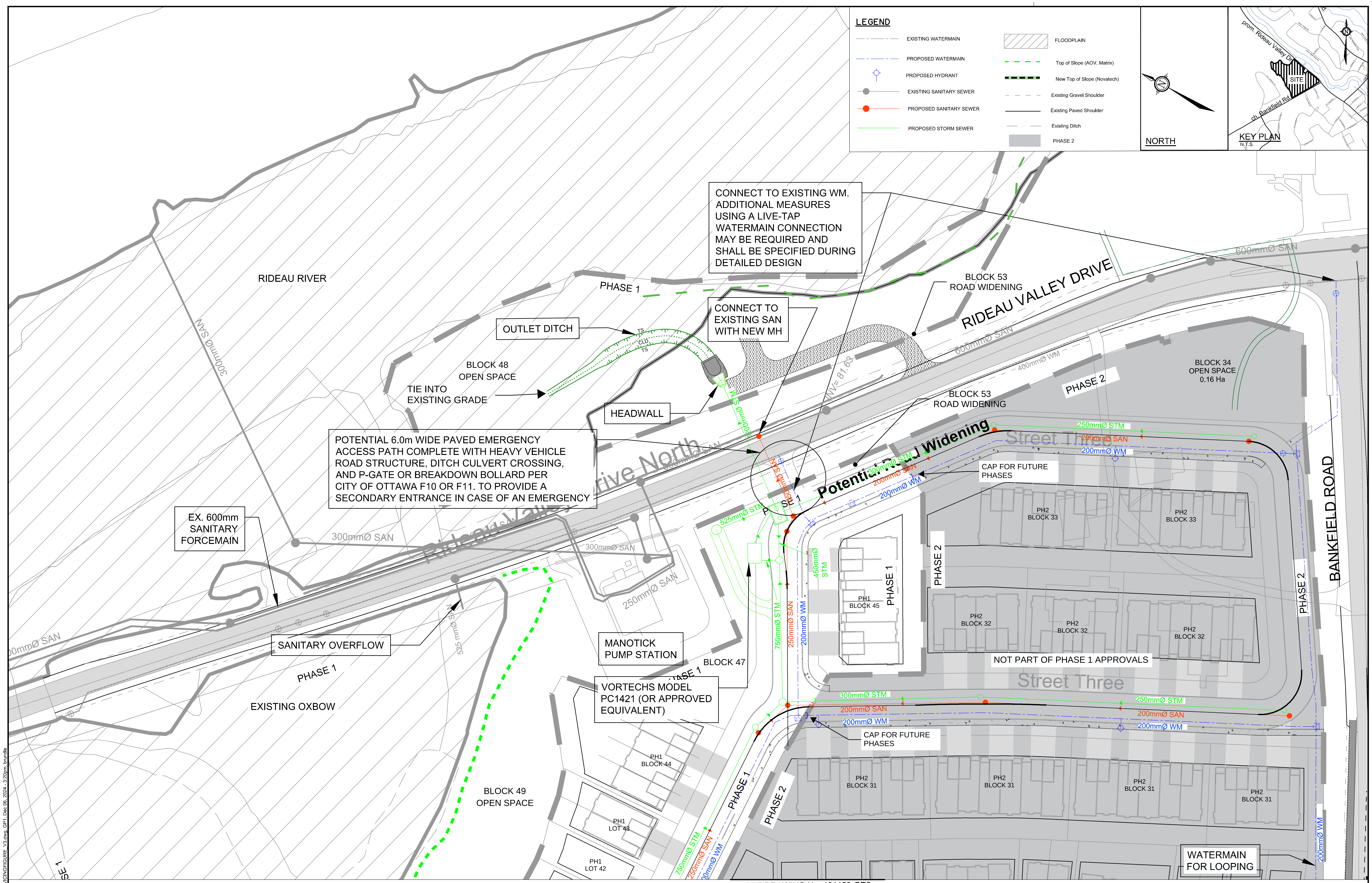
Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services for the on-site servicing located in **Appendix H**.

3.3 General Grading

The grading will direct emergency overland flows from the local roads towards a proposed ditch inlet catchbasin (DICB) located within Block 47, beside the existing Manotick Pump Station. The DICB will convey flows to the stormwater outlet for the Subject Site, ultimately outletting into the Rideau River. In the event of an emergency blockage, the overland flows will be conveyed within the existing roadside ditch on the southwest side of Rideau Valley Drive and outlet into the Oxbox Ditch.

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 macro grading located in **Appendix H**.



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.

No.	REVISION	DATE	BY
4.	ISSUED FOR RE-SUBMISSION REVIEW	MAR 3/25	BHB
3.	ISSUED FOR RE-SUBMISSION REVIEW	DEC 20/24	BHB
2.	ISSUED FOR RE-SUBMISSION REVIEW	APR 10/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

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CHECKED BHB
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CHECKED BHB
APPROVED BHB

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Ottawa, Ontario, Canada K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES 1

PROJECT No. 121153-00
REV REV #4
DRAWING No. 121153-GP1

PLAN41.DWG - 841mmx594mm

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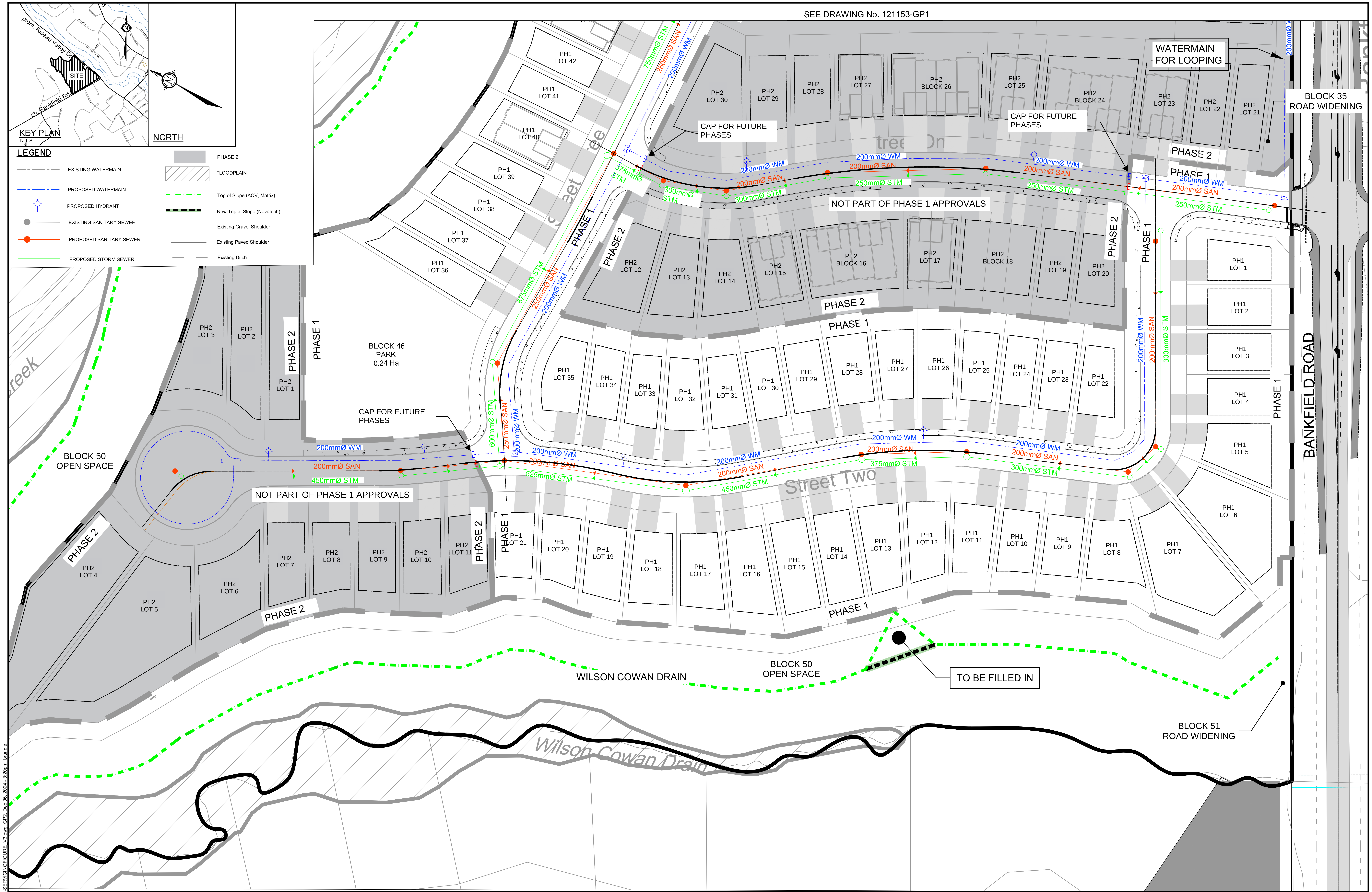

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Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES 2

PROJECT No. 121153-00
REV REV #4
DRAWING No. 121153-GP2

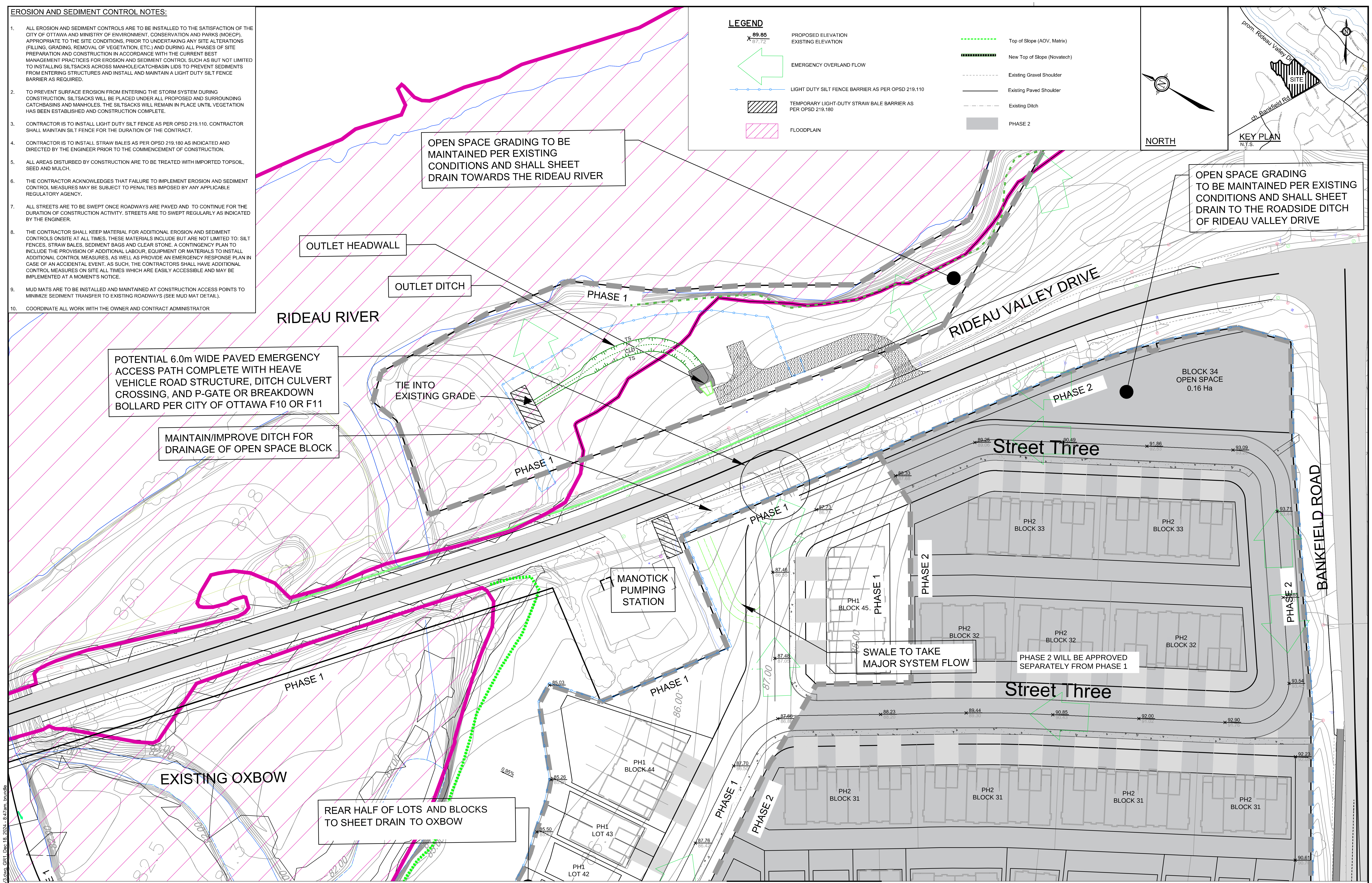
M:\2021\121153\CAD\Design\Figures\121153-SERVICINGFIGURE_V3.dwg, GP2, Dec 09, 2024 - 3:20pm, bbundle

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
▬	PHASE 2



SEE DRAWING No. 121153-GR2

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LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GRADING, EROSION, & SEDIMENT CONTROL PLAN 1

PROJECT No. 121153-00
REV REV #5
DRAWING No. 121153-GR1

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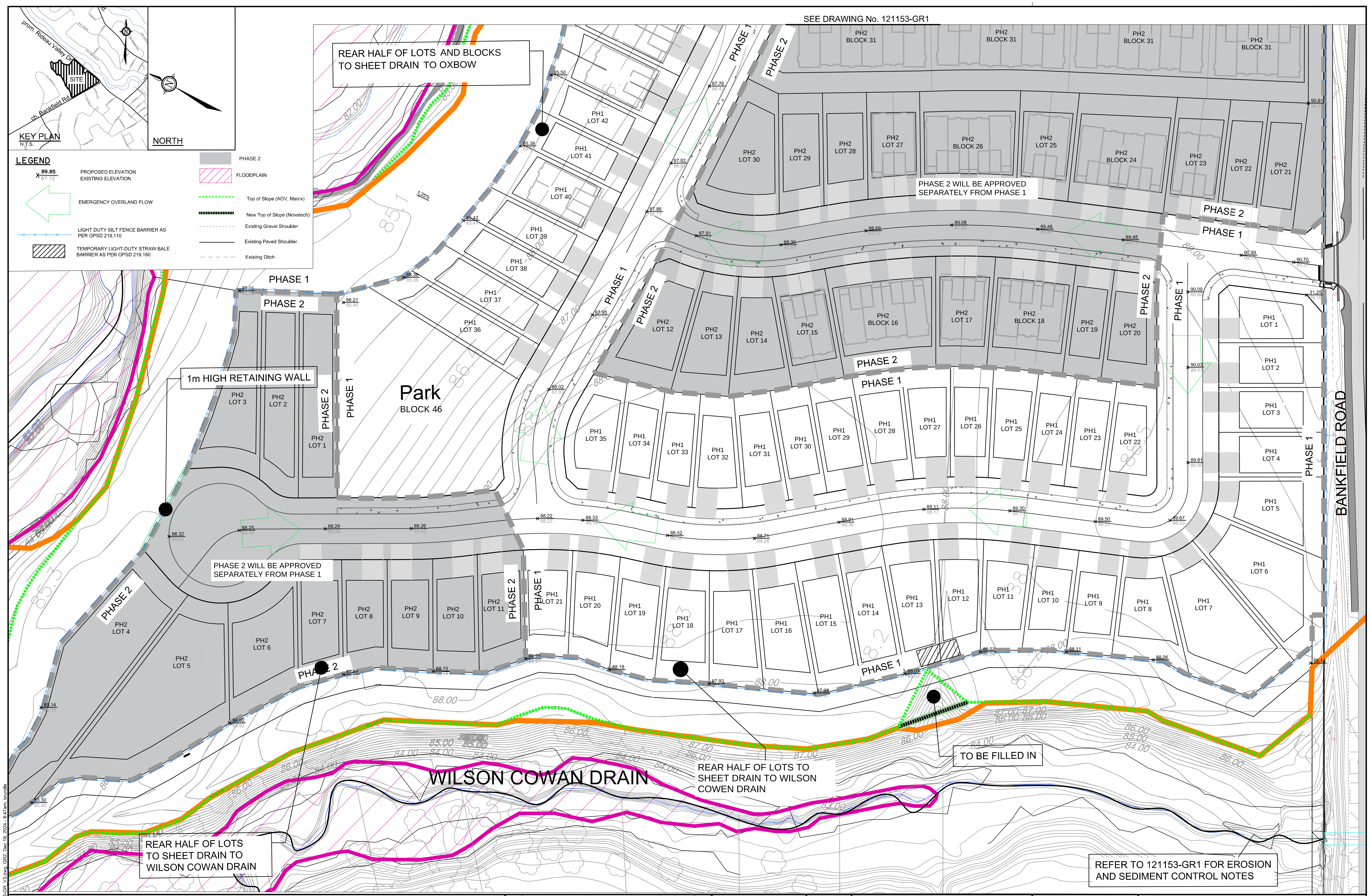

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE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

No.	REVISION	DATE	BY
5.	ISSUED FOR RE-SUBMISSION REVIEW	MAR 3/25	BHB
4.	ISSUED FOR RE-SUBMISSION REVIEW	DEC 13/24	BHB
3.	ISSUED FOR RE-SUBMISSION REVIEW	OCT 23/24	BHB
2.	ISSUED FOR RE-SUBMISSION REVIEW	APR 11/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE
1:500

1:500
0 5 10 15 20

DESIGN	BPR
CHECKED	BHB
DRAWN	BPR
CHECKED	BHB
APPROVED	BHB

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LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GRADING, EROSION, & SEDIMENT CONTROL PLAN 2

PROJECT No.
121153-00

REV
REV # 5

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 existing roadside ditch on the southwest side of Rideau Valley Drive. Refer to **Figure 1.2** – Existing Conditions in **Appendix H**.

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 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 Rideau River. 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 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 existing 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 for Phase 1 will involve the installation of approximately 677 meters of pipe with diameters ranging from 250 mm to 1050 mm. The proposed works for Phase 2 will involve the installation of approximately 473 meters of pipe, with diameters ranging from 250 mm to 450 mm. 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. In the event that the storm sewer system becomes obstructed, the majority of major system flows will be routed to MH150 and ultimately the Rideau River. In the event of an emergency blockage, the major system flows will be conveyed within the existing roadside ditch on the southwest side of Rideau Valley Drive and outlet into the Oxbox 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 catch basins;
- 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 WQT installed upstream of the storm outlet to the Rideau River, represented by MH142 in the model. 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 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, the specifications of which were provided by the manufacturer (Contech). A bypass weir will be installed upstream in STM MH-144 to redirect high flows during larger storm events. The invert of the bypass weir has been set based on the 25mm 6-hour Chicago storm HGL in STM MH-144. The length of the bypass weir is equivalent to the internal length of STM MH-144.

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.

The Macro Servicing Plan indicates the proposed lot development limit and top of slope for the existing drains and 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 been 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 Impact to Existing Oxbow Ditch

While there will be a decrease in the peak flows directed to the Oxbow Ditch, it is expected that there will be no adverse impacts to the current function of the Oxbow as the proposed post-development drainage area to the Oxbow Ditch will generate sufficient runoff to maintain the 'normal' water level and retention volume and the Oxbow Ditch will continue to be periodically inundated by backwater from Mud Creek under post-development conditions.

An overview of the water balance calculations was completed in support of the recommended stormwater outlet as a part of the previously submitted memorandum: *4386 Rideau Valley Drive – Stinson Lands, Oxbow Water Balance* (Novatech, April 16, 2024). The memorandum is included in **Appendix C**.

4.4.6 Alterations to Watercourses

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

- Filling in an existing ditch between Lots 12-14.
- A new stormwater outlet to the Rideau River will be required. This stormwater 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 (since refined) memorandum: *4386 Rideau Valley Drive N – Stinson Lands, SWM Strategy Outline* (Novatech, June 8, 2022). The memorandum is included in **Appendix C**.

A post-development model of the proposed subdivision storm sewers and outlet to the Rideau River 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 Subject 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).

<ins>*6 Hour Chicago Distribution*</ins>:

25mm Event (Water Quality)
2-year Event
5-year Event
100-year Event
100-year Event +20%

<ins>*12 Hour SCS Type II Distribution*</ins>:

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

The Rideau River Flood Risk Mapping from Hogs Back to Kars (RVCA, July 17, 2017) report provides details of the HEC-RAS model prepared to analyze the water levels and peak flows within the Rideau River for various storm events. Water levels and peak flows from Table 11 and 12 in the RVCA report are outlined in **Table 4.2**. Cross Section 17595 is the closest to where the subdivision outlets to the Rideau River.

Table 4.2: Downstream Boundary Conditions

Storm Event	Water Level (m)	Peak Flow (cms)
2-year	82.20	117.49
5-year	82.56	148.28
100-year	83.22	212.70

With the proposed outlet invert at 82.48m, only the 5-year and 100-year water levels in the Rideau River have the potential to have a slight impact on the outlet flows. Due to the drop from where the subdivision outlets at MH140 upstream of the WTQ unit to the ultimate outlet at the Rideau River, it is not expected that the downstream boundary conditions will have an impact on the HGL elevations within the storm sewers.

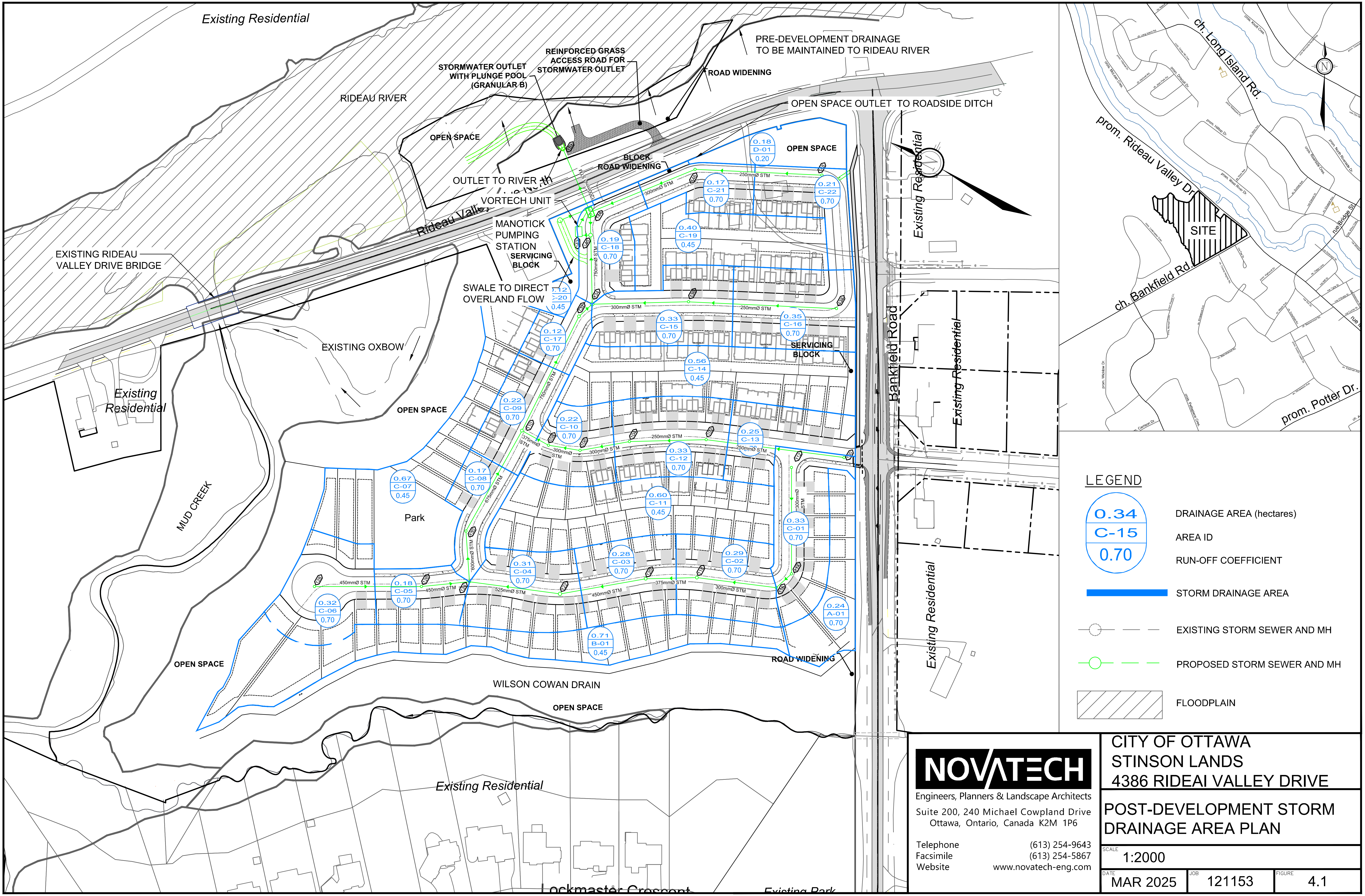
4.5.3 Storm Drainage Areas

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**).

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 **Figure 4.1** - Storm Drainage Area Plan (**112153-STM**) in **Appendix H**. An overview of the modeling parameters is provided in **Table 4.3**.



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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAI VALLEY DRIVE

POST-DEVELOPMENT STORM
DRAINAGE AREA PLAN

SCALE
1:2000

DATE	JOB	FIGURE
MAR 2025	121153	4.1

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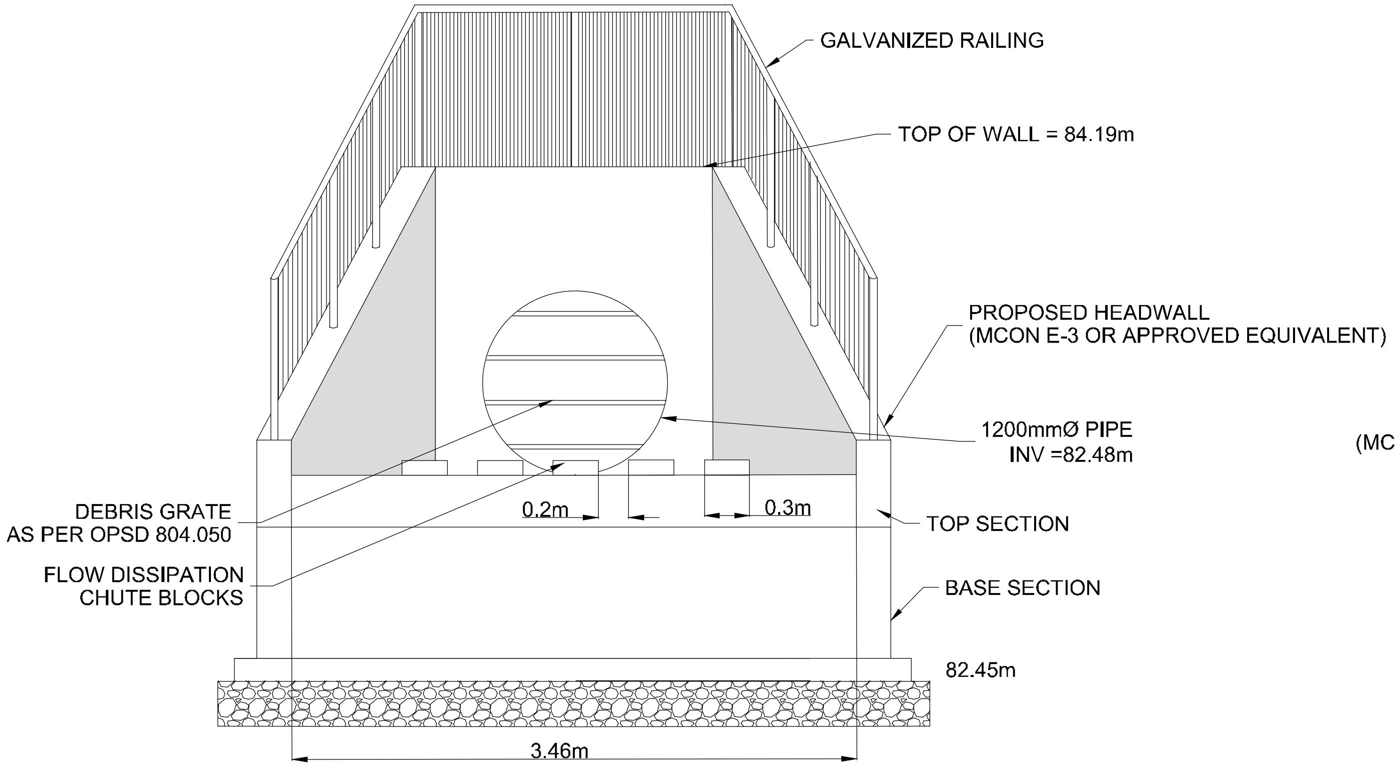

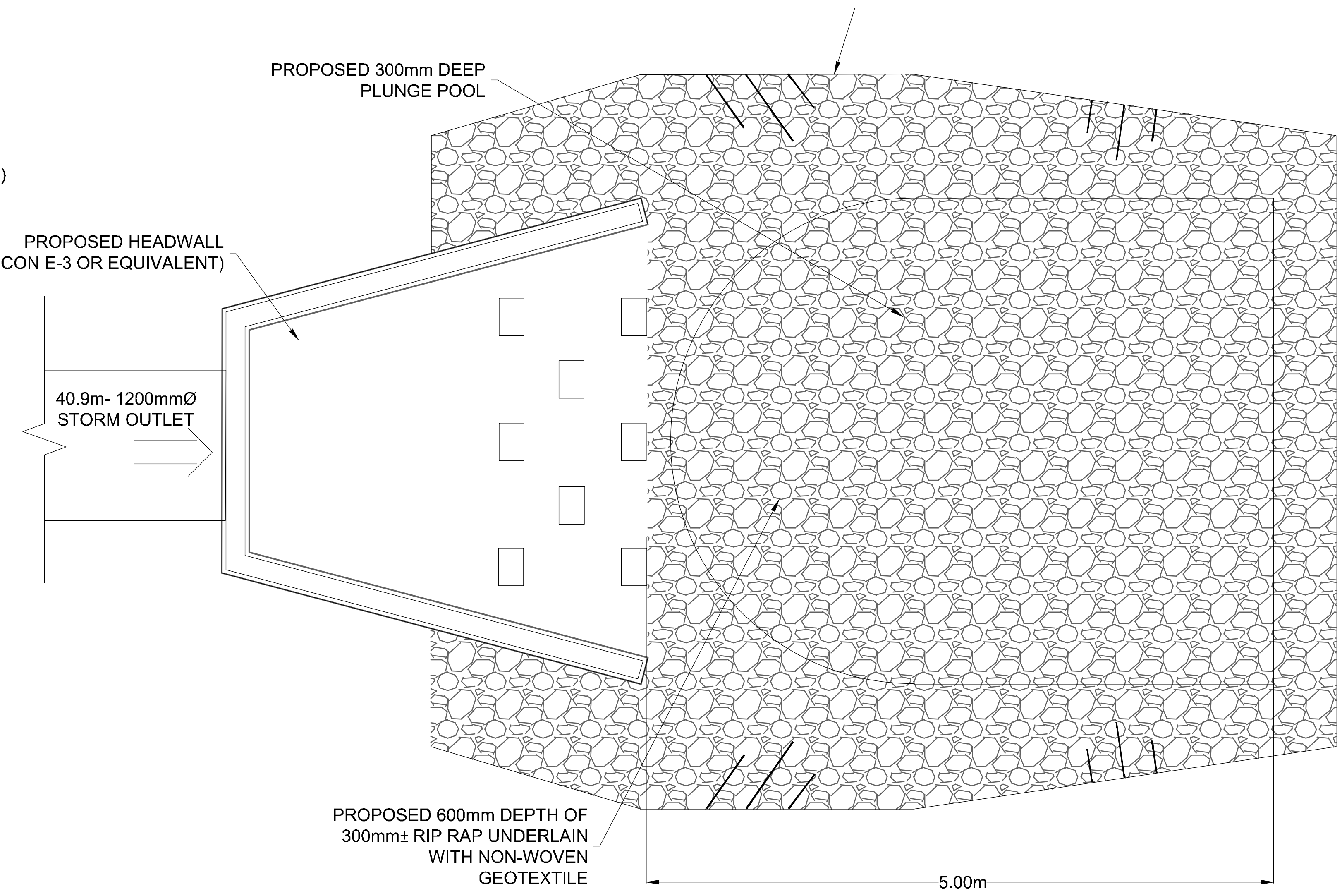

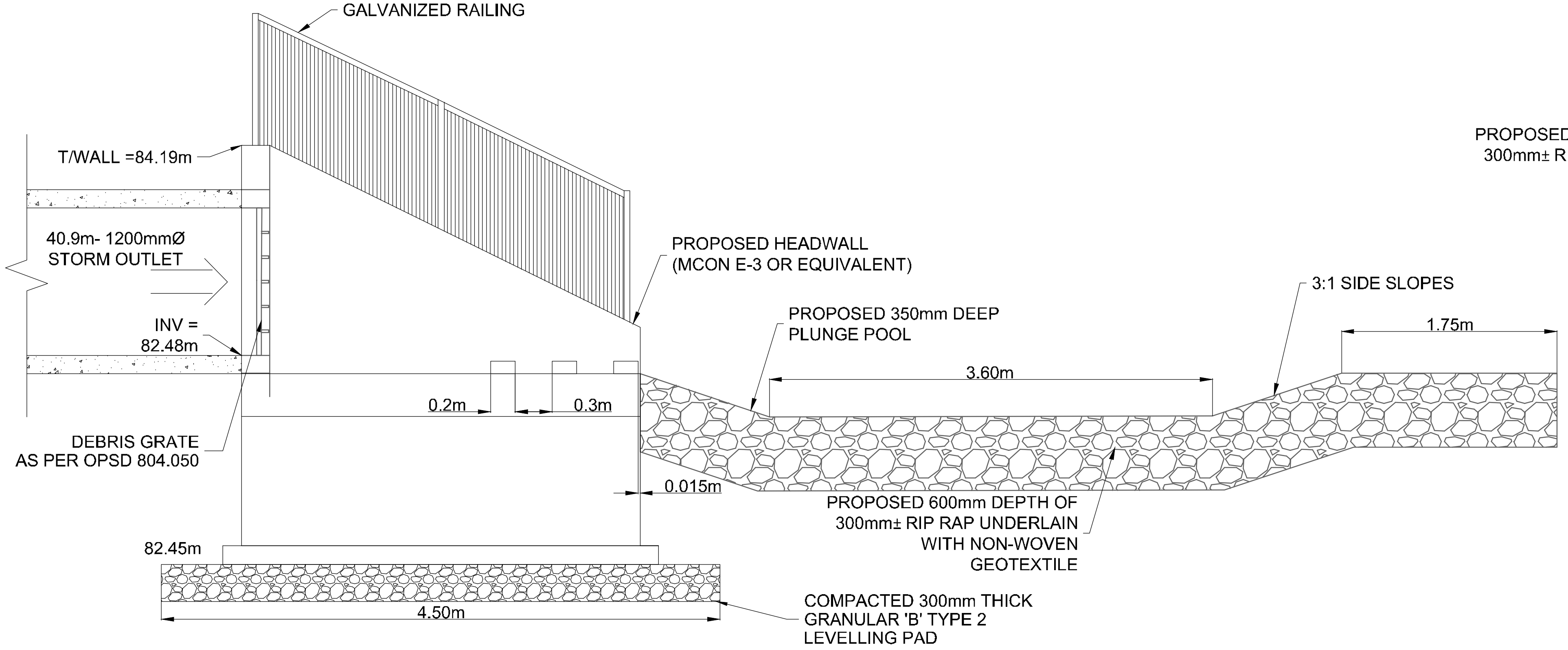

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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

PROPOSED STORM OUTLET
WITH PLUNGE POOL

SCALE
1:50

DATE	JOB	FIGURE
MAR 2025	121153	121153-OUTLET

SHT11X17.DWG - 279mmX432mm

Table 4.3: PCSWMM Subcatchment Area Parameters

Area ID	Catchment Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.240	0.45	36%	100%	25.02	97.54	1.0%
B-01	0.710	0.45	36%	100%	21.31	334.06	1.0%
C-01	0.330	0.70	71%	45%	20.51	161.84	1.0%
C-02	0.290	0.70	71%	45%	24.44	117.42	1.0%
C-03	0.280	0.70	71%	45%	23.37	118.54	1.0%
C-04	0.310	0.70	71%	45%	23.12	135.79	1.0%
C-05	0.180	0.70	71%	45%	23.02	76.46	1.0%
C-06	0.320	0.70	71%	45%	34.25	94.31	1.0%
C-07	0.670	0.45	36%	100%	64.21	106.68	1.0%
C-08	0.170	0.70	71%	45%	22.85	73.96	1.0%
C-09	0.220	0.70	71%	45%	22.23	97.19	1.0%
C-10	0.220	0.70	71%	45%	22.65	98.01	1.0%
C-11	0.600	0.45	36%	100%	19.05	316.00	1.0%
C-12	0.330	0.70	71%	45%	19.65	166.94	1.0%
C-13	0.250	0.70	71%	45%	23.49	106.41	1.0%
C-14	0.560	0.45	36%	100%	14.18	397.06	1.0%
C-15	0.330	0.70	71%	45%	22.08	152.74	1.0%
C-16	0.350	0.70	71%	45%	21.84	160.71	1.0%
C-17	0.120	0.70	71%	45%	22.88	51.13	1.0%
C-18	0.190	0.70	71%	45%	21.60	85.67	1.0%
C-19	0.400	0.45	36%	45%	13.84	289.76	1.0%
C-20	0.120	0.45	36%	0%	22.12	54.25	1.0%
C-21	0.170	0.70	71%	100%	18.95	88.64	1.0%
C-22	0.210	0.70	71%	100%	19.02	111.49	1.0%
D-01	0.180	0.20	0%	0%	20.63	87.76	1.0%

TOTAL: **7.75**

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 **Figure 4.1** - 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**.

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, with the exception of one, are currently on-grade. 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, the Oxbow Ditch, and the Rideau Valley Drive existing roadside ditch 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 Rideau River.

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	36	53	81	182	240	25	44	111
Wilson Cowan Drain	*Pre*	56	140	245	588	767	150	242	506
	Post	50	77	135	339	447	35	78	183
Rideau Valley Drive (culvert)	*Pre*	26	65	118	287	376	64	102	216
	Post	0	1	8	40	60	0	8	29
Rideau River (MH 220)	*Pre*	-	-	-	-	-	-	-	-
	Post	504	750	1,111	1,708	2,067	366	621	1,210

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 (Chicago) (m)	WL Above Obvert (Chicago) (m)
MH100	87.92	90.70	87.92	0.25	88.17	88.08	-0.09
MH102	86.81	89.34	86.81	0.25	87.06	86.99	-0.07
MH104	86.17	88.62	86.17	0.30	86.47	86.87	0.40
MH106	85.63	88.13	85.63	0.38	86.01	86.18	0.18
MH108	85.27	87.85	85.27	0.45	85.72	85.84	0.12
MH110	84.72	87.82	84.72	0.82	85.54	85.62	0.08
MH112	87.53	89.76	87.53	0.30	87.83	87.53	-0.30
MH114	87.03	89.56	87.03	0.30	87.33	87.09	-0.24
MH116	86.91	89.56	86.91	0.30	87.21	87.09	-0.12
MH118	86.47	89.24	86.47	0.38	86.85	86.73	-0.11
MH120	86.09	89.00	86.09	0.52	86.61	86.36	-0.25
MH122	85.63	88.56	85.63	0.60	86.23	86.11	-0.12
MH124	85.19	88.18	85.19	0.60	85.79	85.97	0.18
MH126	85.41	88.20	85.41	0.45	85.86	85.99	0.13
MH128	85.60	88.31	85.60	0.45	86.05	86.00	-0.05
MH130	85.04	87.95	85.04	0.68	85.72	85.84	0.12
MH132	84.49	87.62	84.49	0.82	85.31	85.35	0.04
MH134	84.44	87.55	84.44	0.82	85.26	85.19	-0.07

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)
MH136	86.64	89.40	86.64	0.38	87.02	86.83	-0.19
MH138	90.68	93.25	90.68	0.30	90.98	90.79	-0.19
MH140	83.45	86.17	83.45	1.05	84.50	84.25	-0.25
MH142	82.92	86.64	82.92	1.05	83.97	83.62	-0.35
MH144	82.22	87.91	84.61	0.75	85.36	84.86	-0.50
MH146	87.09	89.29	87.09	0.30	87.39	87.09	-0.30
MH148	90.92	93.22	90.92	0.25	91.17	90.92	-0.25
MH150	83.00	86.38	83.00	0.90	83.90	83.38	-0.52

As shown in the above table, the 100-year HGL elevations are generally 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.

Outlets & Impact

As discussed in **Section** Error! Reference source not found., the majority of the runoff from the Subject Site will be conveyed to the stormwater outlet discharging into the Rideau River, however, there will be some uncontrolled runoff from rear yards and open space / parks to the Wilson Cowan Drain, Oxbow Ditch, and Rideau Valley Drive.

Matrix has reviewed the stormwater outlet discharging into the Rideau River. As outlined within the Fluvial Geomorphic and Erosion Hazard Assessment, Matrix estimated the erosion sensitivity of the receiving floodplain from the stormwater outlet using a permissible velocity approach for observed substrates and selected a critical velocity of 0.91m/s. To ensure that the critical velocity at the outlet is reduced to an acceptable level and there is no risk of erosion at the Rideau River, a plunge pool will be installed. Refer to **Appendix C** for sizing calculations, and **Figure 4.2 - Proposed Outlet with Plunge Pool** in **Appendix H** for the proposed plunge pool design.

Further, as the uncontrolled runoff from rear yards and open space / parks will sheet drain to the Wilson Cowan Drain, Oxbow Ditch, and Rideau Valley Drive, and the post-development flows are less than pre-development (refer to **Table 4.4**), there is not expected to be any concern for erosion in these areas.

During detailed design stage, additional assessment to address erosion mitigation measures will be completed to ensure there will be no negative impacts to the Rideau River, Wilson Cowan Drain, Oxbow Ditch, and Rideau Valley Drive due to the peak flows from the proposed development.

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. A new manhole will be constructed approximately 37 m upstream of existing MHSA58902 within Rideau Valley Drive. From there it will flow through the existing trunk sewer to the existing Manotick Pumping Station located 65m away at 4344 Rideau Valley Drive.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services in **Appendix H** 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 (one operational pump and one 305mm forcemain), however, based on correspondence from City Staff the pumping station is planned to be upgraded to have a capacity of 170 L/s by Q4 2025.

Based on the existing and projected demands of the serviced lands tributary to the existing 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 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 in **Appendix H** 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 170 L/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 the existing Manotick Pump Station can be operated with the larger forcemain during wet weather flows to provide an increased residual flow, in advance of the upgrade.

5.3 Proposed Sanitary Infrastructure

Off-site works

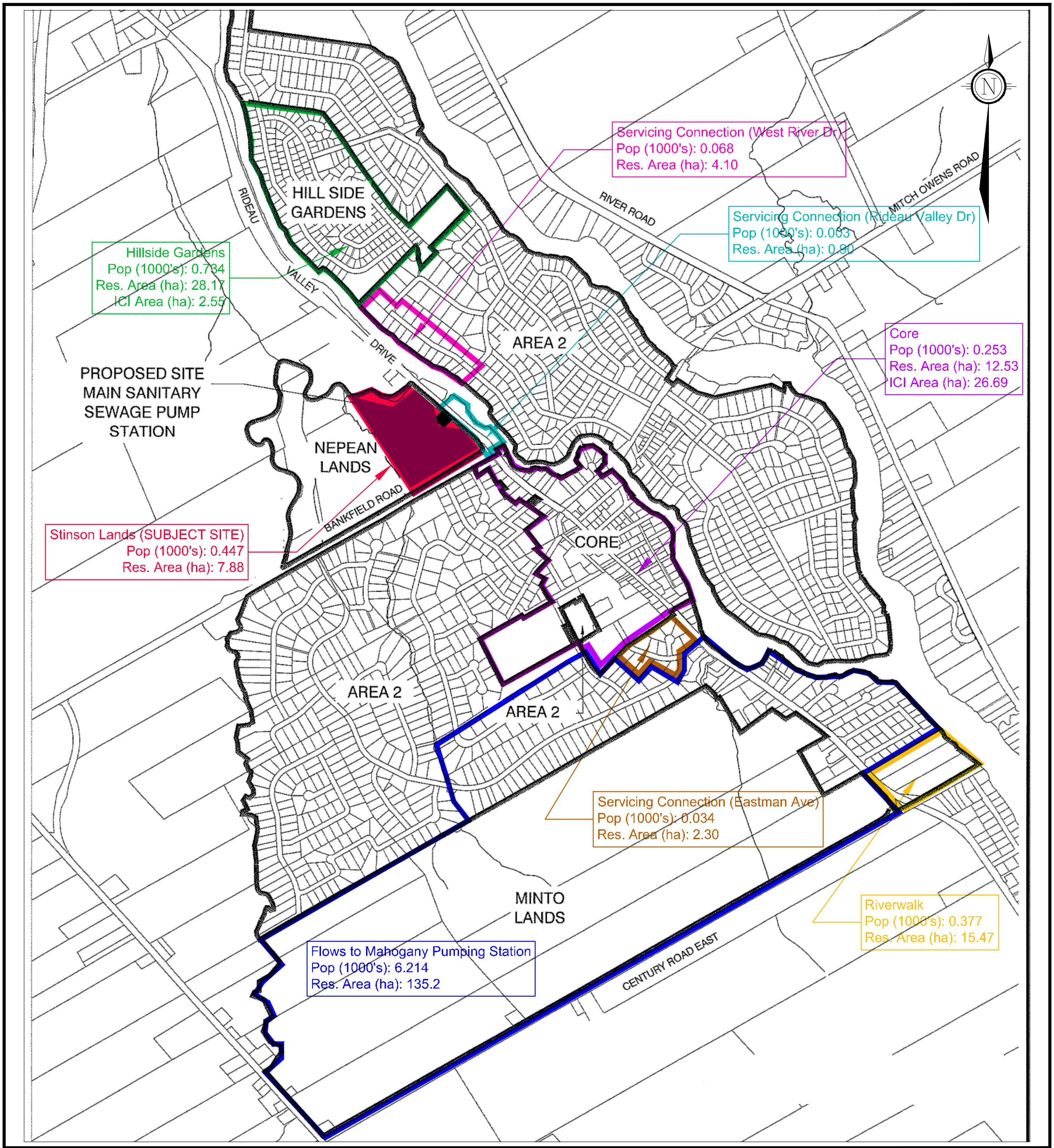
The proposed off-site works will require connecting a 25 m long, 250 mm diameter pipe to an off-site trunk sanitary sewer within the Rideau Valley Drive ROW by constructing a new manhole approximately 37 m upstream of existing MHSA58902. The proposed work will require reinstatement of the existing road to match existing conditions or better and will be completed during Phase 1.

On-site works

The proposed on-site works for Phase 1 will involve the installation of approximately 626 meters, with diameters ranging from 200 mm to 250 mm. The proposed on-site works for Phase 2 will involve the installation of approximately 469 meters of pipe, all with a diameter of 200 mm. On-site sanitary sewers are 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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MANOTICK PS SERVICING AREAS

STINSON LANDS SUBDIVISION

DATE	JOB	FIGURE
MAR 2025	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 7.52 L/s. Refer to **Figure 5.2** – Post-Development Sanitary Drainage Area Plan for reference in **Appendix H**.

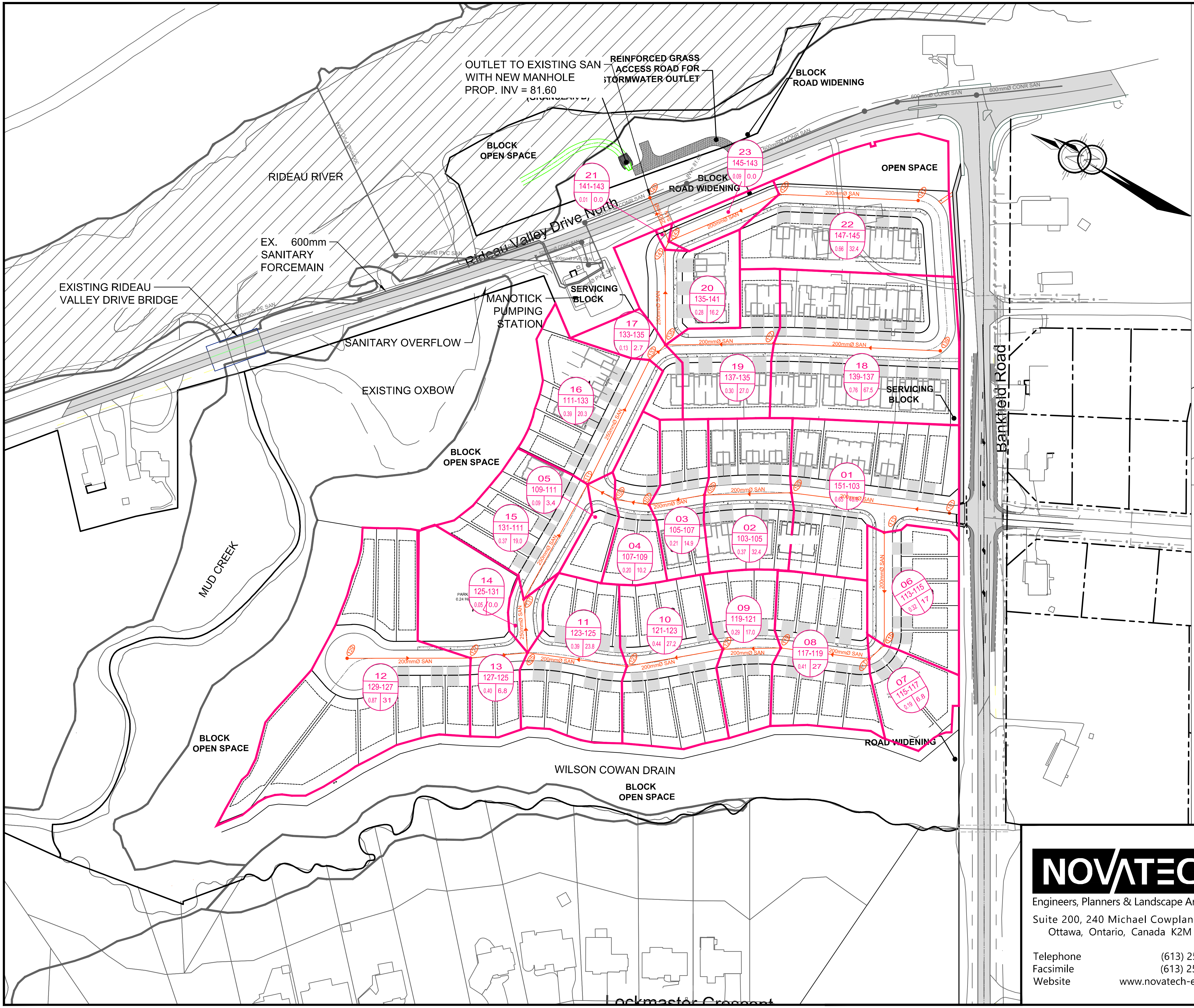
5.5 Hydraulic Grade Line (HGL)

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 83.57m, 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 87.40m. The lowest underside of footing (USF) is conservatively set at 2.35m below the centreline of road which would yield a USF elevation of 85.05m.

As such, the available freeboard between the on-site HGL and the lowest USF is 0.7m. This exceeds 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.



LEGEND

01	AREA ID
101-103	MANHOLE TO MANHOLE
0.65	AREA IN HECTARES
44.0	POPULATION EQUIVALENT
▬	SANITARY DRAINAGE AREA BOUNDARY
—●—	*EXISTING SANITARY SEWER*
—●—	PROPOSED SANITARY SEWER
▨	FLOODPLAIN

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Website www.novatech-eng.com

CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

SANITARY DRAINAGE
AREA PLAN

SCALE	1:2000
DATE	MAR 2025
JOB	121153
FIGURE	5.2

M:\2021\121153\CAD\Design\121153-SAN.dwg, FIG 5.2, Dec 17, 2024 - 10:14am, brundle

SHT11X17.DWG - 279mmX432mm

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 cited concern with a lack of redundancy for the Village of Manotick. To improve the redundancy for the area, Phase 2 of the Manotick Feedermain project will need to be completed. Based on based on correspondence from City Staff the Manotick Feedermain will be completed in 2024.

Refer to **Figures 3.1 and 3.2** – Conceptual General Plan of Services in **Appendix H** 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 400 mm watermain: Connection 1 will be near the sanitary outlet pipe that will be connecting to the existing trunk sewer on Rideau Valley Drive, and Connection 2 will be approximately 140m further south on the same section of street, near the intersection of Rideau Valley Drive and Bankfield Road.

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 for Phase 1 will involve the installation of approximately 813 meters of 200 mm diameter watermain. The proposed on-site works for Phase 2 will involve the installation of approximately 332 meters of 200 mm diameter watermain. Both connections to the off-site works described above will be required for Phase 1. As such, a temporary servicing easement for the watermain within the Phase 2 lands will be required as part of Phase 1.

Proposed hydrant locations have been provided. An additional fire hydrant has been provided along Street Two's dead-end portion in Phase 2 to ensure the required fire flow is available for the furthest lot (lot 4). Hydrant 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 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	3.4 people/unit
Semi-Detached / Townhomes	2.7 people/unit
2-BR Apartments	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 detached / semi-detached / town home units, unless otherwise noted.	10,000 L/min per FUS / OWDG TB-2014
Hydrant spacing and coding	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 detached, semi-detached, and townhome units provided certain site criteria are met.

The criteria are:

- For single detached: a min separation of 10m between the backs of adjacent units.
- Traditional side-by-side semi-detached or 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.

Domestic Demand

The water demand summary for the initial build out (Phase 1) and for the full build out (Phase 1 and 2) of the Subject Site for the average daily and peak hour demands has been provided in **Table 6.2** and **Table 6.3** below, respectively.

Table 6.2: Initial Build Out System Pressure (EPANET)

Condition	Demand (L/s)	Allowable Pressure (psi)	Max/Min Pressure (psi)
Existing Conditions			
AVDY	0.59	80 (Max)	98
PKHR	3.22	40 (Min)	65
Future Conditions			
AVDY	0.59	80 (Max)	86
PKHR	3.22	40 (Min)	68

Table 6.3: Full Build Out System Pressure (EPANET)

Condition	Demand (L/s)	Allowable Pressure (psi)	Max/Min Pressure (psi)
Existing Conditions			
AVDY	1.43	80 (Max)	98
PKHR	7.71	40 (Min)	65
Future Conditions			
AVDY	1.43	80 (Max)	86
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.

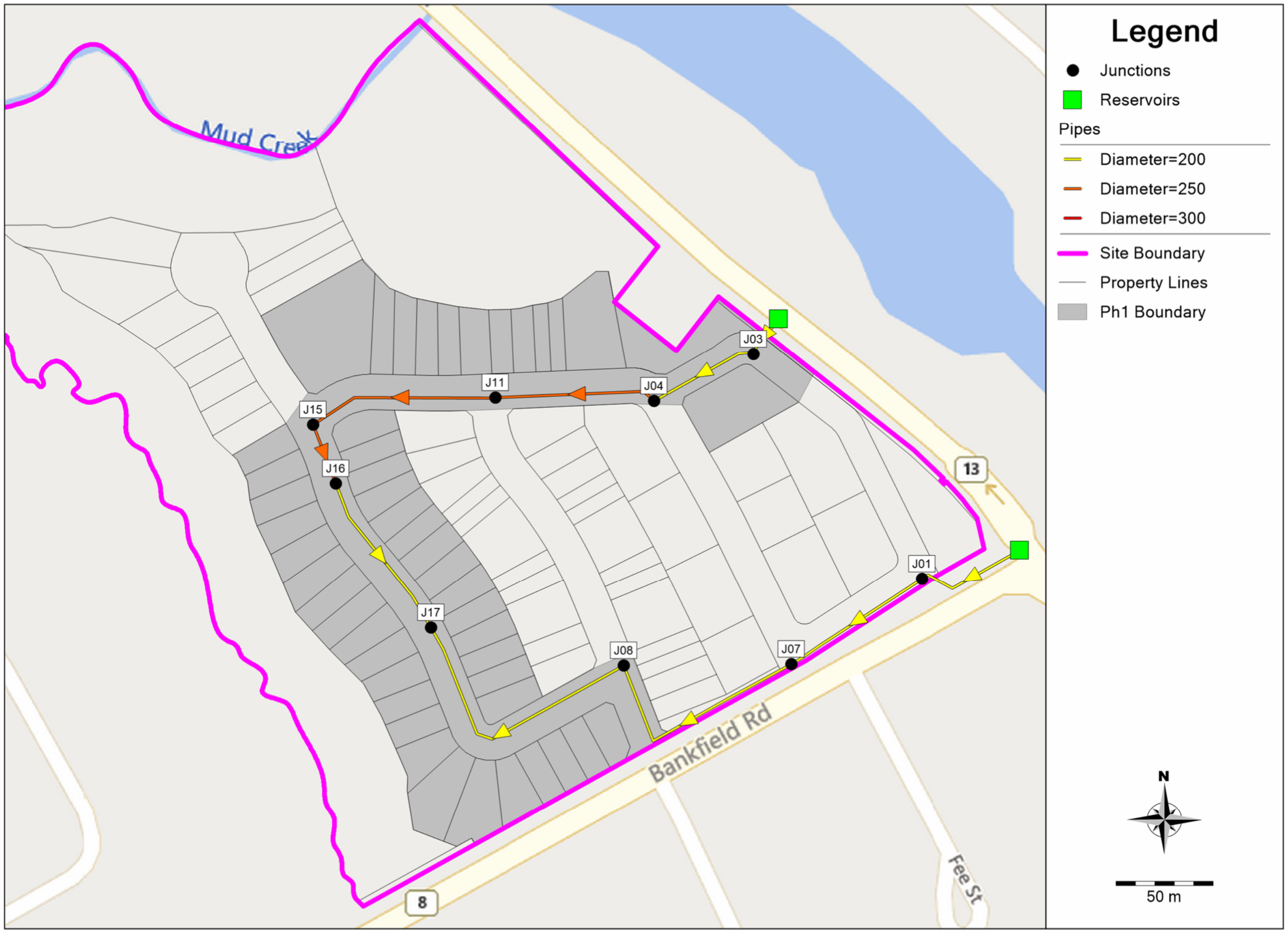
Fire Demand

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.

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.

As mentioned above, four scenarios (and thus, four models) were analysed. For detailed results, refer to the tables provided in **Appendix E** and PCSWMM model schematics provided in **Figure 6.1** - Water Figures_Ph1 and **Figure 6.2** - Water Figures_Ph2 located in **Appendix H**.

Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



Proposed Watermain Sizing, Layout and Junction IDs

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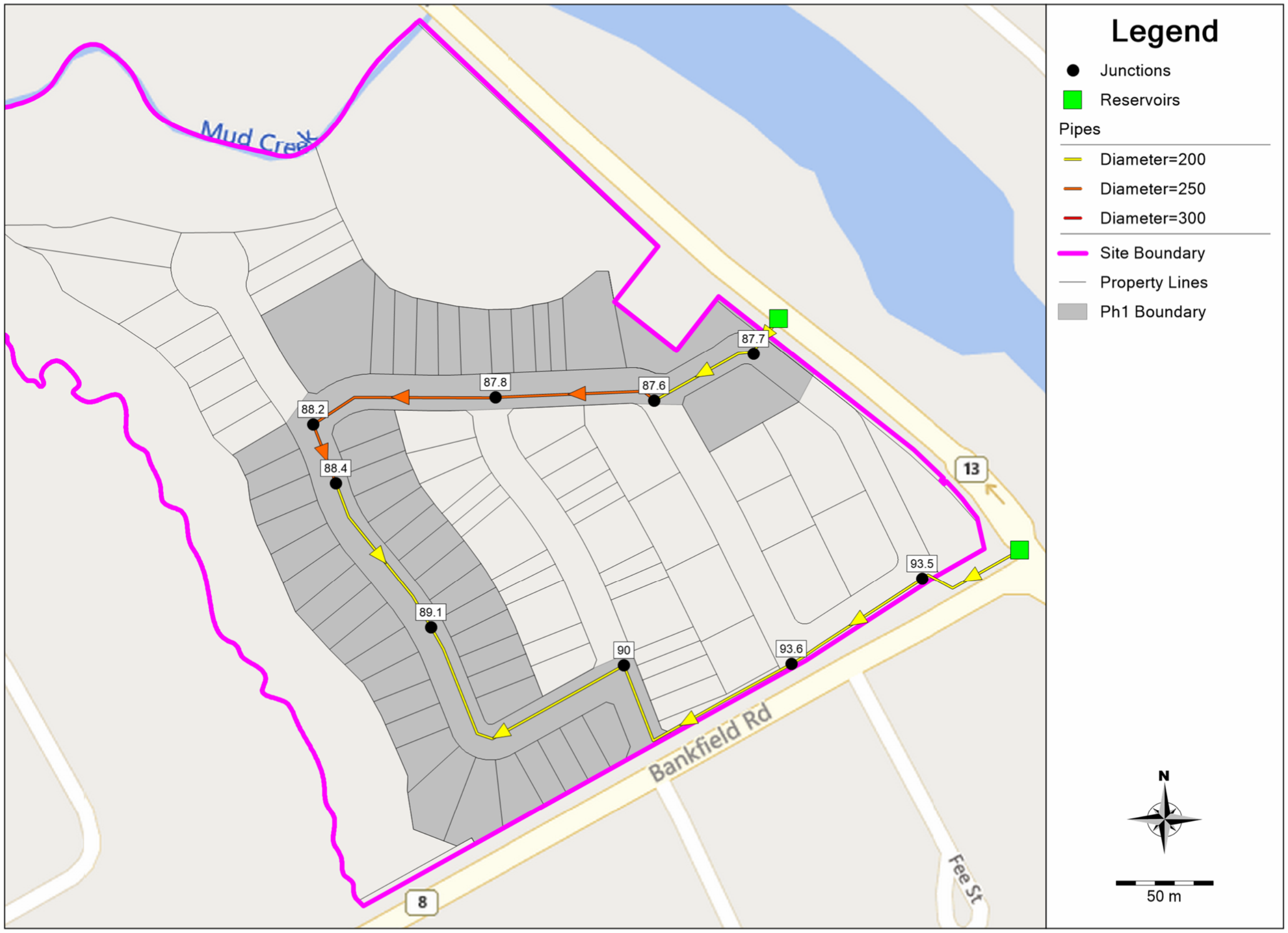
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



Ground Elevations (m)

Date: 2024/04/08
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Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

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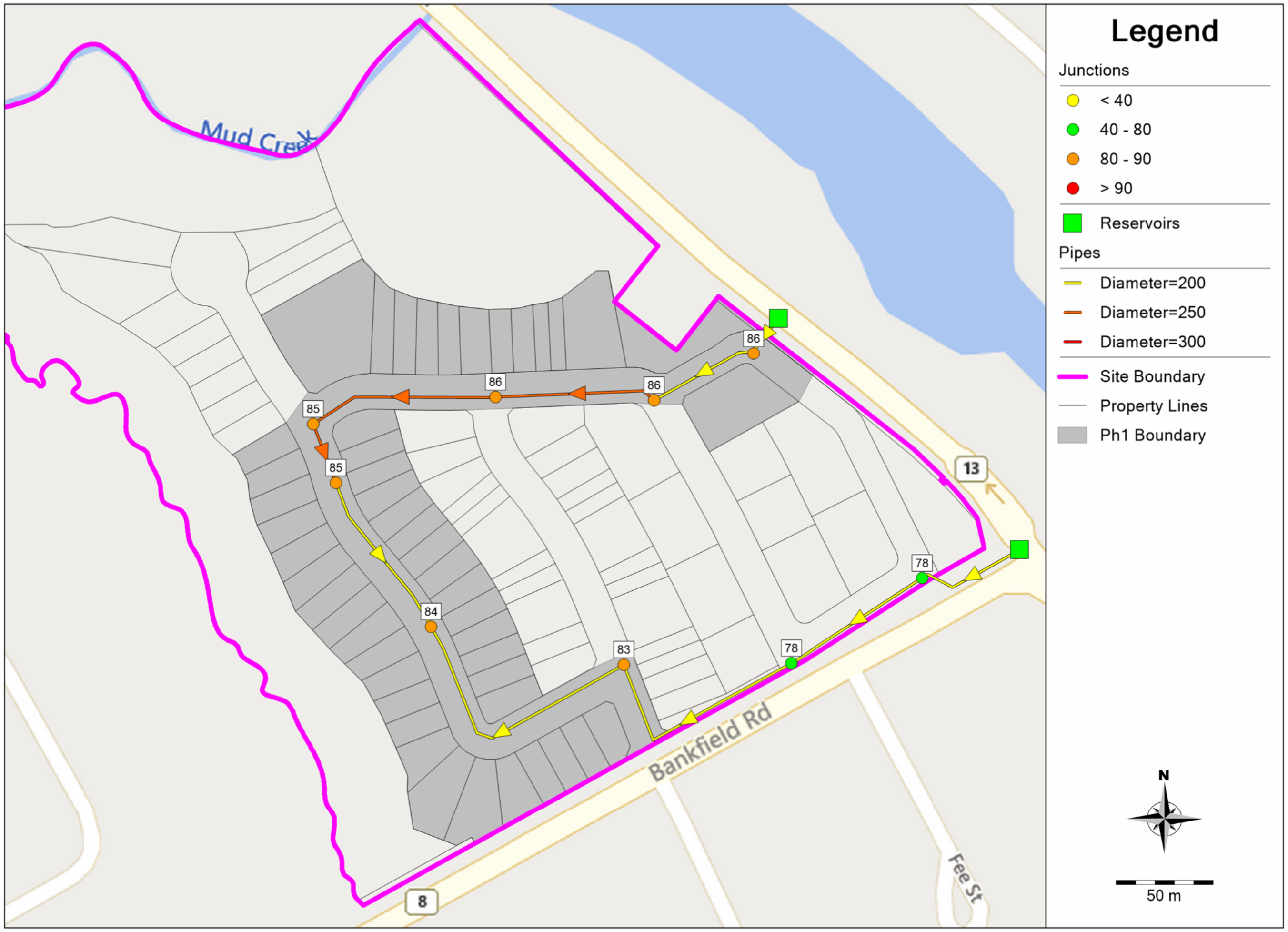

Maximum Pressure During AVDY Conditions – Future

Date: 2024/04/08
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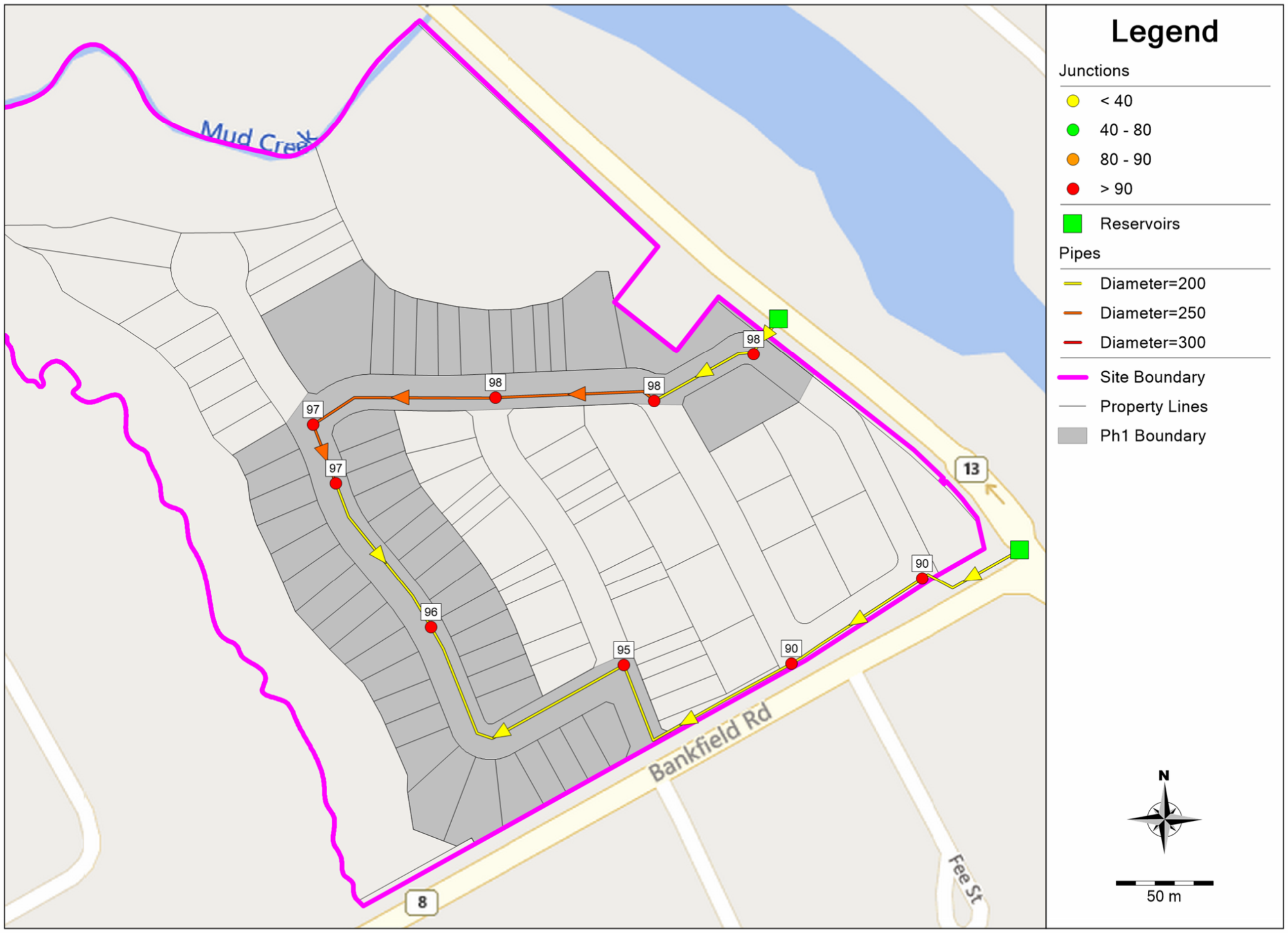
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



Maximum Pressure During AVDY Conditions – Existing

Date: 2024/04/08
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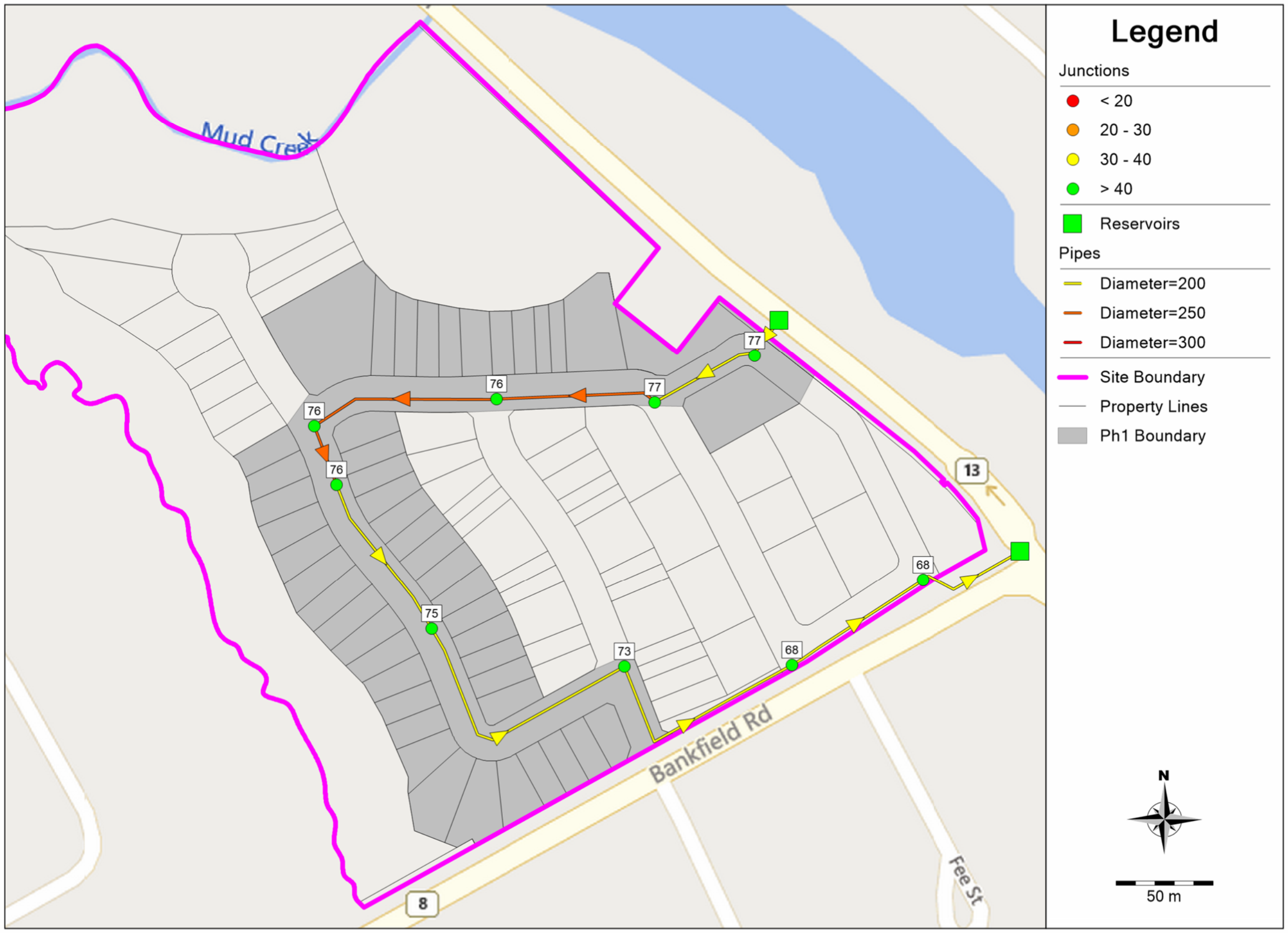
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



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

Date: 2024/04/08
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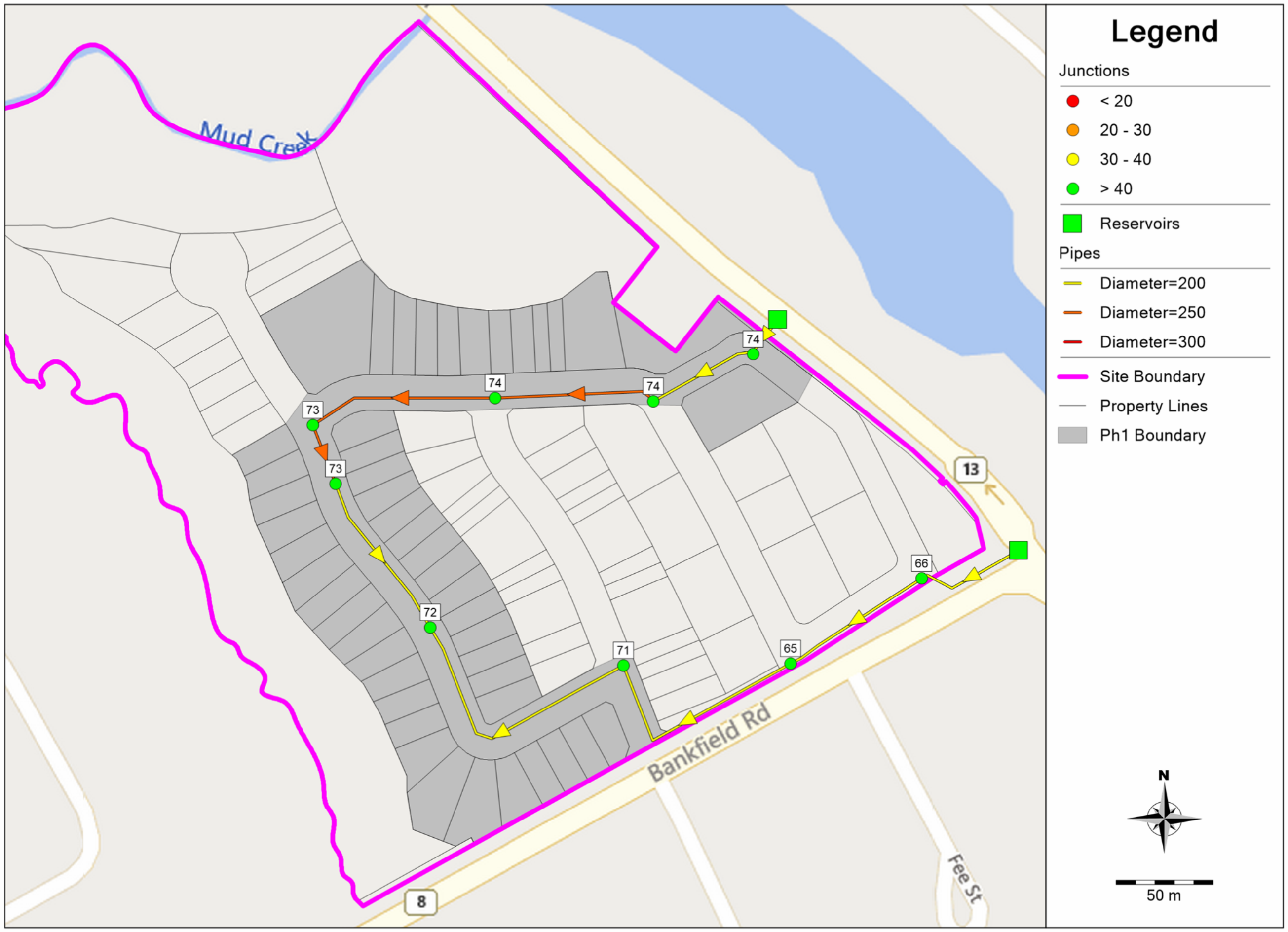
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



Minimum Pressure During PKHR Conditions – Existing

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Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

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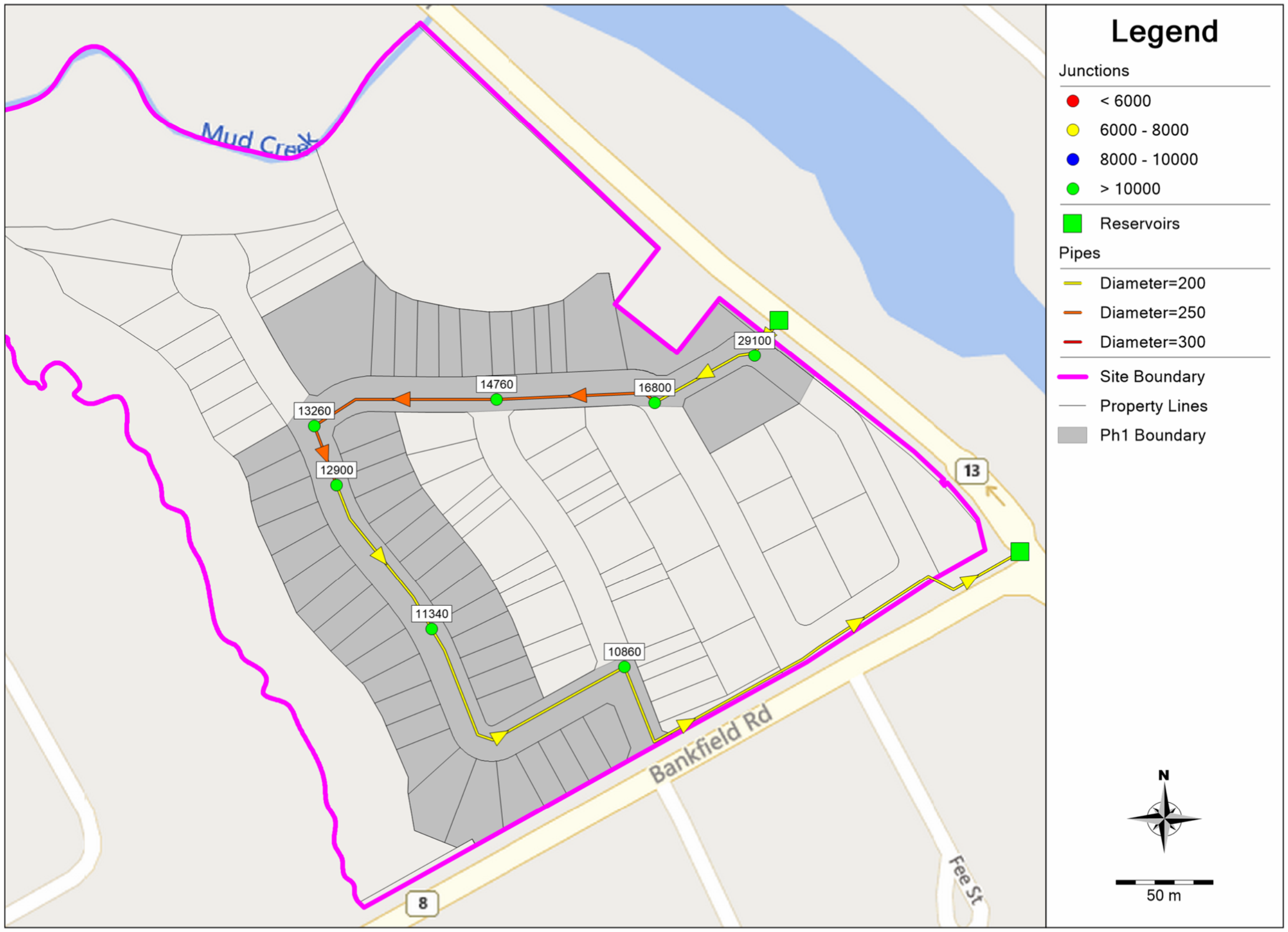

Available Flow at 20psi During MXDY+FF Conditions – Future

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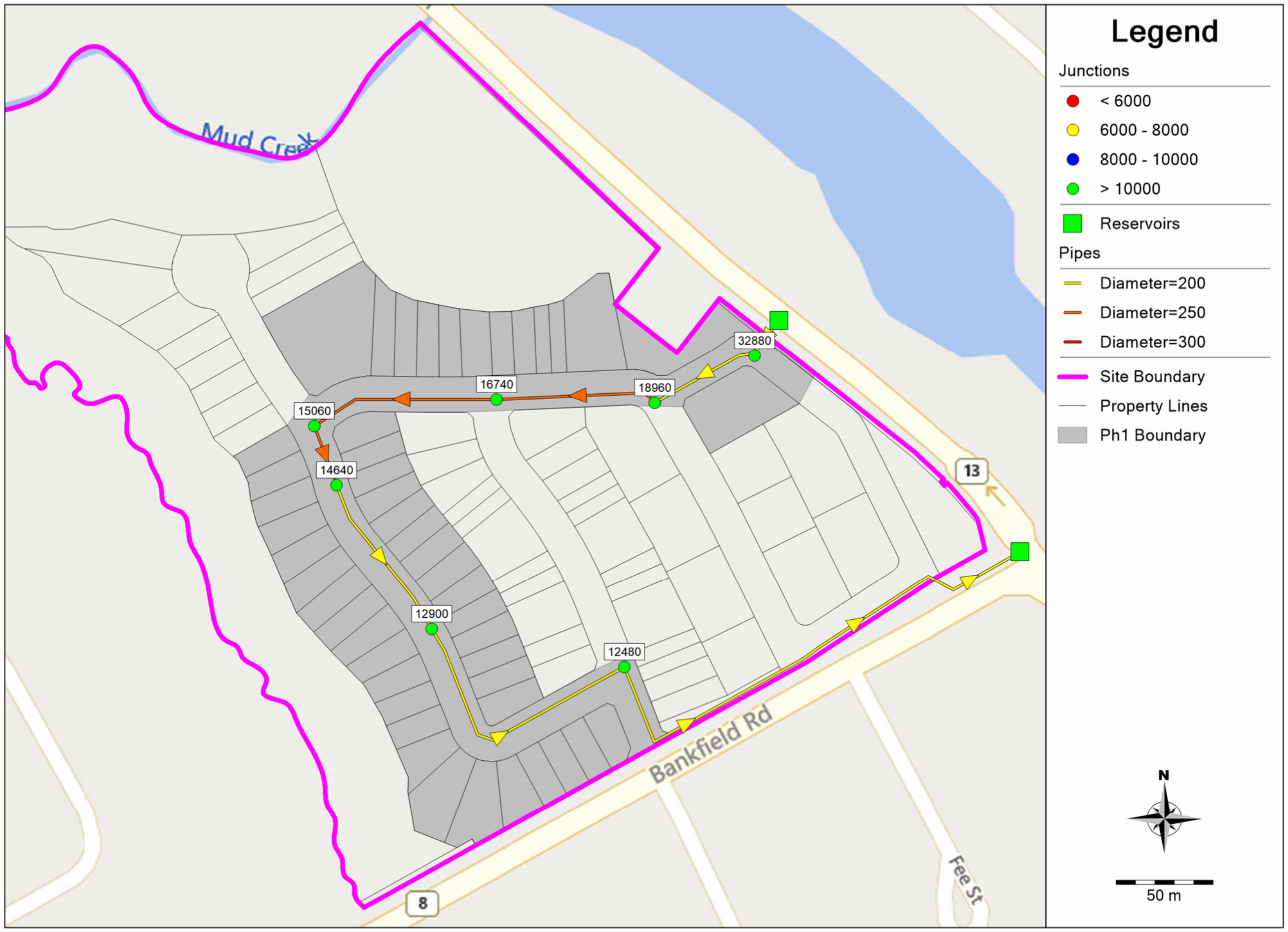
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



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

Date: 2024/04/08
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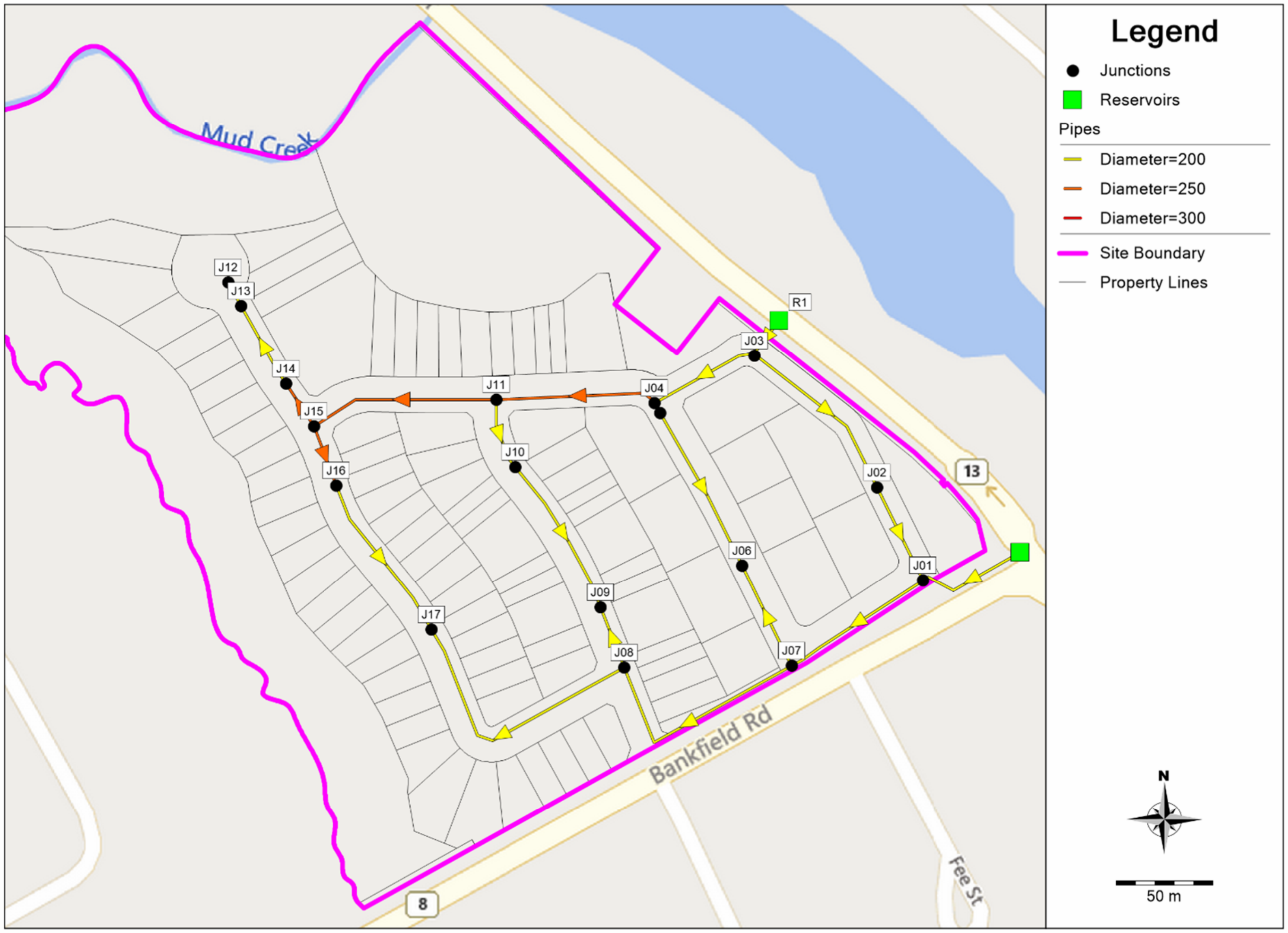
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



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

Date: 2024/12/19
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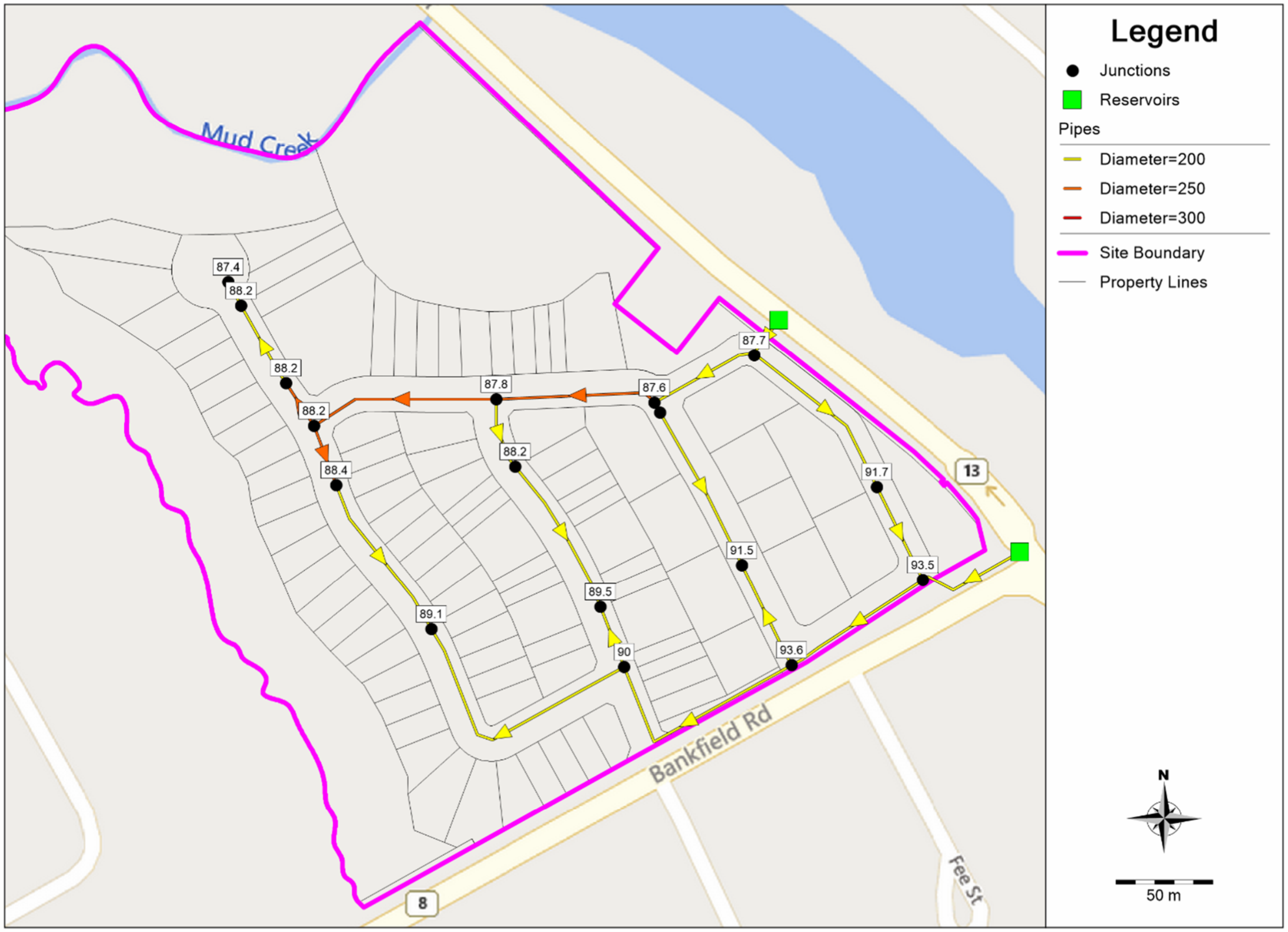
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



Ground Elevations (m)

Date: 2024/12/19
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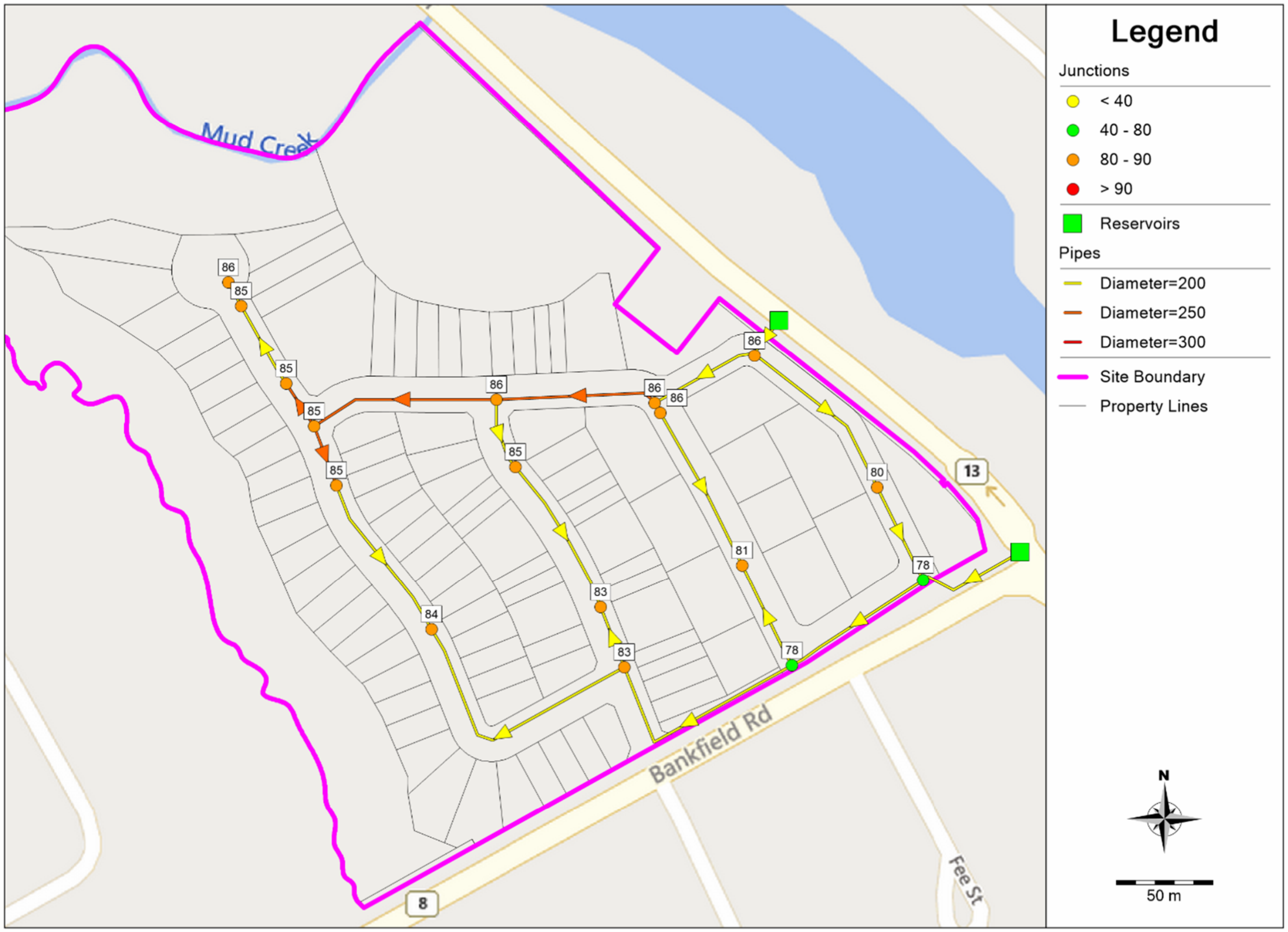
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



Maximum Pressure During AVDY Conditions – Future

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Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic

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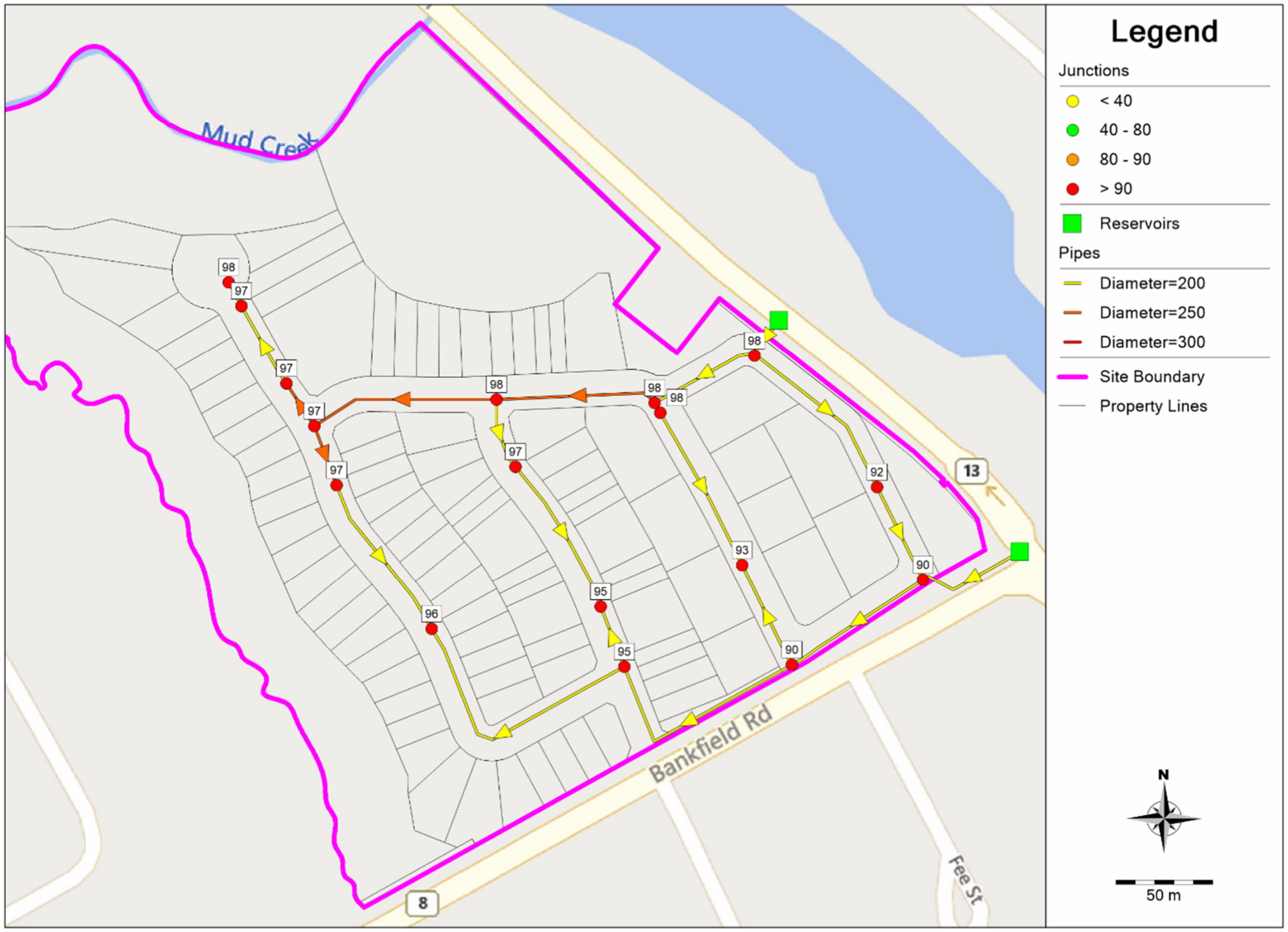

Maximum Pressure During AVDY Conditions – Existing

Date: 2024/12/19
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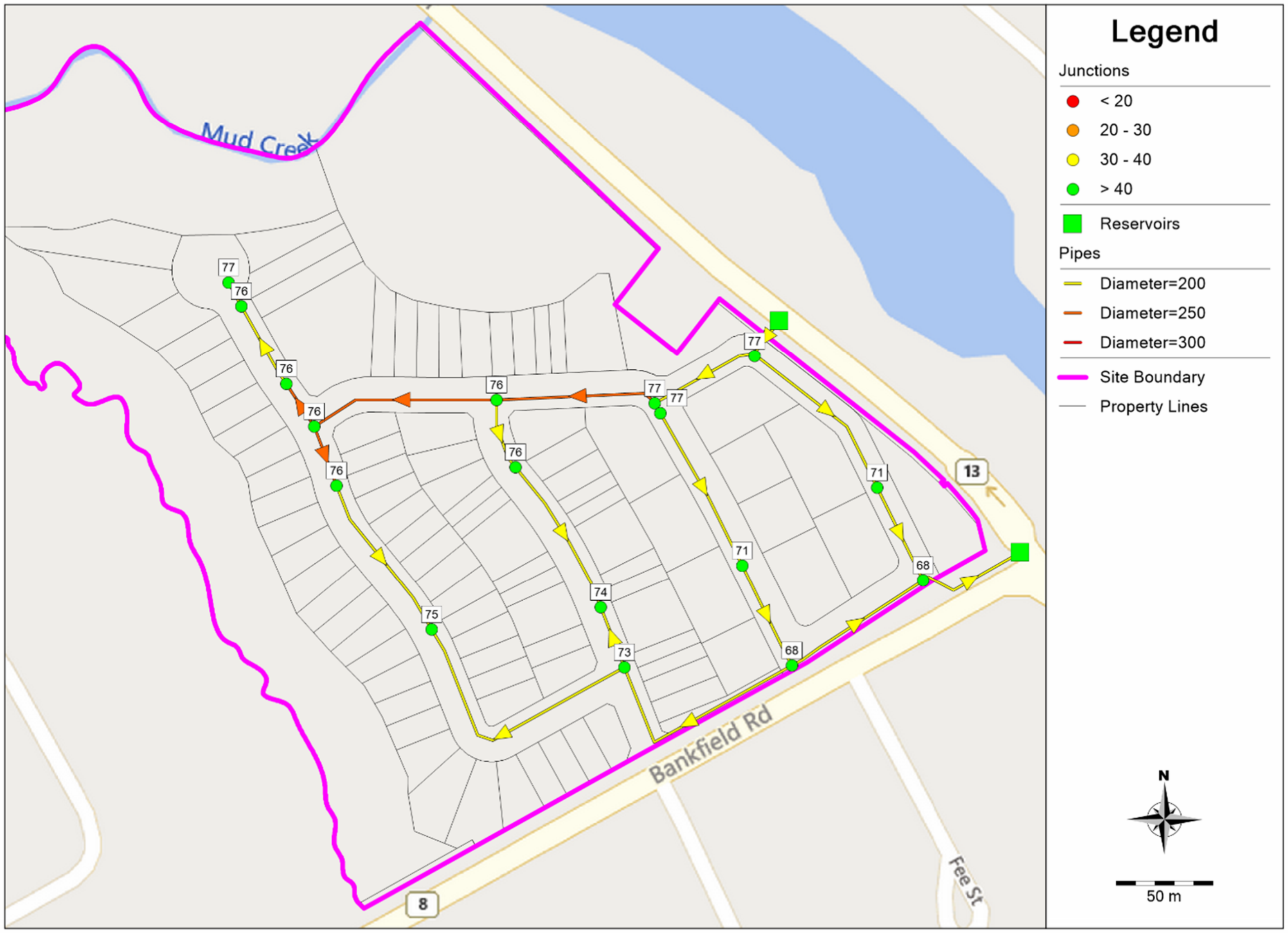
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



Minimum Pressure During PKHR Conditions – Future

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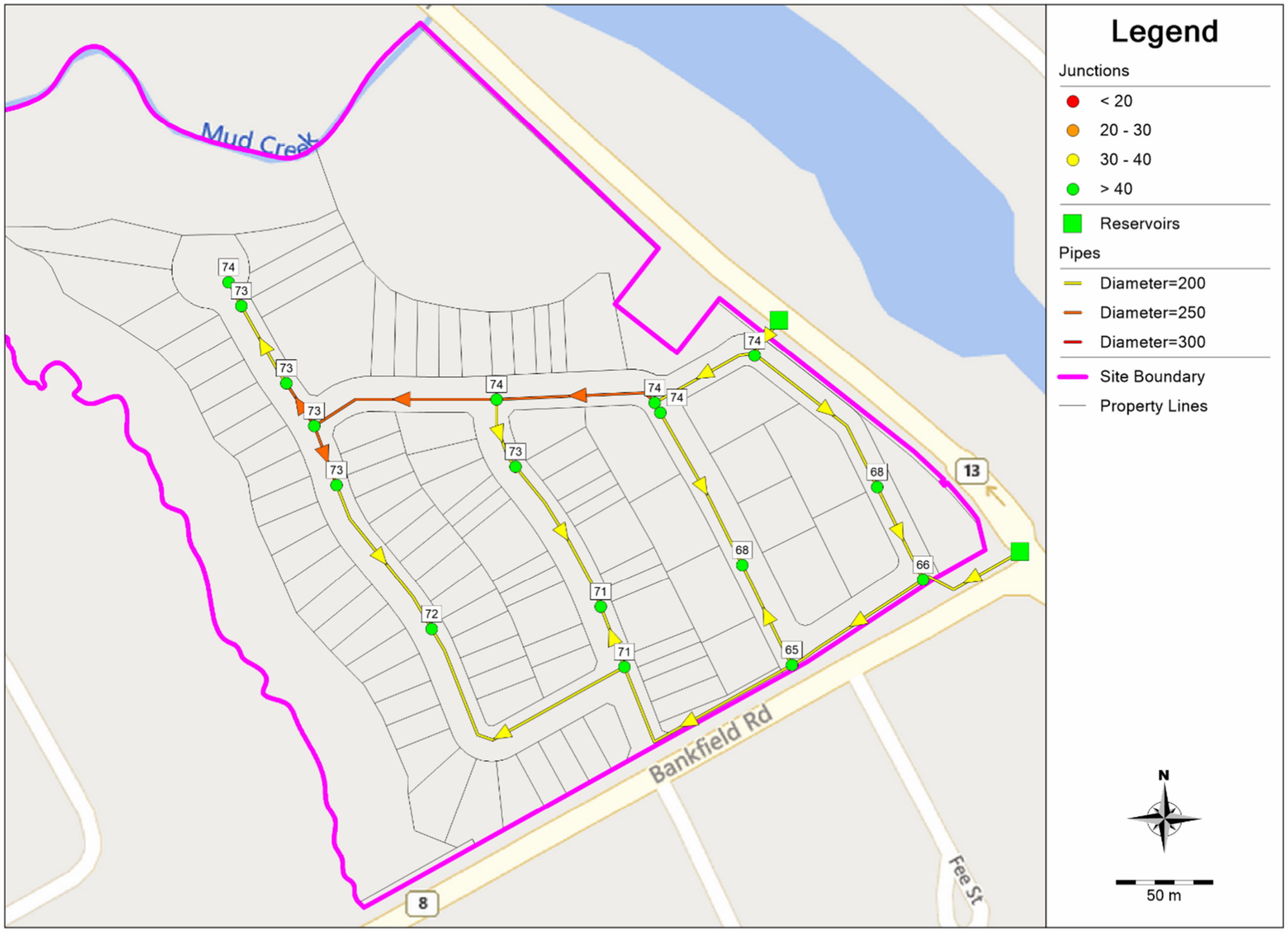
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



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

Date: 2024/12/19
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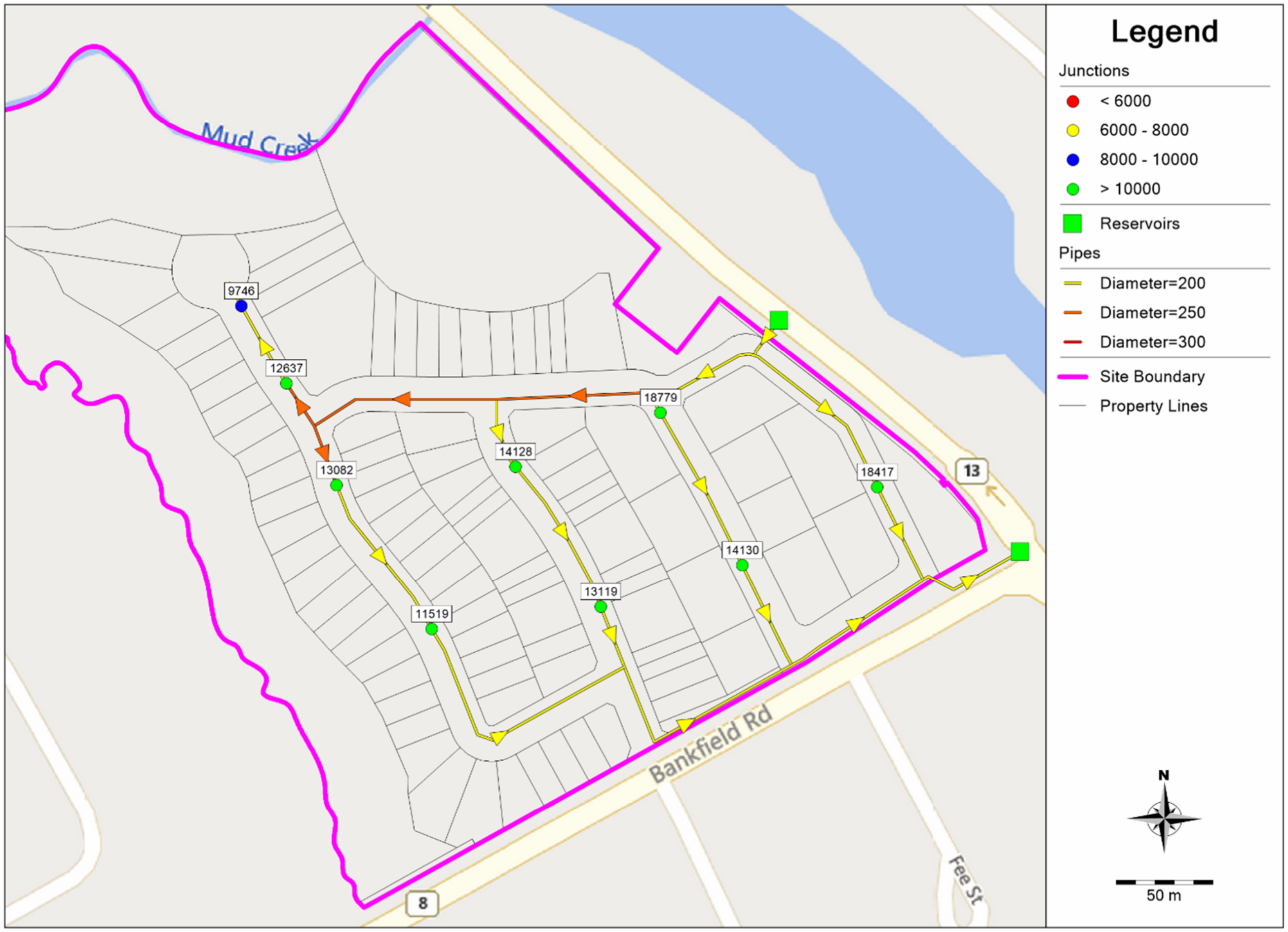
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



<u>Available Flow at 20psi During MXDY+FF Conditions – Future</u>

Date: 2024/12/19
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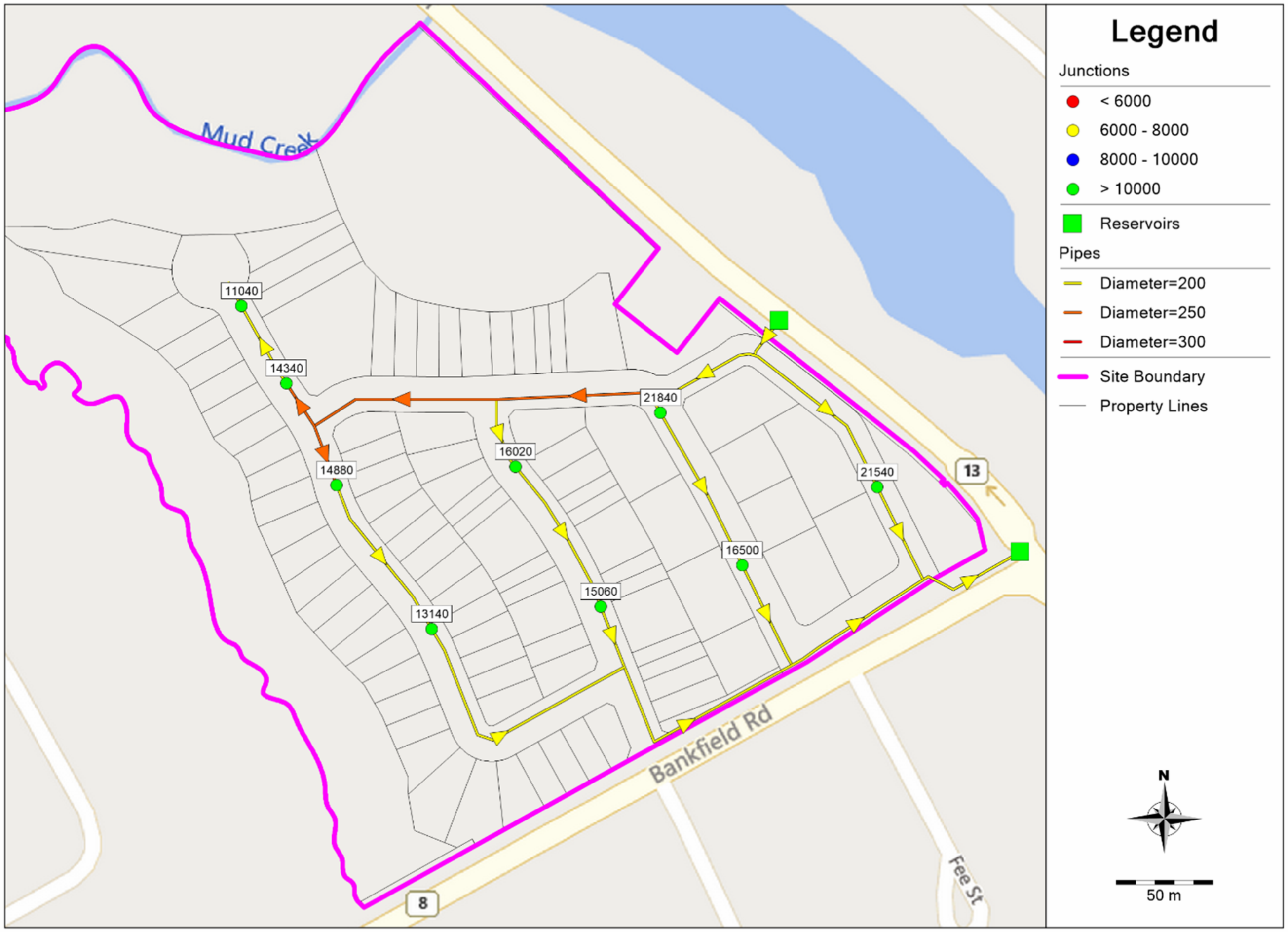
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



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

Date: 2024/12/19
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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 utilities 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; and
- 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 in **Appendix H**.

In addition, due to the dewatering activities required during construction of the proposed infrastructure, a Permit-To-Take-Water (PTTW) application or Environmental Activity and Sector Registry (EASR) 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 / EASR. 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.
- Parks Canada Approval for the Alterations to Shorelines and Watercourses at the Rideau River. Submitted to: Parks Canada. Proponent: Developer.
- MECP Environmental Certificate of Approval (ECA) for the storm / sanitary sewers granted as part of the City of Ottawa’s Transfer of Review or Consolidated Linear Infrastructure programs. Submitted to: City of Ottawa / MECP. Proponent: Developer.
- MECP Pre-authorized Watermain Alteration and Extension granted as part of the 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 and 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 proposed works for Phase 1 will involve the installation of approximately 677 meters of pipe with diameters ranging from 250 mm to 1050 mm and for Phase 2 will involve the installation of approximately 473 meters of pipe, with diameters ranging from 250 mm to 450 mm. The on-site storm sewers will outlet to the Rideau River.
- 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 will outlet to a DICB located in Block 47, and ultimately the same outlet pipe as the minor system, outletting to the Rideau River.

Sanitary and Wastewater Collection System:

- The proposed off-site works will require a new manhole constructed 37 m upstream of existing MHSA58902 of the trunk sanitary sewer within the Rideau Valley Drive ROW 15 m 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 7.52 L/s added by the Subject Site.
- The proposed on-site works for Phase 1 will involve the installation of approximately 626 meters of pipe, with diameters ranging from 200 mm to 250 mm and Phase 2 will involve the installation of approximately 469 meters of pipe with diameter 200 mm 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 400 mm watermain: Connection 1 will be near the sanitary pipe that will be connecting to the existing trunk sewer on Rideau Valley Drive, and Connection 2 will be approximately 140 m further south on the same section of street, near the intersection of Rideau Valley Drive and Bankfield Road.
- The proposed on-site for Phase 1 will involve the installation of approximately 813 meters of 200 mm diameter watermain and for Phase 2 will involve the installation of approximately 332 meters of 200 mm diameter 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 / EASR.
- RVCA Approval and Development, Interference with Wetlands and Alterations to Shorelines and Watercourses” (Ont. Reg. 174/06).
- Parks Canada Approval for the Alterations to Shorelines and Watercourses at the Rideau River.
- MECP ECA for the storm / sanitary sewers.

- MECP Pre-authorized Watermain Alteration and Extension.
- Tree Cutting Permit.
- City of Ottawa Commence Work Notice.
- Road Closure and 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:

Kallie Auld.

Brendan Rundle, B.Eng.
EIT I Land Development

Kallie Auld, P.Eng.
Project Manager I Water Resources

Reviewed by:

LICENSED PROFESSIONAL ENGINEER
B. C. SWEET
100206031
MARCH 11, 2025
PROVINCE OF ONTARIO

Ben Sweet, P.Eng.
Project Manager I Land Development

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

Appendix A
Correspondence

NOVATECH
Engineers, Planners & Landscape Architects

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***	***Novatech***
• 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.	

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***
• *JO scheduled a meeting on June 29th to continue the Fluvial Geomorphological submission requirements*	
SB asked if there are any other items to discuss:	
<u>ROW Widths</u>	
• 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

MEMORANDUM

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.

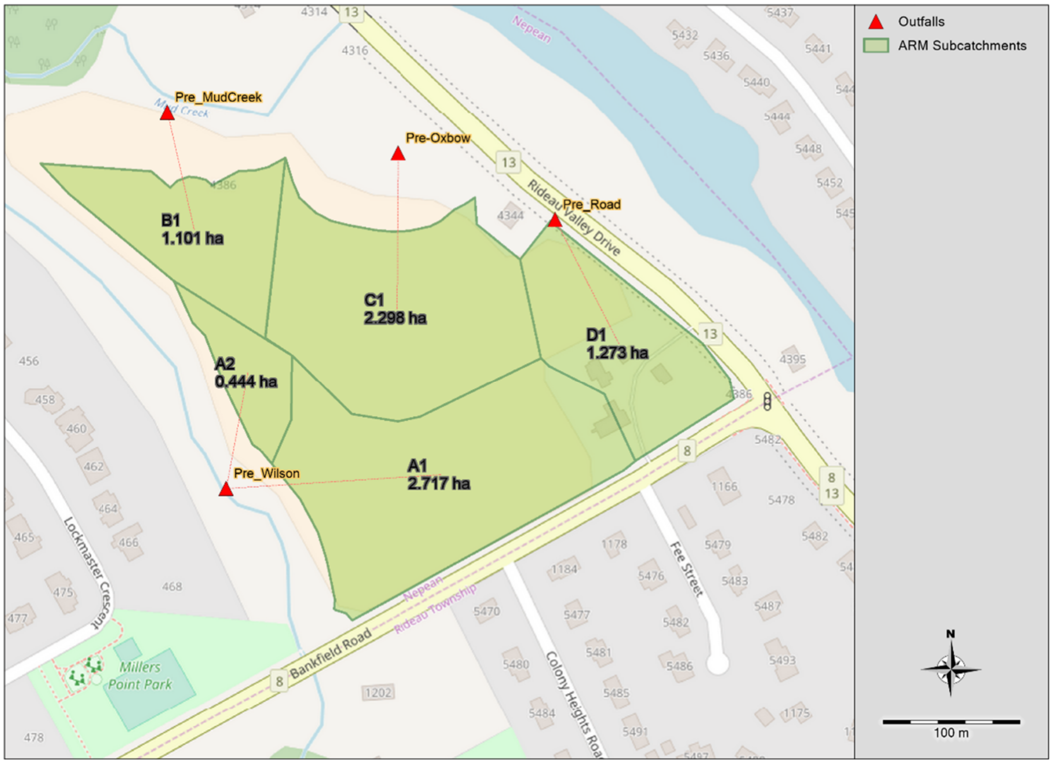

Figure 1: PCSWMM Model Schematic – Existing conditions

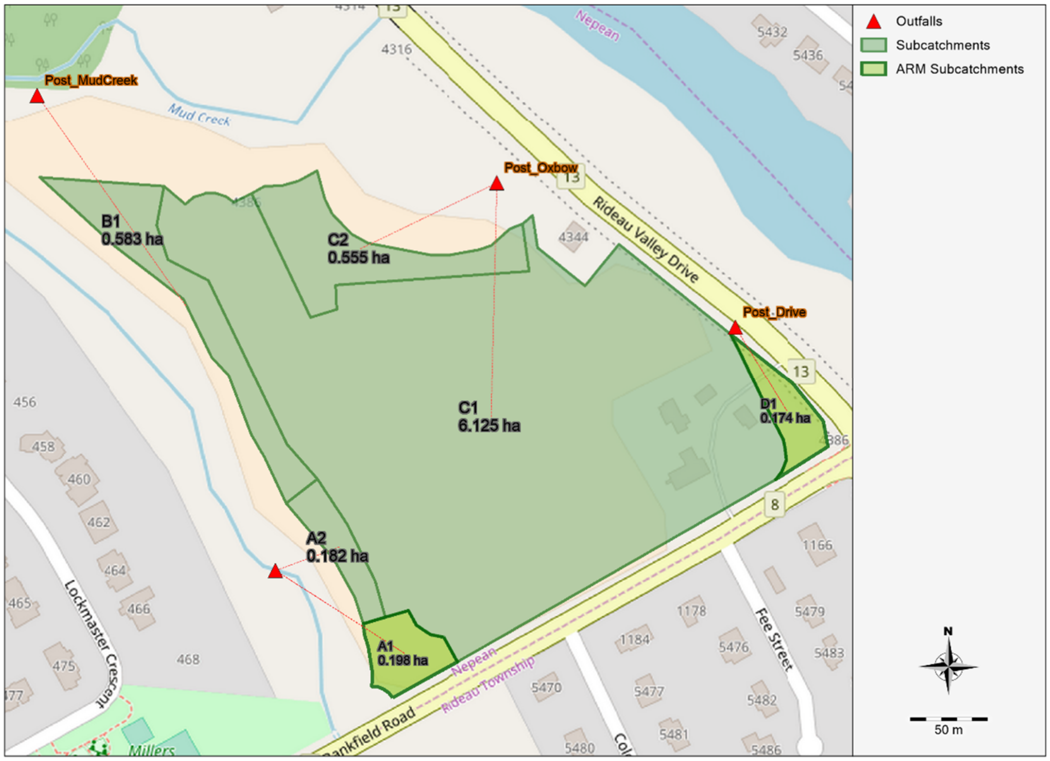

Figure 2: PCSWMM Model Schematic - Proposed Conditions

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.

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.

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

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

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

Quality Control

- 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.

Fluvial Geomorphological Study Requirements

- 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

Other Items

- Impact Assessment of adjacent Municipal Depot (4244 Rideau Valley Drive):
  - 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.
  - 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.
  - 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:
  - 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

NOVATECH
Engineers, Planners & Landscape Architects

M E M O R A N D U M

DATE: JUNE 30, 2023

TO: JOSEPH ZEGORSKI; JOHN BOUGADIS, ERICA OGDEN-FEDAK, DAMIEN WHITTAKER, BRIAN MORGAN, MATTHEW HALEY

FROM: SAM BAHIA, BRENDAN RUNDLE

RE: 4386 RIDEAU VALLEY DRIVE – STINSON LANDS – STORMWATER & SANITARY OUTFLOWS TO EXISTING OXBOW

CC: RYAN MACDOUGALL, GREG WINTERS

Background & Purpose

As requested, Novatech has reviewed the previous design by IBI Group for the Manotick Pump Station Sanitary Overflow (PS Overflow) and its outlet to the existing Oxbow within the property of 4386 Rideau Valley Drive (Subdivision). We offer a preliminarily refined design that incorporates and addresses some key items raised by the City:

- PS wastewater overflow and containment strategy,
- accommodating a storm outlet for the Stinson Lands' proposed subdivision,
- addressing erosion mitigation,
- reducing and mitigating negative impact to the Oxbow's ecological function,
- landownership of the Oxbow.

Manotick PS Design (2008 IBI)

During the 2008 PS design, Parks Canada had required the PS Overflow to have a containment area prior to discharge into the Rideau River, to reduce downstream impact. Highlights of the IBI design are below:

- The Manotick PS's 1200mm diameter overflow invert at the PS's wet well is ~83.60m (which is the governing elements of the HGL analysis), prior to being directed into the Overflow chamber/MH. This overflow operates during catastrophic events only.
- Overflow wastewater is directed through a 525mm diameter pipe towards the Oxbow, from the PS overflow chamber/MH along Rideau Valley Drive N (SB lanes). The pipe is currently stubbed outside of the PS limits.
- A headwall (allowing for stoplogs) was proposed along the Oxbow, just upstream of its confluence with Mud Creek. The bottom elevation of the weir was set below the Oxbow's permanent pool. The pool would be controlled by an existing highpoint (similar to a broad crested weir) just upstream of the Mud Creek confluence. This highpoint has the potential to erode over time, which was not the mandate of the IBI design to address.
- A berm of elevation of 83.80m was proposed around the Oxbow NWL elevation to contain spill volumes prior to discharge to Mud Creek/Rideau River. The approximate volume within the bermed area, assuming stoplogs were installed up to elevation 83.80m was ~4900m³ (5 hours of storage at peak flow of 270L/s), excluding any upstream structure/pipe volume storage. Notwithstanding, after discussions with J Moffat of IBI

Group via email and telephone conversation, he could not recollect if this was a design factor, nor a required target volume for the spill containment.

- Informal access via an existing driveway was proposed for any clean up or maintenance required to the of the sanitary overflow headwall and Oxbow Headwall.

Due to landownership issues with the Oxbow, not being owned by the City, the overflow and containment berm were never constructed per the 2008 design.

Floodplain Elevations for Mud Creek (2019 RVCA)

Updated floodplain mapping for consideration is summarized below:

- 2-year event = 82.22m
- 5-year event = 82.23m
- 10-year event = 82.25m
- 20-year event = 82.27m
- 100-year event = 82.61m

Proposed Stormwater and Sanitary Containment (2023 Novatech)

As a result of the proposed subdivision requiring an outlet to the Oxbow; therefore a coordinated solution is outlined below to accommodate both the PS Overflow containment and the Subdivision's storm outlet at the Oxbow:

- Construct the previously proposed sanitary overflow from its current stub (TBC) to the Oxbow at invert ~82.00m. A plunge pool at the PS Overflow headwall (that can accommodate stoplogs) should be considered to allow for primary containment and storage within the upstream pipes/structures prior to discharge into the naturalized area of the Oxbow. A containment berm is required. Maintaining informal access via an existing driveway to operate and place stoplogs at headwalls for containment during a spill. Consultation with Wastewater Operations would be necessary (**PS Works,** by the City).
- Construct a stormwater outlet with an invert elevation of 82.90m from the proposed Stinson Lands subdivision, with its own plunge pool and open channel to connect it to the Oxbow (**Subdivision Works,** by the proponent).
- Like the 2008 IBI design, a refined Oxbow Headwall with a rectangular weir that allows for the installation of stoplogs during catastrophic events should be constructed within the Oxbow. The headwall should be located at an area that reduces the impact to existing trees and with close access to Rideau Valley Drive. The 2008 IBI design is to be modified by establishing a weir bottom elevation that mimics the Oxbow's current normal water level of 81.35m to maintain its ecological function/habitat and would mitigate against erosion potential of the Oxbow outlet channel. The top of the weir wall/stoplogs is to be set at 82.60m to allow for secondary containment and storage of ~7700m^3 which is 50% greater than the previously available storage (**Shared Works**).
- The Oxbow ownership can be conveyed by the Proponent to the City at registration.

Other design coordination and criteria that should be considered:

- Further consultation is required with the City, environmental/ecological consultant, MECP and the geomorphology consultant to determine if the proposed works are acceptable. If the works are acceptable and subject to any mitigation measures, this can be discussed in due course.

- A berm is to be constructed at elevation 82.60m to maximize the containment. This may require a minor RVCA fill permit although there is minimal floodplain volume loss.
- The pump station overflow of 83.60m is greater than the 100-year floodplain elevation of Mud Creek (OSDG requires the overflow to be > 25-year HGL of the receiver). The 2008 IBI HGL analysis is still applicable.
- Oxbow Headwall weir width is to be 2.2m (2.4m long dimensional lumber, less 100mm for a recess on both sides), that accommodates the Stinson Land's post-development flows from the Oxbow for all the various design events/criteria. Based on a quick review, and subject to modelling for the subdivision minor system/Oxbow, the 100-year +20% HGL, and 100-year floodplain are ~82.20m and 82.61m, the forgoing boundary conditions are well below the stormwater outlet invert (82.90m) and the lowest USF (85.50m) within the subdivision.
- Additional erosion mitigation measures may be required at the Oxbow/Mud Creek confluence.

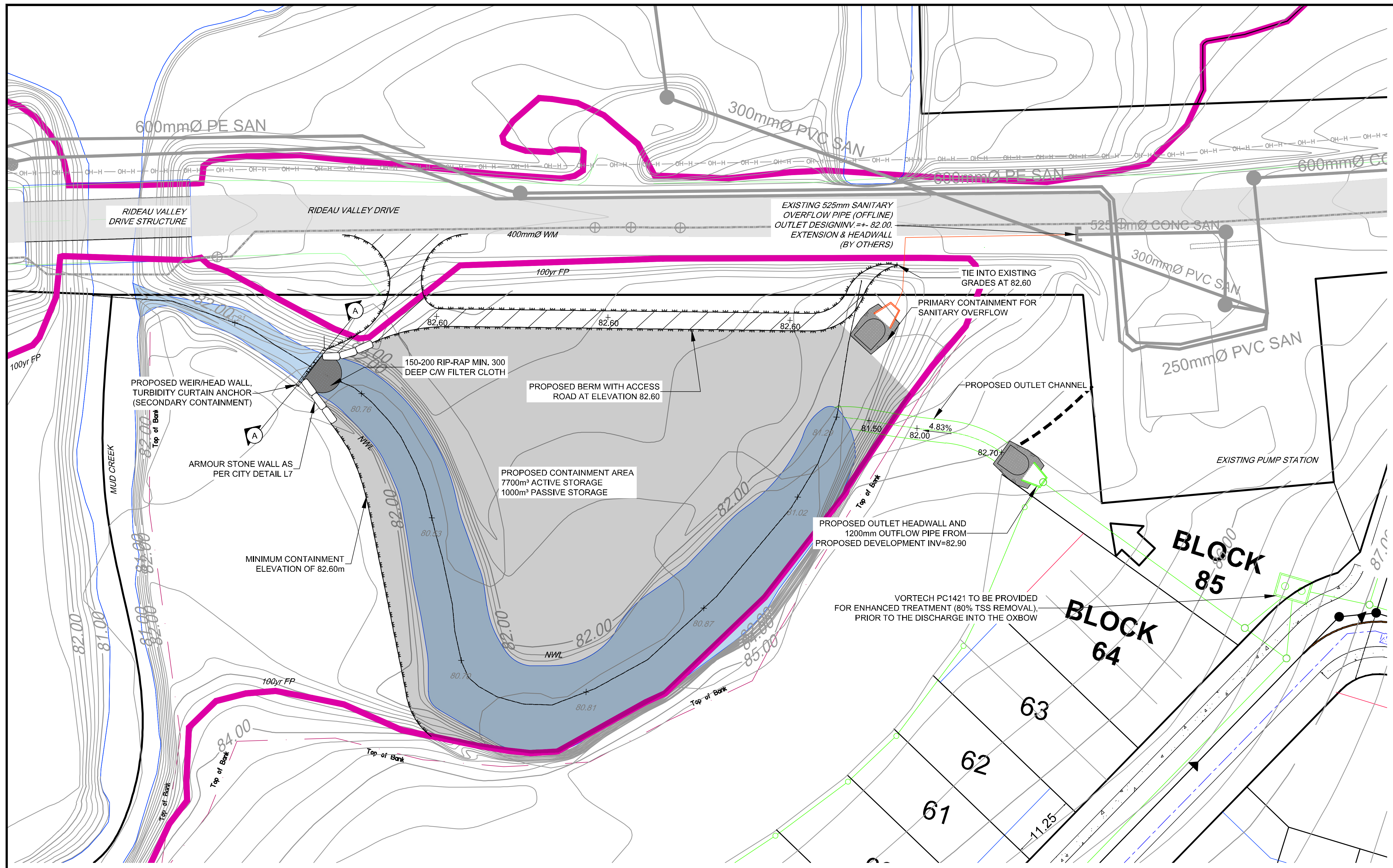
Next Steps and Conclusion

In our opinion, the proposed stormwater and sanitary PS works within the Oxbow would be a win-win for both the City and the Proponent. Subject to further discussions regarding the mitigation, we envision the following next steps to advance this:

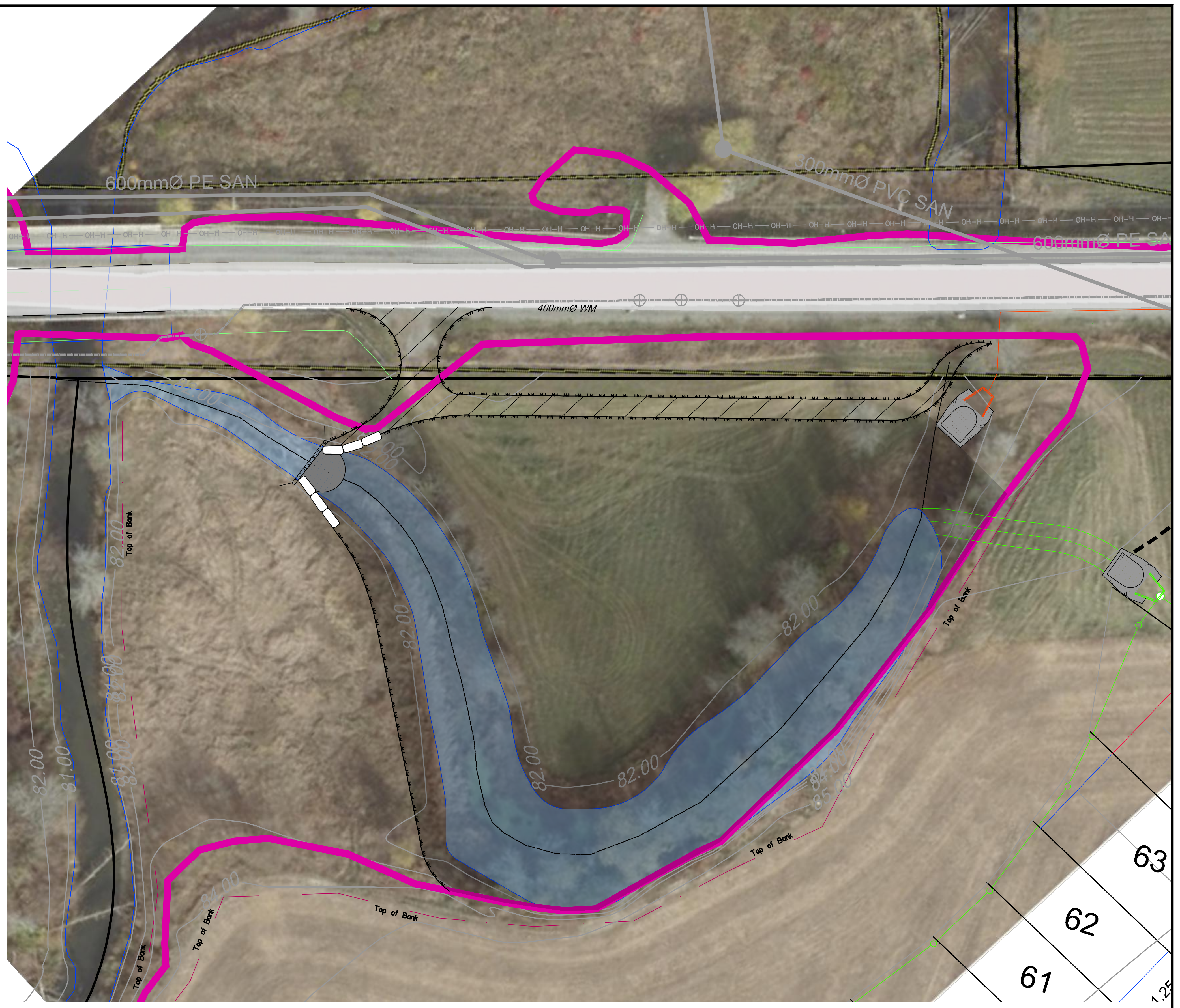
- Agreement in principle of the above approach (after buy-in from MECP and Operations)
- Draft Plan Approval, so we can begin detailed design on behalf of the Proponent.
- Coordinate the detailed design of the Oxbow Headwall between the City and Proponent's Engineers
- Design approvals and permits
- Costs and landownership:
    - PS Overflow Works by the City
    - Subdivision Works by the Proponent
    - Shared Works to be shared, subject to a cost recovery clause/term within the Subdivision Agreement.
    - Oxbow lands can be included within the DP and M-Plan as a block, so it can be dedicated to the City to operate the PS Overflow, Subdivision storm outlet, and Oxbow Headwall.
- Timing:
    - The PS Overflow and Subdivision Works can be completed independently.
    - The Shared Works should be coordinated by both parties in advance but can be installed by either party at any time.
    - Notwithstanding, there may an opportunity to coordinate other works by both the City and the Proponent within Rideau Valley Drive, to reduce construction traffic impacts/closures (extension of the overflow, subdivision sanitary/watermain connections).

Please feel free to call and arrange a second meeting to discuss further.

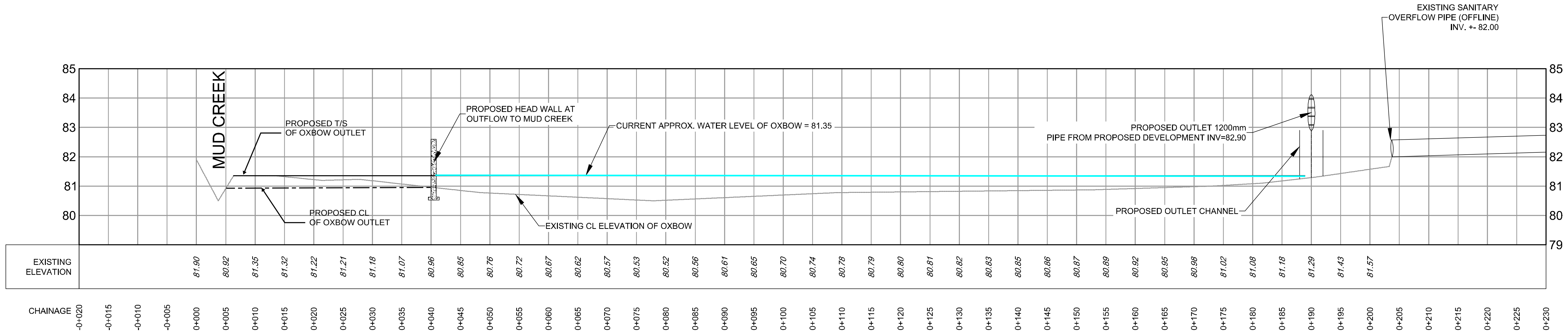
Attach (121153-Oxbow Preliminary Design)



OXBOW - PLAN VIEW
PLAN VIEW - SCALE: 1:500

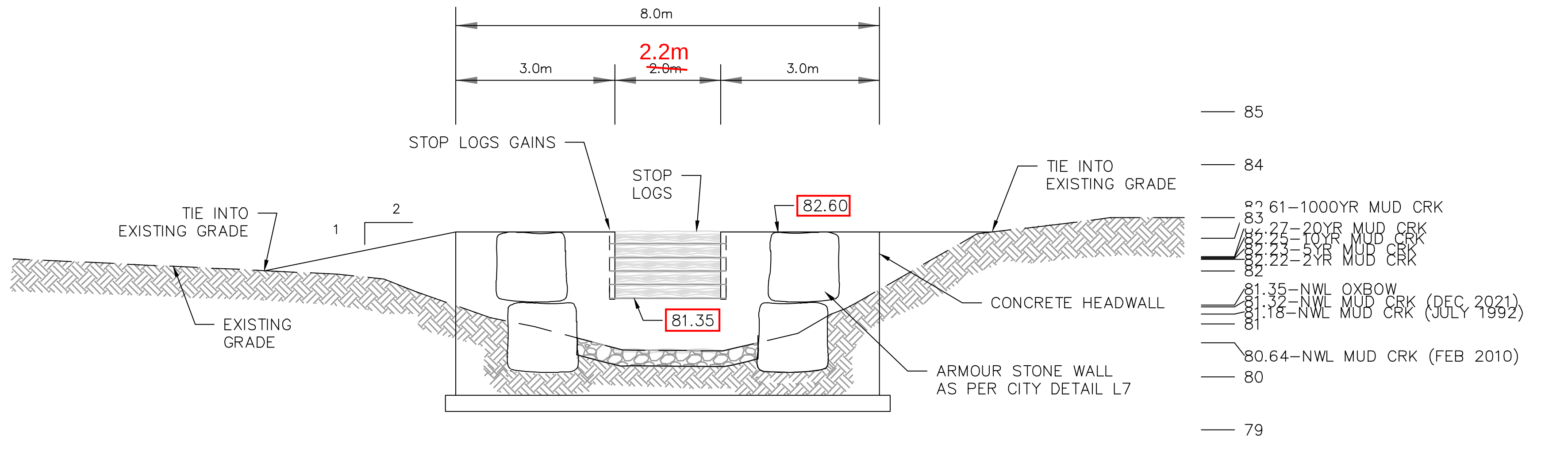


OXBOW - AERIAL VIEW
PLAN VIEW - SCALE: 1:500



CHAINAGE	EXISTING ELEVATION
-0+020	
-0+015	
-0+010	
-0+005	
0+000	81.90
0+005	80.92
0+010	81.35
0+015	81.32
0+020	81.22
0+025	81.21
0+030	81.18
0+035	81.07
0+040	80.96
0+045	80.85
0+050	80.76
0+055	80.72
0+060	80.67
0+065	80.62
0+070	80.57
0+075	80.53
0+080	80.52
0+085	80.56
0+090	80.61
0+095	80.65
0+100	80.70
0+105	80.74
0+110	80.78
0+115	80.79
0+120	80.80
0+125	80.81
0+130	80.82
0+135	80.83
0+140	80.85
0+145	80.86
0+150	80.87
0+155	80.89
0+160	80.92
0+165	80.95
0+170	80.98
0+175	81.02
0+180	81.08
0+185	81.18
0+190	81.29
0+195	81.43
0+200	81.57
0+205	
0+210	
0+215	
0+220	
0+225	
0+230	

OXBOW
PROFILE VIEW - SCALE: 1:500 (H) & 1:25 (V)



SECTION A-A - HEAD WALL DETAIL
CROSS-SECTION - SCALE: 1:100

M:\2021\121153\CAD\Design\Figures\121153-Oxbow.dwg, Oxbow Fig, Jun 30, 2023 - 1:12pm, brundle

NOTE:
1. OXBOW IS TO GENERALLY REMAIN UNTOUCHED TO MAINTAINT ITS NATURAL FEATURES AND ECOLOGICAL FUNCTION.
2. OWNER AND CONTRACTOR ARE OBTAIN ALL NECESSARY DFO, PARKS CANADA, MECP, RVCA, AND CITY PERMITS.
3. ANY IN-WATER WORKS ARE TO BE COMPLETED BETWEEN JULY 1 AND JANUARY 1.
4. PRIMARY AND SECONDARY CONTAINMENT OF WASTEWATER FLOWS DURING PUMP STATION CATASTROHPIC EVENTS ARE TO BE CLEANED UP BY CITY OPERATIONS, SHOULD A SPILL OCCUR.
5. STORMWATER RUNOFF FROM THE PROPOSED SUBDIVISION IS TO BE TREATED VIA A WATER QULAITY TREATMENT UNIT (WQT) FOR ENHANCED LEVEL TREATMENT PER MECP GUIDELINES (80% TSS REMOVAL).

No.	REVISION	DATE	BY
I.	PRELIMINARY DESIGN	JUN 09/23	BHB

SCALE
AS SHOWN

DESIGN	BPR
CHECKED	BHB
DRAWN	BPR
CHECKED	BHB
APPROVED	BHB

FOR REVIEW ONLY

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Ottawa, Ontario, Canada K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

CITY of OTTAWA
STINSON SUBDIVISION
4386 RIDEAU VALLEY DRIVE

DRAWING NAME
OXBOW PLAN AND PROFILE
WITH SWM/SAN OVERFLOW
CONTAINMENT

PROJECT No. 121153
REV REV #1
DRAWING No. 121153-OXBOW

PLAN24x36.DWG - 914.4mmx609.6mm

Date: September 13, 2023

File No.: D07-16-22-0026

To: Sam Bahia & Brendan Rundle - Novatech

From: Erica Ogden-Fedak – City of Ottawa

CC: Ryan Polkinghorne, Matthew Hayley, John Bougadis, Joseph Zegorski, Hasnaa Zaknoun, Eva Spal, Brian Morgan, & Damien Whittaker – City of Ottawa
Ryan MacDougall – Uniform
Greg Winters, James Ireland - Novatech

Re: **4386 Rideau Valley Drive – Stormwater & Sanitary Outflows to Existing Oxbow**

The City of Ottawa has reviewed the Memorandum from Novatech dated June 30, 2023, regarding the Stormwater and Sanitary Outflows to the existing oxbow related to the Plan of Subdivision application at 4386 Rideau Valley Drive in the Village of Manotick.

Stormwater Outlet

The City has determined that the proposed stormwater outlet to the oxbow is not acceptable for the operation of the oxbow. Based on internal discussions amongst City departments, and review of the information provided, the City has concerns regarding:

- the future maintenance of the oxbow feature when used as a stormwater facility,
- impacts to the significant wildlife habitat (including possible species at risk) within the oxbow and;
- increased velocity and erosion.

The City requests that the stormwater outlet be directly to Mud Creek. The new stormwater outlet location must ensure velocity is addressed, appropriate maintenance and access corridors are provided to the outlet structure, and baseflow is maintained to the oxbow feature.

Sanitary Emergency Overflow

The City will proceed with the original IBI design for the Sanitary Emergency Overflow. The timing of the emergency overflow project will require coordination with the proposed plan of subdivision to ensure access to the lands for the installation of the emergency overflow. The required upgrades to the pump station to increase capacity cannot be completed without the completion of the emergency overflow. As the proposed plan of subdivision is dependent on the increased capacity at the pumping station, coordination between the development application and construction of the emergency overflow will be required by all parties.

Next Steps

- Please proceed with a revised submission for the Plan of Subdivision application which incorporates an alternative stormwater outlet.
- Coordination for access to construct the Sanitary Emergency Overflow prior to registration of the subdivision.

Date: October 6, 2023

File No.: D07-16-22-0026

To: Sam Bahia & Brendan Rundle - Novatech

From: Erica Ogden-Fedak – City of Ottawa

CC: Ryan Polkinghorne, Matthew Hayley, John Bougadis, Joseph Zegorski, Hasnaa Zaknoun, Eva Spal, Brian Morgan, & Damien Whittaker – City of Ottawa
Ryan MacDougall – Uniform
Greg Winters, James Ireland - Novatech

Re: **Follow-up - 4386 Rideau Valley Drive – Stormwater & Sanitary Outflows to Existing Oxbow**

As a follow up to the City's initial memorandum, dated September 13, 2023, regarding the Stormwater & Sanitary Outflows to the Existing Oxbow at 4386 Rideau Valley Drive, please find below two options to be considered.

As outlined in our initial memorandum, the City continues to have concerns regarding the future maintenance requirements for the oxbow feature when used as a stormwater facility, impacts to significant wildlife habitat and increased velocity and erosion.

The City's Infrastructure & Water Services Department has advised that maintenance within the oxbow will not be provided, and it is anticipated that over time the oxbow will fill with sediment and silt.

Option 1 – Relocate Stormwater Outlet to Mud Creek

As outlined in our initial memo, relocating the stormwater outlet directly to Mud Creek continues to be the City's preferred approach to stormwater management for the proposed Plan of Subdivision.

Should the applicant choose to proceed with this option, the draft plan of subdivision application can proceed independently from the City led project for the Emergency Sanitary Overflow.

The timing of the emergency overflow project will continue to require coordination with the proposed plan of subdivision to ensure access to the lands for the installation of the emergency overflow. The required upgrades to the pump station to increase capacity cannot be completed without the completion of the emergency overflow. As the proposed plan of subdivision is dependent on the increased capacity at the pumping station, coordination between the development application and construction of the emergency overflow will be required by all parties.

Option 2 – Combined Stormwater Outlet and Emergency Sanitary Overflow to Oxbow

The City is willing to consider a combined stormwater outlet and emergency sanitary overflow to the oxbow, but will require that, as a part of the City's project for capacity upgrades to the Manotick Pumping Station, a consultant be retained to review the options for both the stormwater outlet and emergency sanitary overflow to the oxbow. This process will require discussions with the Ministry of the Environmental, Conservation and Parks regarding the Environmental Compliance Approval, as well as Parks Canada regarding impacts to the Rideau River.

It is anticipated that this process will take longer to resolve than Option 1. The City is not prepared to issue Draft Plan Approval until this process has been resolved. The City does not guarantee that this process will result in a stormwater outlet to the oxbow.

Next Steps

Please advise the City of your selected option for the stormwater management outlet.

NOVATECH
Engineers, Planners & Landscape Architects

M E M O R A N D U M

DATE: **JANUARY 30, 2024**

TO: **ERICA OGDEN-FIDAK**

FROM: **SAM BAHIA, BEN SWEET**

RE: **4386 RIDEAU VALLEY DRIVE – STINSON LANDS – STORMWATER & SANITARY OUTFLOWS TO EXISTING OXBOW**

CC: **ADAM BROWN, JOHN RIDDELL, GREG WINTERS, RYAN MACDOUGALL**

As discussed in mid-January, we have revisited the stormwater alternatives for 4386 Rideau Valley Drive (Subject Site).

Prior Alternatives

The previous alternatives to address the Subject Site and ownership issues of the Oxbow, described below.

- **Alternative 1:** Minor and Major Storm outlet to the Oxbow (by Uniform) + Manotick PS Overflow to the Oxbow and a modified Weir at the Oxbow/Mud Creek Confluence that could be used to detain overflow volumes (by the City).
- **Alternative 2:** Minor and Major Storm outlet to Mud Creek (by Uniform) + Manotick PS Overflow to the Oxbow and Weir at the Oxbow/Mud Creek Confluence that could be used to detain overflow volumes (by the City).
- We had investigated directing the Minor Storm System to the Rideau River by crossing Rideau Valley Drive, near the Oxbow, north of the Manotick PS. It proved to be technically difficult and costly as it would have required an open cut road crossing of Rideau Valley Drive and potential conflicts with two live wastewater Manotick PS forcemains, a deep sanitary trunk from Hillside Gardens, the Manotick PS Overflow, and a vulnerable in-service watermain for the Village.

City Infrastructure Planning Staff had concerns with Alternative 1 as it complicated existing approvals for the PS Overflow (from Parks Canada and MECP) due to the introduction of post-development storm flows from the Subject Site to the Oxbow. Furthermore, Stormwater Operations were concerned with maintenance of the environmentally sensitive Oxbow as it provides conveyance for post-development treated flows.

Uniform and Novatech had concerns with Alternative 2 as it would require additional modelling and input from a Drainage Act perspective, as it connects to Mud Creek, which has status under the Act. In addition, Mud Creek which is erosion sensitive would require additional mitigation measures because of post-development flows and volumes. Furthermore, the Oxbow's hydrologic function would be reduced if the flows are directed to Mud Creek.

New Alternative 3

Upon further review, the following alternative has been contemplated.

- **Alternative 3:** Minor and Major Storm outlet to the Rideau River, south of the Manotick PS, (by Uniform) + Manotick PS Overflow to the Oxbow and Weir at the Oxbow/Mud Creek Confluence that could be used to detain overflow volumes (by the City).

Alternative 3 would still require an open cut road crossing of Rideau Valley Drive but it would be at the same location of the open cut required for the sanitary servicing outlet for the Subject Site. It would also avoid potential conflict with the two live wastewater Manotick PS forcemains, and the Manotick PS Overflow, given the crossing would occur above the deeper gravity sanitary trunk.

Refer to Drawing 121153-GP (Alternatives Markup) attached which demonstrates all the alternatives.

It should be noted that Alternatives 2 and 3 result in an additional cost premium of 10% above Alternative 1.

Next Steps and Conclusion

Alternative 3 appears to be the best solution moving forward as it addresses City Infrastructure Planning Staff and Stormwater Operations concerns with respect to the existing approvals for the PS Overflow and maintenance of the environmentally sensitive Oxbow, and Uniform/Novatech's concerns with having a direct outlet to Mud Creek that becomes contingent on Drainage Act approvals.

In addition, upon review of the Oxbow water balance under post-development conditions, there will be sufficient runoff from the rear yards of units backing on to Mud Creek to maintain the normal water level and retention volume to preserve the Oxbow's hydrologic function. It is also important to note that the Oxbow will also periodically be inundated by backwater effects from Mud Creek during spring freshets and annual storm events.

Uniform is prepared to move forward with Alternative 3 despite the cost premium to continue to advance the file, if City Staff can provide buy-in. Alternative 3 would also allow Uniform to carve out the Oxbow lands in advance of subdivision registration pending further discussions/agreement about timing and continued drainage rights to the Oxbow (for the rear yards). This would allow the City to advance the Mantoick PS Upgrades and the previously approved PS Overflow.

We trust the above addresses City Staff's concerns brought forward in late 2023.

Please feel free to call to discuss further. We can also arrange a second meeting should there be further questions and concerns.

Attachment(s):

- 121153-GP (Alternatives Markup)

M:\2021\121153\CAD\Design\Figures\121153-SERVICINGFIGURE_V3.dwg, PLANS-A1, Aug 21, 2023 - 3:21pm, brunde

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
2.	ISSUED FOR REVIEW	JAN 30/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE

DESIGN BPR
CHECKED BHB
DRAWN BPR
CHECKED BHB
APPROVED BHB

FOR REVIEW ONLY

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

LOCATION
CITY OF OTTAWA
STINSON LANDS SUBDIVISION

DRAWING NAME
GENERAL PLAN OF SERVICES

PROJECT No. 121153-00
REV REV # 2
DRAWING No. 121153-GP

PLANA1.DWG - 841mmx594mm

Date: February 22, 2024
File No.: D07-16-22-0026
To: Sam Bahia & Ben Sweet - Novatech
From: Erica Ogden-Fedak – City of Ottawa
CC: Ryan Polkinghorne, Matthew Hayley, John Bougadis, Joseph Zegorski, Hasnaa Zaknoun, Eva Spal, Brian Morgan, Damien Whittaker, Pamela Hayes, Justin Caouette – City of Ottawa
Eric Lalande, Amanda Lange, Evelyn Liu - RVCA
Ryan MacDougall – Uniform
Greg Winters, James Ireland, John Riddell - Novatech

Re: **Follow-up - 4386 Rideau Valley Drive – Stormwater & Sanitary Outflows**

The City of Ottawa has reviewed the Novatech Memorandum dated January 30, 2024, regarding *"4386 Rideau Valley Drive – Stinson Lands – Stormwater & Sanitary Outflows to Existing Oxbow".*

> **Alternative 3**: Minor and Major Storm outlet to the Rideau River, south of the Manotick Pump Station, (by Uniform) and Manotick Pump Station Overflow to the Oxbow and Weir at the Oxbow/Mud Creek Confluence that could be used to detain overflow volumes (by the City).

The City is conceptually satisfied with Alternative 3 and is comfortable with the applicant proceeding to design this stormwater alternative.

The following comments should be considered in the design of the stormwater outlet:

- Avoid impacts to existing water and sanitary services within Rideau Valley Drive.
- Transfer of oxbow lands to the City, prior to subdivision registration, to allow capacity upgrades which the subdivision requires to proceed.
- Input from Parks Canada for a stormwater outlet directly to the Rideau River will be collected through the next subdivision submission circulation. Depending on the location of the outlet, permits from Parks Canada may be required. Any coordination with Parks Canada should be liaised through the City of Ottawa.
- Stormwater outlet will require appropriate access for vehicles, to allow future maintenance.
- Transfer of the land for the stormwater outlet to the City will be required through the subdivision process.

- Ensure the OGS is accessible and oriented towards Alternative 3.
- Erosion Control measures should be incorporated with the stormwater outlet.
- Maintain rear yard overland flow from lots backing onto the oxbow.
- Permits from the Conservation Authority will be required.
- In water works will have timing restrictions for construction activities.
- Stormwater design parameters (quantity/quality) will be handled through detailed design and should be sufficient for ECA approval of the outlet.
- Timing of construction should be considered and impacts to traffic on Rideau Valley Drive.
- Alternative 3 would not require an engineering review for the Mud Creek Drain hydrology/hydraulics.
- As the Wilson-Cowan Drain watershed boundary would be modified, the City would be required to appoint a Drainage Engineer to undertake a S.65 Report to adjust the assessment schedules for future maintenance to reflect these changes.

Next Steps

Please proceed with a complete resubmission for the subdivision application, which includes Alternative 3 for the stormwater outlet. This submission will be circulated to all parties for review.

Appendix B
Servicing Report Checklist

Development Servicing Study Checklist

Project Name: Stinson Lands
Project Number: 121153
Date: March 11, 2025

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 3.1 & 3.2	
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.	Y		
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 3.3 & 3.4	

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.	Y	6	
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 & 3.2	
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 & 6.2	

Development Servicing Study Checklist

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.	Y	5	
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.	Y	5	
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		

Development Servicing Study Checklist

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	2.2	

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.	Y	App A	
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: 12/10/2024
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																	CAPACITY								
			AREA								FLOW									PROPOSED SEWER PIPE SIZING / DESIGN								
STREET	FROM MH	TO MH	AREA ID	HIGH DENSITY	ROAD	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)			PEAK FLOW	TOTAL UNCONTROLLED PEAK FLOW (QDesign)	TOTAL RESTRICTED PEAK FLOW (Q)	PIPE PROPERTIES					FULL FLOW CAPACITY	FULL FLOW VELOCITY	TIME OF FLOW	QPEAK DESIGN / QFULL
														2yr	5yr	100yr				LENGTH	SIZE / MATERIAL	ID ACTUAL	ROUGHNESS	DESIGN GRADE				
				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	100	102	C13		0.25				0.25 0.00 0.00	0.70	0.49 0.00 0.00	0.49 0.00 0.00	10.00 10.00 10.00	76.81			37.37 0.00 0.00	37.4		82.8	250 PVC	0.254	0.013	1.30	70.7	1.40	0.99	52.8%
	102	104	-						0.00 0.00 0.00		0.00 0.00 0.00	0.49 0.00 0.00	10.99 10.99 10.99	73.21			35.62 0.00 0.00	35.6		45.7	250 PVC	0.254	0.013	1.30	70.7	1.40	0.55	50.4%
	104	106	C12		0.33				0.33 0.00 0.00	0.70	0.64 0.00 0.00	1.13 0.00 0.00	11.53 11.53 11.53	71.38			80.56 0.00 0.00	80.6		29.9	300 PVC	0.305	0.013	1.50	123.6	1.69	0.29	65.2%
	106	108	-						0.00 0.00 0.00		0.00 0.00 0.00	1.13 0.00 0.00	11.83 11.83 11.83	70.43			79.49 0.00 0.00	79.5		19.0	375 PVC	0.381	0.013	1.50	224.0	1.96	0.16	35.5%
	108	110	C10		0.22				0.22 0.00 0.00	0.70	0.43 0.00 0.00	1.56 0.00 0.00	11.99 11.99 11.99	69.93			108.86 0.00 0.00	108.9		18.1	450 PVC	0.457	0.013	1.00	297.4	1.81	0.17	36.6%
Street 2	112	114	C01		0.33				0.33 0.00 0.00	0.70	0.64 0.00 0.00	0.64 0.00 0.00	10.00 10.00 10.00	76.81			49.32 0.00 0.00	49.3		63.3	300 PVC	0.305	0.013	0.75	87.4	1.20	0.88	56.5%
	114	116	-						0.00 0.00 0.00		0.00 0.00 0.00	0.64 0.00 0.00	10.88 10.88 10.88	73.58			47.25 0.00 0.00	47.3		12.2	300 PVC	0.305	0.013	0.75	87.4	1.20	0.17	54.1%
	116	118	-						0.00 0.00 0.00		0.00 0.00 0.00	0.64 0.00 0.00	11.05 11.05 11.05	72.99			46.87 0.00 0.00	46.9		47.7	300 PVC	0.305	0.013	0.75	87.4	1.20	0.66	53.7%
	118	120	C02		0.29				0.29 0.00 0.00	0.70	0.56 0.00 0.00	1.21 0.00 0.00	10.88 10.88 10.88	73.58			88.78 0.00 0.00	88.8		30.4	375 PVC	0.381	0.013	0.75	158.4	1.39	0.37	56.0%
	120	122	C03		0.28				0.28 0.00 0.00	0.70	0.54 0.00 0.00	1.75 0.00 0.00	11.25 11.25 11.25	72.33			126.68 0.00 0.00	126.7		51.8	525 CONC	0.533	0.013	0.75	388.5	1.74	0.50	32.6%
	122	124	C04		0.31				0.31 0.00 0.00	0.70	0.60 0.00 0.00	2.35 0.00 0.00	11.74 11.74 11.74	70.70			166.48 0.00 0.00	166.5		54.5	600 CONC	0.610	0.013	0.75	554.7	1.90	0.48	30.0%
Street 2	128	126	C06		0.32				0.32 0.00 0.00	0.70	0.62 0.00 0.00	0.62 0.00 0.00	10.00 10.00 10.00	76.81			47.83 0.00 0.00	47.8		63.8	450 PVC	0.457	0.013	0.25	148.7	0.91	1.17	32.2%
	126	124	C05		0.18				0.18 0.00 0.00	0.70	0.35 0.00 0.00	0.97 0.00 0.00	11.17 11.17 11.17	72.57			70.61 0.00 0.00	70.6		28.7	450 PVC	0.457	0.013	0.25	148.7	0.91	0.53	47.5%
Street 3	124	130	-						0.00 0.00 0.00		0.00 0.00 0.00	3.33 0.00 0.00	12.22 12.22 12.22	69.21			230.32 0.00 0.00	230.3		30.3	600 CONC	0.610	0.013	0.25	320.3	1.10	0.46	71.9%
	130	110	C08,C11		0.17	0.60			0.77 0.00 0.00	0.54	1.16 0.00 0.00	4.49 0.00 0.00	12.68 12.68 12.68	67.85			304.79 0.00 0.00	304.8		68.4	675 CONC	0.686	0.013	0.25	438.5	1.19	0.96	69.5%
Street 3	110	132	C09,C14		0.22	0.56			0.78 0.00 0.00	0.56	1.21 0.00 0.00	7.26 0.00 0.00	13.64 13.64 13.64	65.17			472.88 0.00 0.00	472.9		76.1	825 CONC	0.838	0.013	0.25	748.8	1.36	0.93	63.2%
	132	134	C17		0.12				0.12 0.00 0.00	0.70	0.23 0.00 0.00	7.49 0.00 0.00	15.00 15.00 15.00	61.77			462.63 0.00 0.00	462.6		10.7	825 CONC	0.838	0.013	0.25	748.8	1.36	0.13	61.8%
Street 3	148	146	C22		0.21				0.21 0.00 0.00	0.70	0.41 0.00 0.00	0.41 0.00 0.00	10.00 10.00 10.00	76.81			31.39 0.00 0.00	31.4		75.6	250 PVC	0.254	0.013	5.00	138.7	2.74	0.46	22.6%
	146	144	C19,C21		0.17	0.40			0.57 0.00 0.00	0.56	0.89 0.00 0.00	1.30 0.00 0.00	10.46 10.46 10.46	75.08			97.27 0.00 0.00	97.3		67.5	300 PVC	0.305	0.013	3.00	174.7	2.39	0.47	55.7%
	144	140	C20						0.00 0.00 0.00		0.00 0.00 0.00	1.30 0.00 0.00	10.93 10.93 10.93	73.41			95.10 0.00 0.00	95.1		13.6	750 CONC	0.762	0.013	3.80	2264.0	4.96	0.05	4.2%
Street 3	138	136	C16		0.35				0.35 0.00 0.00	0.70	0.68 0.00 0.00	0.68 0.00 0.00	10.00 10.00 10.00	76.81			52.31 0.00 0.00	52.3		88.3	300 PVC	0.305	0.013	4.50	214.0	2.93	0.50	24.4%
	136	134	C15		0.33				0.33 0.00 0.00	0.70	0.64 0.00 0.00	1.32 0.00 0.00	10.50 10.50 10.50	74.93			99.15 0.00 0.00	99.2		57.4	375 PVC	0.381	0.013	3.00	316.8	2.78	0.34	31.3%
Street 3	134	140	C18		0.19				0.19 0.00 0.00	0.70	0.37 0.00 0.00	9.18 0.00 0.00	15.13 15.13 15.13	61.46			564.38 0.00 0.00	564.4		41.1	825 CONC	0.838	0.013	1.00	1497.5	2.71	0.25	37.7%

STORM SEWER DESIGN SHEET

LOCATION			DEMAND																	CAPACITY								
			AREA								FLOW									PROPOSED SEWER PIPE SIZING / DESIGN								
														RAIN INTENSITY (mm/hr)						PIPE PROPERTIES								
STREET	FROM MH	TO MH	AREA ID	HIGH DENSITY	ROAD	REAR YARD 1	REAR YARD 2	PARK	TOTAL AREA	WEIGHTED RUNOFF COEFFICIENT	INDIVI 2.78 AR	ACCUM 2.78 AR	TIME OF CONC	2yr	5yr	100yr	PEAK FLOW	TOTAL UNCONTROLLED PEAK FLOW (QDesign)	TOTAL RESTRICTED PEAK FLOW (Q)	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.)	(%)
Easement Block	140	142	-						0.00		0.00	10.48	15.38	60.88			637.90	**637.9**		5.1	1050 CONC	1.067	0.013	1.00	2848.8	3.19	0.03	22.4%
									0.00		0.00	0.00	15.38				0.00											
									0.00		0.00	0.00	15.38				0.00											
	142	144	-						0.00		0.00	10.48	15.41	60.82			637.26	**637.3**		9.9	1050 CONC	1.067	0.013	1.00	2848.8	3.19	0.05	22.4%
									0.00		0.00	0.00	15.41				0.00											
									0.00		0.00	0.00	15.41				0.00											
	144	OUTLET							0.00		0.00	10.48	15.46	60.70			636.02	**636.0**		40.0	1200 CONC	1.219	0.013	0.10	1286.2	1.10	0.61	49.5%
									0.00		0.00	0.00	15.46				0.00											
									0.00		0.00	0.00	15.46				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

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

Stinson Lands
Pre-Development Model Schematic

NOVATECH
Engineers, Planners & Landscape Architects

3/28/2024
PREPARED BY: NOVATECH

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Stinson Lands

Post-Development Model Parameters

NOVATECH
Engineers, Planners & Landscape Architects

Area ID	Catchment Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.240	0.45	36%	100%	25.02	97.54	1.0%
B-01	0.710	0.45	36%	100%	21.31	334.06	1.0%
C-01	0.330	0.70	71%	45%	20.51	161.84	1.0%
C-02	0.290	0.70	71%	45%	24.44	117.42	1.0%
C-03	0.280	0.70	71%	45%	23.37	118.54	1.0%
C-04	0.310	0.70	71%	45%	23.12	135.79	1.0%
C-05	0.180	0.70	71%	45%	23.02	76.46	1.0%
C-06	0.320	0.70	71%	45%	34.25	94.31	1.0%
C-07	0.670	0.45	36%	100%	64.21	106.68	1.0%
C-08	0.170	0.70	71%	45%	22.85	73.96	1.0%
C-09	0.220	0.70	71%	45%	22.23	97.19	1.0%
C-10	0.220	0.70	71%	45%	22.65	98.01	1.0%
C-11	0.600	0.45	36%	100%	19.05	316.00	1.0%
C-12	0.330	0.70	71%	45%	19.65	166.94	1.0%
C-13	0.250	0.70	71%	45%	23.49	106.41	1.0%
C-14	0.560	0.45	36%	100%	14.18	397.06	1.0%
C-15	0.330	0.70	71%	45%	22.08	152.74	1.0%
C-16	0.350	0.70	71%	45%	21.84	160.71	1.0%
C-17	0.120	0.70	71%	45%	22.88	51.13	1.0%
C-18	0.190	0.70	71%	45%	21.60	85.67	1.0%
C-19	0.400	0.45	36%	45%	13.84	289.76	1.0%
C-20	0.120	0.45	36%	0%	22.12	54.25	1.0%
C-21	0.170	0.70	71%	100%	18.95	88.64	1.0%
C-22	0.210	0.70	71%	100%	19.02	111.49	1.0%
D-01	0.180	0.20	0%	0%	20.63	87.76	1.0%
TOTAL:	**7.75**						

3/28/2024
PREPARED BY: NOVATECH

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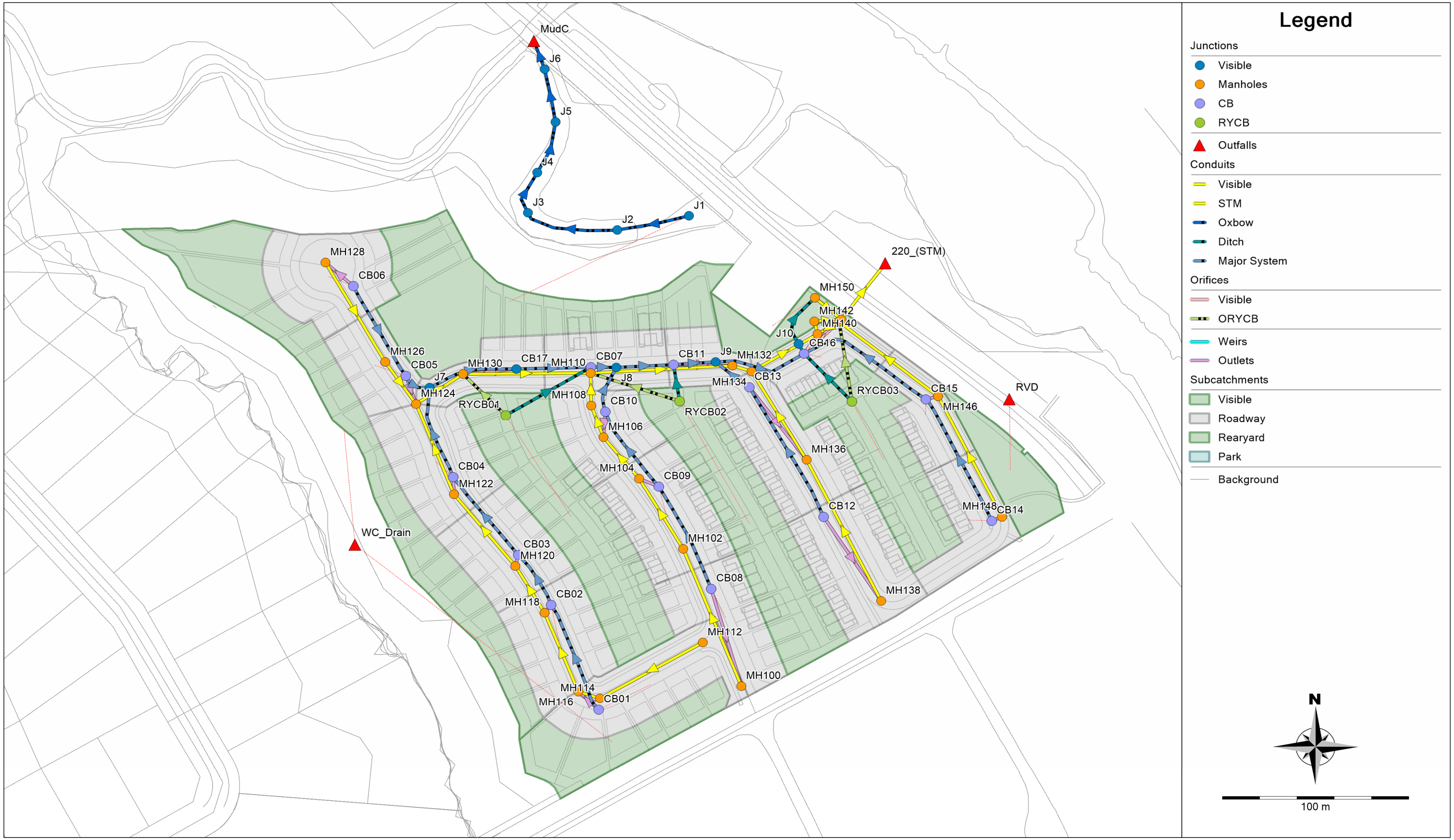
Stinson Lands
Overall Model Schematic (Post-Development)

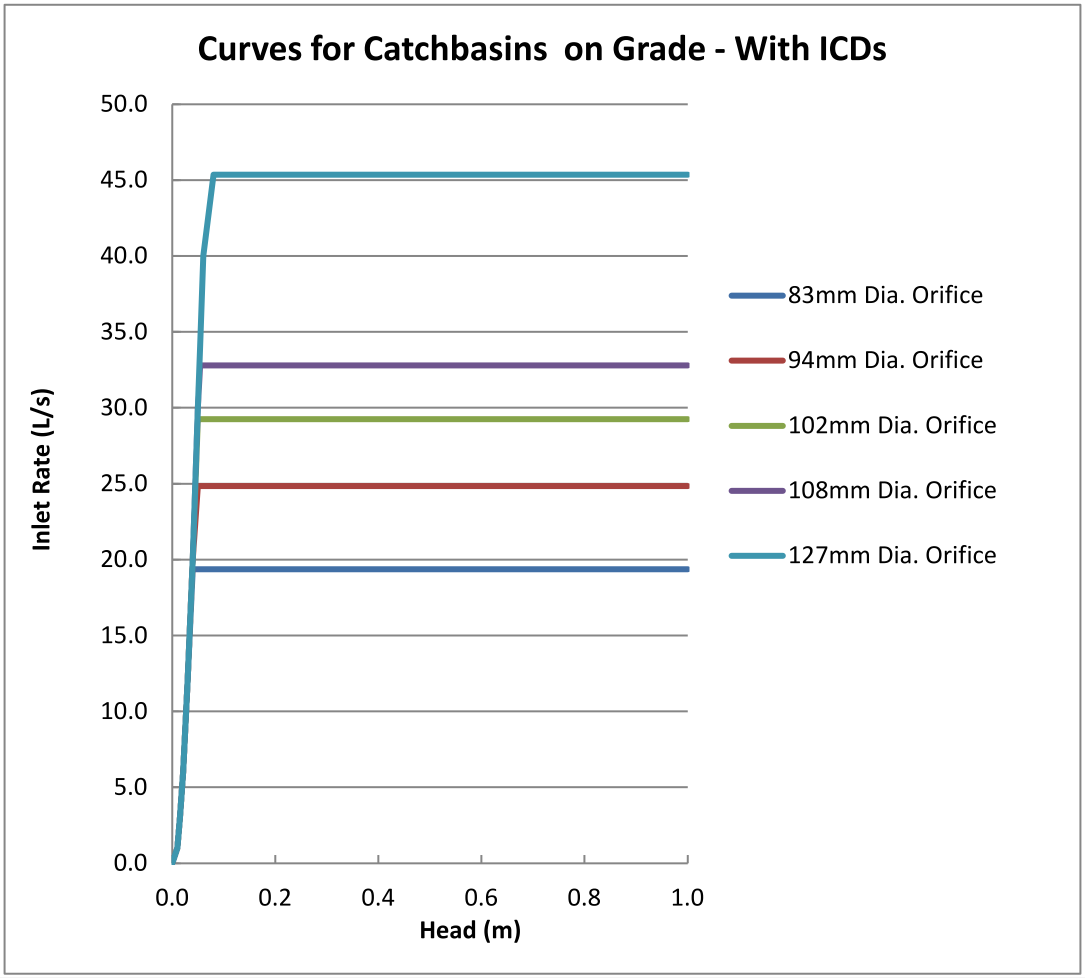
NOVATECH
Engineers, Planners & Landscape Architects



3/28/2024
PREPARED BY: NOVATECH

M:\2021\121153\DATA\Calculations\Sewer Calcs\SWM\PCSWMM Models\Draft Plan\Second Submission\121153-ModelParams_DP_R2.xlsx

Stinson Lands
Catchbasin (On-Grade) with ICD Curves



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).

Date: 8/2/2022

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)
MH100	87.92	90.70	87.92	0.25	88.17	88.08	-0.09
MH102	86.81	89.34	86.81	0.25	87.06	86.99	-0.07
MH104	86.17	88.62	86.17	0.30	86.47	86.87	0.40
MH106	85.63	88.13	85.63	0.38	86.01	86.18	0.18
MH108	85.27	87.85	85.27	0.45	85.72	85.84	0.12
MH110	84.72	87.82	84.72	0.82	85.54	85.62	0.08
MH112	87.53	89.76	87.53	0.30	87.83	87.53	-0.30
MH114	87.03	89.56	87.03	0.30	87.33	87.09	-0.24
MH116	86.91	89.56	86.91	0.30	87.21	87.09	-0.12
MH118	86.47	89.24	86.47	0.38	86.85	86.73	-0.11
MH120	86.09	89.00	86.09	0.52	86.61	86.36	-0.25
MH122	85.63	88.56	85.63	0.60	86.23	86.11	-0.12
MH124	85.19	88.18	85.19	0.60	85.79	85.97	0.18
MH126	85.41	88.20	85.41	0.45	85.86	85.99	0.13
MH128	85.60	88.31	85.60	0.45	86.05	86.00	-0.05
MH130	85.04	87.95	85.04	0.68	85.72	85.84	0.12
MH132	84.49	87.62	84.49	0.82	85.31	85.35	0.04
MH134	84.44	87.55	84.44	0.82	85.26	85.19	-0.07
MH136	86.64	89.40	86.64	0.38	87.02	86.83	-0.19
MH138	90.68	93.25	90.68	0.30	90.98	90.79	-0.19
MH140	83.45	86.17	83.45	1.05	84.50	84.25	-0.25
MH142	82.92	86.64	82.92	1.05	83.97	83.62	-0.35
MH144	82.22	87.91	84.61	0.75	85.36	84.86	-0.50
MH146	87.09	89.29	87.09	0.30	87.39	87.09	-0.30
MH148	90.92	93.22	90.92	0.25	91.17	90.92	-0.25
MH150	83.00	86.38	83.00	0.90	83.90	83.38	-0.52

3/28/2024
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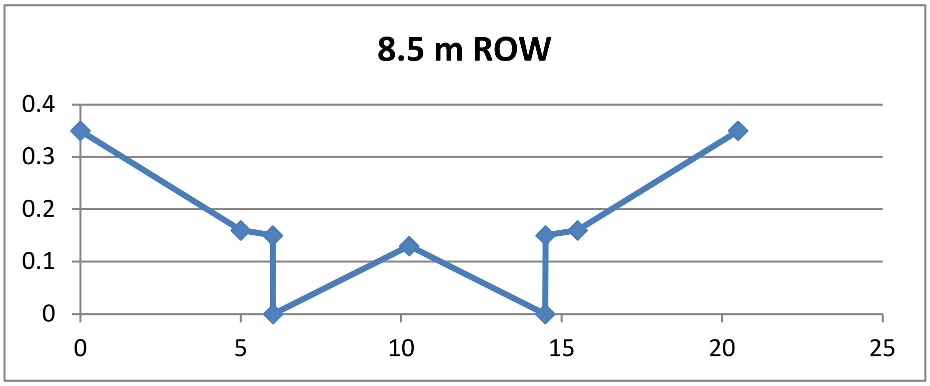
M:\2021\121153\DATA\Calculations\Sewer Calcs\SWM\PCSWMM Models\Draft Plan\Second Submission\121153-ModelParams_DP_R2.xlsx

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



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

C100-6.stm		C100-6+20%.stm	
Duration min	Intensity mm/hr	Duration min	Intensity 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

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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

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.2 (Build 5.2.4)
------------------------------------------------------------

WARNING 03: negative offset ignored for Link C1_1
WARNING 03: negative offset ignored for Link C1_1
WARNING 03: negative offset ignored for Link C1_2
WARNING 03: negative offset ignored for Link C1_2
WARNING 03: negative offset ignored for Link C1_3
WARNING 03: negative offset ignored for Link C1_3
WARNING 03: negative offset ignored for Link C1_4
WARNING 03: negative offset ignored for Link C1_4
WARNING 03: negative offset ignored for Link C1_5
WARNING 03: negative offset ignored for Link C1_5
WARNING 03: negative offset ignored for Link C1_7
WARNING 03: negative offset ignored for Link C1_7
WARNING 02: maximum depth increased for Node CB07
WARNING 02: maximum depth increased for Node CB15
WARNING 02: maximum depth increased for Node CB16
WARNING 02: maximum depth increased for Node J1
WARNING 02: maximum depth increased for Node J2
WARNING 02: maximum depth increased for Node J3
WARNING 02: maximum depth increased for Node J4
WARNING 02: maximum depth increased for Node J6
WARNING 02: maximum depth increased for Node RYCB01

*************
Element Count
*************
Number of rain gages ...... 1
Number of subcatchments ... 25
Number of nodes ........... 61
Number of links ........... 79
Number of pollutants ...... 0
Number of land uses ....... 0


****************
Raingage Summary
****************
                                                    Data       Recording



Name                 Data Source                    Type       Interval
------------------------------------------------------------------------
Raingage             04-C100yr-6hr                  INTENSITY   10 min.


********************
Subcatchment Summary
********************
Name                       Area     Width   %Imperv    %Slope Rain Gage            Outlet
-----------------------------------------------------------------------------------------------------------
A-01                       0.24     97.54     36.00    1.0000 Raingage             WC_Drain
B-01                       0.71    334.06     36.00    1.0000 Raingage             WC_Drain
C-01                       0.33    161.84     71.00    1.0000 Raingage             CB01
C-02                       0.29    117.42     71.00    1.0000 Raingage             CB02
C-03                       0.28    118.54     71.00    1.0000 Raingage             CB03
C-04                       0.31    135.79     71.00    1.0000 Raingage             CB04
C-05                       0.18     76.46     71.00    1.0000 Raingage             CB05
C-06                       0.32     94.31     71.00    1.0000 Raingage             CB06
C-07                       0.69    106.68     36.00    1.0000 Raingage             J1
C-08                       0.17     73.96     71.00    1.0000 Raingage             CB17
C-09                       0.22     97.19     71.00    1.0000 Raingage             CB07
C-10                       0.22     98.01     71.00    1.0000 Raingage             CB10
C-11                       0.60    316.00     36.00    1.0000 Raingage             RYCB01
C-12                       0.33    166.94     71.00    1.0000 Raingage             CB09
C-13                       0.25    106.41     71.00    1.0000 Raingage             CB08
C-14                       0.56    397.06     36.00    1.0000 Raingage             RYCB02
C-15                       0.34    152.74     71.00    1.0000 Raingage             CB13
C-16                       0.35    160.71     71.00    1.0000 Raingage             CB12
C-17                       0.12     51.13     71.00    1.0000 Raingage             CB11
C-18                       0.18     85.67     71.00    1.0000 Raingage             CB16
C-19                       0.40    289.76     36.00    1.0000 Raingage             RYCB03
C-20                       0.12     54.25     36.00    1.0000 Raingage             MH150
C-21                       0.17     88.64     71.00    1.0000 Raingage             CB15
C-22                       0.21    111.49     71.00    1.0000 Raingage             CB14
D-01                       0.18     87.76      0.00    1.0000 Raingage             RVD

************
Node Summary
************
                                         Invert      Max.    Ponded    External
```

```
Name                 Type                 Elev.     Depth      Area    Inflow
-------------------------------------------------------------------------------
CB01                 JUNCTION             89.57      0.35       0.0
CB02                 JUNCTION             89.13      0.35       0.0
CB03                 JUNCTION             88.95      0.35       0.0
CB04                 JUNCTION             88.40      0.35       0.0
CB05                 JUNCTION             88.10      0.35       0.0
CB06                 JUNCTION             88.17      0.35       0.0
CB07                 JUNCTION             87.75      0.38       0.0
CB08                 JUNCTION             89.73      0.35       0.0
CB09                 JUNCTION             88.57      0.35       0.0
CB10                 JUNCTION             87.79      0.35       0.0
CB11                 JUNCTION             87.64      0.35       0.0
CB12                 JUNCTION             91.57      0.35       0.0
CB13                 JUNCTION             88.34      0.35       0.0
CB14                 JUNCTION             93.00      0.35       0.0
CB15                 JUNCTION             89.15      0.46       0.0
CB16                 JUNCTION             85.96      1.75       0.0
CB17                 JUNCTION             87.84      0.35       0.0
J1                   JUNCTION             81.67      2.83       0.0
J10                  JUNCTION             87.38      0.48       0.0
J2                   JUNCTION             80.87      4.43       0.0
J3                   JUNCTION             80.78      4.43       0.0
J4                   JUNCTION             80.50      3.92       0.0
J5                   JUNCTION             80.78      2.00       0.0
J6                   JUNCTION             81.23      2.60       0.0
J7                   JUNCTION             87.95      0.35       0.0
J8                   JUNCTION             87.72      0.35       0.0
J9                   JUNCTION             87.58      0.35       0.0
MH100                JUNCTION             87.92      2.78       0.0
MH102                JUNCTION             86.81      2.53       0.0
MH104                JUNCTION             86.17      2.45       0.0
MH106                JUNCTION             85.63      2.50       0.0
MH108                JUNCTION             85.27      2.58       0.0
MH110                JUNCTION             84.72      3.10       0.0
MH112                JUNCTION             87.53      2.23       0.0
MH114                JUNCTION             87.03      2.53       0.0
MH116                JUNCTION             86.91      2.65       0.0
MH118                JUNCTION             86.47      2.77       0.0
MH120                JUNCTION             86.09      2.91       0.0
MH122                JUNCTION             85.63      2.93       0.0
```

```
MH124                JUNCTION             85.19      2.99       0.0
MH126                JUNCTION             85.41      2.79       0.0
MH128                JUNCTION             85.60      2.71       0.0
MH130                JUNCTION             85.04      2.91       0.0
MH132                JUNCTION             84.49      3.13       0.0
MH134                JUNCTION             84.44      3.11       0.0
MH136                JUNCTION             86.64      2.76       0.0
MH138                JUNCTION             90.68      2.57       0.0
MH140                JUNCTION             83.45      2.72       0.0
MH142                JUNCTION             82.92      3.72       0.0
MH144                JUNCTION             82.22      5.69       0.0
MH144_A              JUNCTION             82.22      4.69       0.0
MH146                JUNCTION             87.09      2.20       0.0
MH148                JUNCTION             90.92      2.30       0.0
MH150                JUNCTION             83.00      3.38       0.0
RYCB01               JUNCTION             87.05      2.00       0.0
RYCB02               JUNCTION             86.75      1.75       0.0
RYCB03               JUNCTION             86.70      1.75       0.0
220_(STM)            OUTFALL              80.00      3.70       0.0
MudC                 OUTFALL              80.97      2.60       0.0
RVD                  OUTFALL               0.00      0.00       0.0
WC_Drain             OUTFALL               0.00      0.00       0.0


************
Link Summary
************
Name             From Node        To Node          Type            Length    %Slope Roughness
---------------------------------------------------------------------------------------------
100-102          MH100            MH102            CONDUIT           82.8    1.3040    0.0130
102-104          MH102            MH104            CONDUIT           45.7    1.2900    0.0130
104-106          MH104            MH106            CONDUIT           29.9    1.5041    0.0130
106-108          MH106            MH108            CONDUIT           19.0    1.5248    0.0130
108-110          MH108            MH110            CONDUIT           18.1    0.9952    0.0130
110-132          MH110            MH132            CONDUIT           76.1    0.2628    0.0130
112-114          MH112            MH114            CONDUIT           63.3    0.7422    0.0130
114-116          MH114            MH116            CONDUIT           12.2    0.7372    0.0130
116-118          MH116            MH118            CONDUIT           47.7    0.7547    0.0130
118-120          MH118            MH120            CONDUIT           30.4    0.7560    0.0130
120-122          MH120            MH122            CONDUIT           51.8    0.7526    0.0130
122-124          MH122            MH124            CONDUIT           54.5    0.7528    0.0130
```

```
124-130          MH124            MH130            CONDUIT            30.3    0.2642    0.0130
126-124          MH126            MH124            CONDUIT            28.7    0.2439    0.0130
128-126          MH128            MH126            CONDUIT            63.8    0.2507    0.0130
130-110          MH130            MH110            CONDUIT            68.4    0.2630    0.0130
132-134          MH132            MH134            CONDUIT            10.7    0.2793    0.0130
134-140          MH134            MH140            CONDUIT            41.1    0.9973    0.0130
136-134          MH136            MH134            CONDUIT            57.5    2.9949    0.0130
138-136          MH138            MH136            CONDUIT            88.3    4.5000    0.0130
140-142          MH140            MH142            CONDUIT             7.4    2.0374    0.0130
142-144A         MH142            MH144_A          CONDUIT            13.4    0.7445    0.0130
144-140          MH144            MH140            CONDUIT            11.2    4.5542    0.0130
144A-220         MH144_A          220_(STM)        CONDUIT            40.9    0.0977    0.0130
146-144          MH146            MH144            CONDUIT            67.5    3.0071    0.0130
148-146          MH148            MH146            CONDUIT            75.7    4.9971    0.0130
225-144A         MH150            MH144_A          CONDUIT            18.6    0.9690    0.0130
C1               CB06             CB05             CONDUIT            57.5    0.1218    0.0160
C1_1             J1               J2               CONDUIT            39.3    2.0378    0.0350
C1_2             J3               J4               CONDUIT            25.6    1.0923    0.0350
C1_3             J2               J3               CONDUIT            51.0    0.1766    0.0350
C1_4             J4               J5               CONDUIT            30.3   -0.9238    0.0350
C1_5             J5               J6               CONDUIT            30.0   -1.5018    0.0350
C1_7             J6               MudC             CONDUIT            16.9    1.5397    0.0350
C10              J8               CB11             CONDUIT            30.6    0.2613    0.0160
C11              CB11             J9               CONDUIT            22.7    0.2641    0.0160
C12              CB10             J8               CONDUIT            26.4    0.2647    0.0160
C13              CB09             CB10             CONDUIT            51.2    1.5247    0.0160
C14              CB08             CB09             CONDUIT            63.8    1.8187    0.0160
C15              CB12             CB13             CONDUIT            82.5    3.9163    0.0160
C16              CB13             J9               CONDUIT            23.1    3.2863    0.0160
C17              CB14             CB15             CONDUIT            76.6    5.0312    0.0160
C18              CB15             CB16             CONDUIT            78.1    2.2916    0.0160
C19              J9               CB16             CONDUIT            51.2    0.4301    0.0160
C21              RYCB01           CB07             CONDUIT            52.9    1.7406    0.0350
C22              RYCB02           CB11             CONDUIT            20.6    2.4813    0.0350
C23              RYCB03           CB16             CONDUIT            36.9    2.0058    0.0350
C24_1            J10              CB16             CONDUIT             6.2    0.3206    0.0250
C24_2            J10              MH150            CONDUIT            31.3    4.3235    0.0250
C3               CB01             CB02             CONDUIT            64.2    0.6852    0.0160
C4               CB02             CB03             CONDUIT            33.4    0.5397    0.0160
C5               CB03             CB04             CONDUIT            55.7    0.9876    0.0160
C6               CB04             J7               CONDUIT            53.0    0.8497    0.0160
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C7               CB05             J7               CONDUIT            15.8    0.9494    0.0160
C8_1             J7               CB17             CONDUIT            48.8    0.2256    0.0160
C8_2             CB17             CB07             CONDUIT            39.9    0.2254    0.0160
C9               CB07             J8               CONDUIT            13.7    0.2186    0.0160
OL16             CB16             MH144            ORIFICE
OR1              CB16             MH144            ORIFICE
OR10             RYCB03           MH144            ORIFICE
OR8              RYCB01           MH130            ORIFICE
OR9              RYCB02           MH110            ORIFICE
W1               MH144            MH144_A          WEIR
OL1              CB06             MH128            OUTLET
OL10             CB02             MH118            OUTLET
OL11             CB03             MH120            OUTLET
OL12             CB12             MH138            OUTLET
OL13             CB13             MH136            OUTLET
OL14             CB14             MH148            OUTLET
OL15             CB15             MH146            OUTLET
OL17             CB17             MH130            OUTLET
OL2              CB05             MH124            OUTLET
OL3              CB04             MH122            OUTLET
OL4              CB07             MH110            OUTLET
OL5              CB11             MH132            OUTLET
OL6              CB10             MH106            OUTLET
OL7              CB09             MH104            OUTLET
OL8              CB08             MH100            OUTLET
OL9              CB01             MH116            OUTLET


*********************
Cross Section Summary
*********************
                                      Full     Full     Hyd.     Max.   No. of     Full
Conduit          Shape               Depth     Area     Rad.    Width  Barrels     Flow
---------------------------------------------------------------------------------------
100-102          CIRCULAR             0.25     0.05     0.06     0.25        1    70.85
102-104          CIRCULAR             0.25     0.05     0.06     0.25        1    70.47
104-106          CIRCULAR             0.30     0.07     0.08     0.30        1   123.95
106-108          CIRCULAR             0.38     0.11     0.10     0.38        1   225.88
108-110          CIRCULAR             0.46     0.16     0.11     0.46        1   296.40
110-132          CIRCULAR             0.84     0.55     0.21     0.84        1   767.25
112-114          CIRCULAR             0.30     0.07     0.08     0.30        1    87.07
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114-116          CIRCULAR             0.30     0.07     0.08     0.30        1    86.78
116-118          CIRCULAR             0.30     0.07     0.08     0.30        1    87.80
118-120          CIRCULAR             0.38     0.11     0.10     0.38        1   159.04
120-122          CIRCULAR             0.53     0.22     0.13     0.53        1   388.47
122-124          CIRCULAR             0.61     0.29     0.15     0.61        1   556.77
124-130          CIRCULAR             0.61     0.29     0.15     0.61        1   329.85
126-124          CIRCULAR             0.46     0.16     0.11     0.46        1   146.74
128-126          CIRCULAR             0.46     0.16     0.11     0.46        1   148.77
130-110          CIRCULAR             0.69     0.37     0.17     0.69        1   450.11
132-134          CIRCULAR             0.84     0.55     0.21     0.84        1   791.00
134-140          CIRCULAR             0.84     0.55     0.21     0.84        1  1494.58
136-134          CIRCULAR             0.38     0.11     0.10     0.38        1   316.56
138-136          CIRCULAR             0.30     0.07     0.08     0.30        1   214.39
140-142          CIRCULAR             1.07     0.89     0.27     1.07        1  4068.54
142-144A         CIRCULAR             1.07     0.89     0.27     1.07        1  2459.46
144-140          CIRCULAR             0.76     0.46     0.19     0.76        1  2478.68
144A-220         CIRCULAR             1.22     1.17     0.30     1.22        1  1271.11
146-144          CIRCULAR             0.30     0.07     0.08     0.30        1   175.26
148-146          CIRCULAR             0.25     0.05     0.06     0.25        1   138.69
225-144A         CIRCULAR             0.91     0.66     0.23     0.91        1  1857.01
C1               ROW                  0.35     3.76     0.19    20.50        1  2664.75
C1_1             OX_1                 2.83    33.83     1.79    16.00        1 203027.18
C1_2             OX_3                 3.92    55.62     2.04    23.80        1 267499.83
C1_3             OX_2                 4.43    65.19     2.60    20.80        1 148067.09
C1_4             OX_4                 1.69    16.31     0.90    17.80        1 41633.67
C1_5             OX_5                 1.97    15.54     1.15    13.08        1 59581.93
C1_7             OX_6                 2.60    17.45     1.52    10.70        1 81913.90
C10              ROW                  0.35     3.76     0.19    20.50        1  3903.19
C11              ROW                  0.35     3.76     0.19    20.50        1  3924.11
C12              ROW                  0.35     3.76     0.19    20.50        1  3928.98
C13              ROW                  0.35     3.76     0.19    20.50        1  9429.05
C14              ROW                  0.35     3.76     0.19    20.50        1 10297.79
C15              ROW                  0.35     3.76     0.19    20.50        1 15111.49
C16              ROW                  0.35     3.76     0.19    20.50        1 13842.70
C17              ROW                  0.35     3.76     0.19    20.50        1 17127.85
C18              ROW                  0.35     3.76     0.19    20.50        1 11559.55
C19              ROW                  0.35     3.76     0.19    20.50        1  5007.75
C21              TRIANGULAR           0.30     0.27     0.14     1.80        1   277.42
C22              TRIANGULAR           0.30     0.27     0.14     1.80        1   331.24
C23              TRIANGULAR           0.30     0.27     0.14     1.80        1   297.81
C24_1            TRIANGULAR           0.30     0.27     0.14     1.80        1   166.68
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C24_2            TRIANGULAR           0.30     0.27     0.14     1.80        1   612.13
C3               ROW                  0.35     3.76     0.19    20.50        1  6320.85
C4               ROW                  0.35     3.76     0.19    20.50        1  5609.55
C5               ROW                  0.35     3.76     0.19    20.50        1  7588.45
C6               ROW                  0.35     3.76     0.19    20.50        1  7039.04
C7               ROW                  0.35     3.76     0.19    20.50        1  7440.39
C8_1             ROW                  0.35     3.76     0.19    20.50        1  3626.74
C8_2             ROW                  0.35     3.76     0.19    20.50        1  3625.23
C9               ROW                  0.35     3.76     0.19    20.50        1  3569.92


****************
Transect Summary
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Transect OX_1
Area:
            0.0019     0.0075     0.0153     0.0244     0.0351
            0.0471     0.0606     0.0755     0.0919     0.1095
            0.1273     0.1454     0.1637     0.1823     0.2011
            0.2201     0.2393     0.2587     0.2784     0.2983
            0.3184     0.3387     0.3593     0.3801     0.4011
            0.4224     0.4439     0.4656     0.4875     0.5096
            0.5320     0.5546     0.5774     0.6005     0.6238
            0.6473     0.6710     0.6950     0.7191     0.7435
            0.7682     0.7930     0.8181     0.8434     0.8690
            0.8947     0.9207     0.9469     0.9733     1.0000
Hrad:
            0.0158     0.0337     0.0567     0.0775     0.0969
            0.1155     0.1335     0.1510     0.1682     0.1920
            0.2192     0.2458     0.2718     0.2973     0.3222
            0.3467     0.3707     0.3943     0.4174     0.4402
            0.4625     0.4845     0.5062     0.5275     0.5485
            0.5692     0.5896     0.6097     0.6295     0.6491
            0.6685     0.6876     0.7065     0.7252     0.7436
            0.7619     0.7799     0.7978     0.8155     0.8330
            0.8504     0.8676     0.8846     0.9015     0.9183
            0.9349     0.9514     0.9677     0.9839     1.0000
Width:
            0.1415     0.2625     0.3162     0.3699     0.4235
            0.4772     0.5309     0.5845     0.6382     0.6631
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```
              0.6715     0.6799     0.6884     0.6968     0.7052
              0.7136     0.7221     0.7305     0.7389     0.7473
              0.7557     0.7642     0.7726     0.7810     0.7894
              0.7979     0.8063     0.8147     0.8231     0.8315
              0.8400     0.8484     0.8568     0.8652     0.8737
              0.8821     0.8905     0.8989     0.9074     0.9158
              0.9242     0.9326     0.9410     0.9495     0.9579
              0.9663     0.9747     0.9832     0.9916     1.0000

  Transect OX_2
  Area:
              0.0009     0.0035     0.0078     0.0139     0.0217
              0.0312     0.0419     0.0540     0.0674     0.0820
              0.0979     0.1146     0.1322     0.1508     0.1698
              0.1891     0.2087     0.2285     0.2486     0.2689
              0.2895     0.3104     0.3315     0.3529     0.3745
              0.3964     0.4186     0.4410     0.4637     0.4866
              0.5098     0.5333     0.5570     0.5810     0.6052
              0.6297     0.6545     0.6795     0.7048     0.7303
              0.7561     0.7822     0.8085     0.8351     0.8619
              0.8890     0.9164     0.9440     0.9719     1.0000
  Hrad:
              0.0168     0.0336     0.0504     0.0672     0.0839
              0.1034     0.1232     0.1424     0.1611     0.1795
              0.2007     0.2225     0.2438     0.2654     0.2928
              0.3194     0.3455     0.3709     0.3958     0.4201
              0.4440     0.4674     0.4903     0.5129     0.5350
              0.5567     0.5781     0.5991     0.6198     0.6402
              0.6603     0.6802     0.6997     0.7190     0.7380
              0.7568     0.7754     0.7938     0.8119     0.8299
              0.8476     0.8652     0.8826     0.8998     0.9169
              0.9338     0.9506     0.9672     0.9837     1.0000
  Width:
              0.0615     0.1229     0.1844     0.2459     0.3073
              0.3579     0.4038     0.4496     0.4954     0.5412
              0.5767     0.6083     0.6399     0.6689     0.6781
              0.6873     0.6965     0.7057     0.7149     0.7241
              0.7333     0.7425     0.7517     0.7609     0.7701
              0.7793     0.7885     0.7977     0.8069     0.8161
              0.8253     0.8345     0.8437     0.8528     0.8620
              0.8712     0.8804     0.8896     0.8988     0.9080
```

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              0.9172     0.9264     0.9356     0.9448     0.9540
              0.9632     0.9724     0.9816     0.9908     1.0000

  Transect OX_3
  Area:
              0.0006     0.0022     0.0050     0.0089     0.0138
              0.0199     0.0271     0.0354     0.0448     0.0553
              0.0670     0.0797     0.0935     0.1085     0.1245
              0.1412     0.1585     0.1762     0.1943     0.2130
              0.2322     0.2518     0.2720     0.2926     0.3137
              0.3353     0.3574     0.3800     0.4030     0.4266
              0.4506     0.4751     0.5001     0.5256     0.5516
              0.5781     0.6051     0.6325     0.6605     0.6889
              0.7178     0.7472     0.7771     0.8075     0.8384
              0.8697     0.9015     0.9339     0.9667     1.0000
  Hrad:
              0.0187     0.0375     0.0562     0.0749     0.0936
              0.1124     0.1311     0.1498     0.1685     0.1873
              0.2060     0.2247     0.2434     0.2622     0.2826
              0.3094     0.3353     0.3606     0.3852     0.4092
              0.4328     0.4558     0.4784     0.5006     0.5225
              0.5440     0.5651     0.5860     0.6066     0.6270
              0.6471     0.6670     0.6867     0.7063     0.7256
              0.7448     0.7638     0.7826     0.8014     0.8200
              0.8384     0.8568     0.8750     0.8931     0.9112
              0.9291     0.9470     0.9647     0.9824     1.0000
  Width:
              0.0330     0.0660     0.0990     0.1320     0.1650
              0.1980     0.2310     0.2640     0.2970     0.3300
              0.3630     0.3960     0.4290     0.4619     0.4912
              0.5057     0.5203     0.5348     0.5493     0.5639
              0.5784     0.5929     0.6075     0.6220     0.6366
              0.6511     0.6656     0.6802     0.6947     0.7092
              0.7238     0.7383     0.7529     0.7674     0.7819
              0.7965     0.8110     0.8255     0.8401     0.8546
              0.8692     0.8837     0.8982     0.9128     0.9273
              0.9418     0.9564     0.9709     0.9855     1.0000

  Transect OX_4
  Area:
              0.0004     0.0016     0.0036     0.0065     0.0101
```

```
              0.0146     0.0198     0.0259     0.0328     0.0405
              0.0490     0.0583     0.0684     0.0793     0.0910
              0.1036     0.1169     0.1311     0.1461     0.1619
              0.1784     0.1958     0.2141     0.2331     0.2529
              0.2735     0.2950     0.3172     0.3403     0.3642
              0.3889     0.4144     0.4407     0.4678     0.4957
              0.5244     0.5540     0.5843     0.6155     0.6474
              0.6802     0.7138     0.7482     0.7833     0.8188
              0.8545     0.8905     0.9267     0.9632     1.0000
  Hrad:
              0.0185     0.0369     0.0554     0.0739     0.0923
              0.1108     0.1292     0.1477     0.1662     0.1846
              0.2031     0.2216     0.2400     0.2585     0.2770
              0.2954     0.3139     0.3323     0.3508     0.3693
              0.3877     0.4062     0.4247     0.4431     0.4616
              0.4801     0.4985     0.5170     0.5355     0.5539
              0.5724     0.5908     0.6093     0.6278     0.6462
              0.6647     0.6832     0.7016     0.7201     0.7386
              0.7570     0.7755     0.7939     0.8187     0.8494
              0.8798     0.9101     0.9403     0.9702     1.0000
  Width:
              0.0219     0.0439     0.0658     0.0877     0.1097
              0.1316     0.1535     0.1755     0.1974     0.2193
              0.2413     0.2632     0.2851     0.3071     0.3290
              0.3509     0.3729     0.3948     0.4167     0.4387
              0.4606     0.4825     0.5045     0.5264     0.5483
              0.5703     0.5922     0.6141     0.6361     0.6580
              0.6799     0.7019     0.7238     0.7457     0.7677
              0.7896     0.8116     0.8335     0.8554     0.8774
              0.8993     0.9212     0.9432     0.9575     0.9646
              0.9717     0.9788     0.9858     0.9929     1.0000

  Transect OX_5
  Area:
              0.0004     0.0018     0.0040     0.0070     0.0110
              0.0158     0.0216     0.0282     0.0357     0.0440
              0.0533     0.0634     0.0744     0.0863     0.0991
              0.1127     0.1272     0.1426     0.1589     0.1761
              0.1942     0.2131     0.2329     0.2536     0.2752
              0.2976     0.3210     0.3452     0.3703     0.3962
              0.4231     0.4507     0.4788     0.5071     0.5357
```

```
              0.5646     0.5938     0.6232     0.6530     0.6831
              0.7135     0.7441     0.7751     0.8063     0.8379
              0.8697     0.9018     0.9343     0.9670     1.0000
  Hrad:
              0.0167     0.0333     0.0500     0.0666     0.0833
              0.0999     0.1166     0.1332     0.1499     0.1665
              0.1832     0.1998     0.2165     0.2332     0.2498
              0.2665     0.2831     0.2998     0.3164     0.3331
              0.3497     0.3664     0.3830     0.3997     0.4163
              0.4330     0.4497     0.4663     0.4830     0.4996
              0.5163     0.5387     0.5660     0.5931     0.6200
              0.6467     0.6731     0.6993     0.7254     0.7512
              0.7769     0.8023     0.8276     0.8527     0.8777
              0.9024     0.9271     0.9515     0.9758     1.0000
  Width:
              0.0266     0.0531     0.0797     0.1062     0.1328
              0.1593     0.1859     0.2124     0.2390     0.2655
              0.2921     0.3186     0.3452     0.3717     0.3983
              0.4249     0.4514     0.4780     0.5045     0.5311
              0.5576     0.5842     0.6107     0.6373     0.6638
              0.6904     0.7169     0.7435     0.7700     0.7966
              0.8231     0.8404     0.8492     0.8581     0.8670
              0.8758     0.8847     0.8936     0.9024     0.9113
              0.9202     0.9291     0.9379     0.9468     0.9557
              0.9645     0.9734     0.9823     0.9911     1.0000

  Transect OX_6
  Area:
              0.0005     0.0020     0.0044     0.0079     0.0123
              0.0178     0.0242     0.0316     0.0400     0.0493
              0.0597     0.0710     0.0834     0.0967     0.1110
              0.1263     0.1426     0.1598     0.1781     0.1973
              0.2176     0.2388     0.2610     0.2840     0.3073
              0.3310     0.3551     0.3794     0.4042     0.4292
              0.4546     0.4803     0.5063     0.5327     0.5594
              0.5864     0.6138     0.6415     0.6695     0.6979
              0.7266     0.7557     0.7850     0.8147     0.8448
              0.8752     0.9059     0.9369     0.9683     1.0000
  Hrad:
              0.0161     0.0323     0.0484     0.0646     0.0807
              0.0969     0.1130     0.1292     0.1453     0.1615
```

```
            0.1776     0.1938     0.2099     0.2261     0.2422
            0.2584     0.2745     0.2907     0.3068     0.3230
            0.3391     0.3553     0.3714     0.3951     0.4213
            0.4471     0.4727     0.4979     0.5229     0.5477
            0.5721     0.5964     0.6204     0.6442     0.6677
            0.6911     0.7142     0.7372     0.7600     0.7826
            0.8050     0.8272     0.8493     0.8713     0.8931
            0.9147     0.9363     0.9576     0.9789     1.0000
Width:
            0.0310     0.0619     0.0929     0.1238     0.1548
            0.1857     0.2167     0.2476     0.2786     0.3095
            0.3405     0.3714     0.4024     0.4333     0.4643
            0.4952     0.5262     0.5571     0.5881     0.6190
            0.6500     0.6809     0.7119     0.7279     0.7383
            0.7488     0.7593     0.7697     0.7802     0.7907
            0.8011     0.8116     0.8221     0.8325     0.8430
            0.8535     0.8639     0.8744     0.8849     0.8953
            0.9058     0.9163     0.9267     0.9372     0.9477
            0.9581     0.9686     0.9791     0.9895     1.0000

Transect ROW
Area:
            0.0004     0.0017     0.0038     0.0068     0.0106
            0.0153     0.0209     0.0272     0.0345     0.0426
            0.0515     0.0613     0.0719     0.0834     0.0958
            0.1090     0.1230     0.1379     0.1536     0.1694
            0.1852     0.2014     0.2200     0.2400     0.2607
            0.2820     0.3041     0.3268     0.3502     0.3743
            0.3990     0.4245     0.4506     0.4775     0.5050
            0.5332     0.5621     0.5917     0.6219     0.6529
            0.6845     0.7168     0.7498     0.7835     0.8179
            0.8529     0.8887     0.9251     0.9622     1.0000
Hrad:
            0.0183     0.0366     0.0549     0.0732     0.0915
            0.1098     0.1281     0.1464     0.1648     0.1831
            0.2014     0.2197     0.2380     0.2563     0.2746
            0.2929     0.3112     0.3295     0.3554     0.3914
            0.4272     0.4620     0.4901     0.5155     0.5403
            0.5643     0.5877     0.6104     0.6324     0.6539
            0.6748     0.6952     0.7150     0.7345     0.7534
            0.7720     0.7902     0.8080     0.8255     0.8426
```

```
            0.8595     0.8761     0.8924     0.9084     0.9242
            0.9398     0.9551     0.9703     0.9852     1.0000
Width:
            0.0223     0.0446     0.0670     0.0893     0.1116
            0.1339     0.1562     0.1786     0.2009     0.2232
            0.2455     0.2678     0.2902     0.3125     0.3348
            0.3571     0.3794     0.4018     0.4145     0.4146
            0.4146     0.4537     0.5148     0.5327     0.5507
            0.5687     0.5866     0.6046     0.6226     0.6406
            0.6585     0.6765     0.6945     0.7125     0.7304
            0.7484     0.7664     0.7843     0.8023     0.8203
            0.8383     0.8562     0.8742     0.8922     0.9101
            0.9281     0.9461     0.9641     0.9820     1.0000


****************
Analysis Options
****************
Flow Units ............... LPS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 07/19/2022 00:00:00
Ending Date .............. 07/20/2022 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:01:00
Dry Time Step ............ 00:01:00
Routing Time Step ........ 2.00 sec
Variable Time Step ....... YES
Maximum Trials ........... 8
Number of Threads ........ 8
Head Tolerance ........... 0.001500 m
```

```
  **************************        Volume         Depth
  Runoff Quantity Continuity     hectare-m            mm
  **************************     ---------       -------
  Total Precipitation ......         0.640        82.323
  Evaporation Loss .........         0.000         0.000
  Infiltration Loss ........         0.190        24.488
  Surface Runoff ...........         0.447        57.562
  Final Storage ............         0.003         0.345
  Continuity Error (%) .....        -0.087


  **************************        Volume        Volume
  Flow Routing Continuity        hectare-m      10^6 ltr
  **************************     ---------     ---------
  Dry Weather Inflow .......         0.000         0.000
  Wet Weather Inflow .......         0.447         4.473
  Groundwater Inflow .......         0.000         0.000
  RDII Inflow ..............         0.000         0.000
  External Inflow ..........         0.000         0.000
  External Outflow .........         0.424         4.243
  Flooding Loss ............         0.000         0.000
  Evaporation Loss .........         0.000         0.000
  Exfiltration Loss ........         0.000         0.000
  Initial Stored Volume ....         0.000         0.000
  Final Stored Volume ......         0.024         0.237
  Continuity Error (%) .....        -0.161


  *************************
  Highest Continuity Errors
  *************************
  Node J4 (37.08%)
  Node J5 (29.01%)
  Node J3 (23.06%)
  Node J2 (19.57%)


  ***************************
  Time-Step Critical Elements
```

```
  ***************************
  Link 140-142 (5.14%)


  ********************************
  Highest Flow Instability Indexes
  ********************************
  All links are stable.


  *********************************
  Most Frequent Nonconverging Nodes
  *********************************
  Node 220_(STM) (0.03%)
  Node MudC (0.03%)
  Node RVD (0.03%)
  Node WC_Drain (0.03%)
  Node MH106 (0.02%)


  *************************
  Routing Time Step Summary
  *************************
  Minimum Time Step           :     0.50 sec
  Average Time Step           :     1.97 sec
  Maximum Time Step           :     2.00 sec
  % of Time in Steady State   :     0.00
  Average Iterations per Step :     2.01
  % of Steps Not Converging   :     0.03
  Time Step Frequencies       :
      2.000 -  1.516 sec      :    97.06 %
      1.516 -  1.149 sec      :     2.88 %
      1.149 -  0.871 sec      :     0.03 %
      0.871 -  0.660 sec      :     0.02 %
      0.660 -  0.500 sec      :     0.02 %


  ***************************
  Subcatchment Runoff Summary
  ***************************
```

```
  ------------------------------------------------------------------------------------------------------------------
-------------
                            Total      Total      Total      Total     Imperv       Perv      Total       Total
Peak  Runoff
                           Precip      Runon       Evap      Infil     Runoff     Runoff     Runoff      Runoff
Runoff   Coeff
  Subcatchment                 mm         mm         mm         mm         mm         mm         mm    10^6 ltr
LPS
  ------------------------------------------------------------------------------------------------------------------
-------------
  A-01                      82.32       0.00       0.00      34.54      29.68      18.17      47.84        0.12
83.84   0.581
  B-01                      82.32       0.00       0.00      34.36      29.68      18.36      48.03        0.34
255.14   0.583
  C-01                      82.32       0.00       0.00      15.28      57.90       8.62      66.52        0.22
153.75   0.808
  C-02                      82.32       0.00       0.00      15.32      57.90       8.57      66.47        0.19
131.46   0.807
  C-03                      82.32       0.00       0.00      15.31      57.90       8.58      66.48        0.18
127.26   0.808
  C-04                      82.32       0.00       0.00      15.31      57.90       8.58      66.48        0.21
144.36   0.808
  C-05                      82.32       0.00       0.00      15.31      57.90       8.59      66.49        0.12
80.94   0.808
  C-06                      82.32       0.00       0.00      15.43      57.89       8.46      66.35        0.21
144.15   0.806
  C-07                      82.32       0.00       0.00      36.22      29.66      16.47      46.14        0.32
181.67   0.560
  C-08                      82.32       0.00       0.00      15.30      57.90       8.59      66.49        0.11
77.75   0.808
  C-09                      82.32       0.00       0.00      15.30      57.90       8.60      66.50        0.14
99.55   0.808
  C-10                      82.32       0.00       0.00      15.30      57.90       8.59      66.49        0.15
102.20   0.808
  C-11                      82.32       0.00       0.00      34.24      29.68      18.47      48.15        0.29
221.84   0.585
  C-12                      82.32       0.00       0.00      15.27      57.90       8.63      66.53        0.22
152.26   0.808
  C-13                      82.32       0.00       0.00      15.31      57.90       8.58      66.48        0.17
114.82   0.808
  C-14                      82.32       0.00       0.00      33.98      29.68      18.74      48.41        0.27
221.70   0.588
  C-15                      82.32       0.00       0.00      15.30      57.90       8.60      66.50        0.22
155.52   0.808
  C-16                      82.32       0.00       0.00      15.29      57.90       8.60      66.50        0.23
161.95   0.808
  C-17                      82.32       0.00       0.00      15.30      57.90       8.59      66.49        0.08
53.82   0.808
  C-18                      82.32       0.00       0.00      15.29      57.90       8.60      66.50        0.12
85.41   0.808
  C-19                      82.32       0.00       0.00      33.96      29.68      18.76      48.43        0.19
158.69   0.588
  C-20                      82.32       0.00       0.00      34.40      29.11      18.31      47.43        0.06
42.59   0.576
  C-21                      82.32       0.00       0.00      15.26      57.90       8.63      66.54        0.11
78.14   0.808
  C-22                      82.32       0.00       0.00      15.26      57.90       8.63      66.54        0.14
98.59   0.808
  D-01                      82.32       0.00       0.00      54.51       0.00      27.84      27.84        0.05
40.15   0.338


  ******************
  Node Depth Summary
  ******************

  ---------------------------------------------------------------------------------
                                 Average  Maximum  Maximum  Time of Max    Reported
                                   Depth    Depth      HGL   Occurrence   Max Depth
  Node                 Type       Meters   Meters   Meters  days hr:min      Meters
  ---------------------------------------------------------------------------------
  CB01                 JUNCTION     0.00     0.07    89.64     0  02:10        0.07
  CB02                 JUNCTION     0.00     0.09    89.22     0  02:10        0.09
  CB03                 JUNCTION     0.00     0.09    89.04     0  02:10        0.09
  CB04                 JUNCTION     0.00     0.10    88.50     0  02:10        0.10
  CB05                 JUNCTION     0.00     0.07    88.17     0  02:10        0.07
  CB06                 JUNCTION     0.00     0.11    88.28     0  02:10        0.11
  CB07                 JUNCTION     0.01     0.16    87.91     0  02:10        0.16
  CB08                 JUNCTION     0.00     0.05    89.78     0  02:10        0.05
  CB09                 JUNCTION     0.00     0.07    88.64     0  02:10        0.07
  CB10                 JUNCTION     0.00     0.10    87.89     0  02:10        0.10
  CB11                 JUNCTION     0.01     0.17    87.81     0  02:10        0.17
  CB12                 JUNCTION     0.00     0.05    91.62     0  02:10        0.05
  CB13                 JUNCTION     0.00     0.07    88.41     0  02:10        0.07
  CB14                 JUNCTION     0.00     0.05    93.05     0  02:10        0.05
  CB15                 JUNCTION     0.01     0.07    89.22     0  02:10        0.07
```

```
  CB16                 JUNCTION       0.06     1.75    87.71     0  02:14        1.75
  CB17                 JUNCTION       0.00     0.14    87.98     0  02:11        0.14
  J1                   JUNCTION       0.01     0.12    81.79     0  02:10        0.12
  J10                  JUNCTION       0.01     0.22    87.60     0  02:14        0.22
  J2                   JUNCTION       0.36     0.43    81.30     0  03:39        0.43
  J3                   JUNCTION       0.44     0.52    81.30     0  03:38        0.52
  J4                   JUNCTION       0.70     0.80    81.30     0  03:37        0.80
  J5                   JUNCTION       0.44     0.52    81.30     0  03:37        0.52
  J6                   JUNCTION       0.03     0.07    81.30     0  03:37        0.07
  J7                   JUNCTION       0.01     0.15    88.10     0  02:10        0.14
  J8                   JUNCTION       0.01     0.17    87.89     0  02:10        0.17
  J9                   JUNCTION       0.01     0.17    87.75     0  02:11        0.17
  MH100                JUNCTION       0.01     0.16    88.08     0  02:10        0.16
  MH102                JUNCTION       0.01     0.18    86.99     0  02:10        0.18
  MH104                JUNCTION       0.02     0.70    86.87     0  02:06        0.54
  MH106                JUNCTION       0.02     0.55    86.18     0  02:06        0.54
  MH108                JUNCTION       0.03     0.57    85.84     0  02:10        0.57
  MH110                JUNCTION       0.05     0.90    85.62     0  02:10        0.90
  MH112                JUNCTION       0.00     0.00    87.53     0  00:00        0.00
  MH114                JUNCTION       0.00     0.06    87.09     0  02:04        0.06
  MH116                JUNCTION       0.01     0.18    87.09     0  02:05        0.18
  MH118                JUNCTION       0.02     0.26    86.73     0  02:05        0.26
  MH120                JUNCTION       0.02     0.27    86.36     0  02:09        0.27
  MH122                JUNCTION       0.03     0.48    86.11     0  02:12        0.48
  MH124                JUNCTION       0.04     0.78    85.97     0  02:11        0.78
  MH126                JUNCTION       0.02     0.58    85.99     0  02:11        0.58
  MH128                JUNCTION       0.02     0.40    86.00     0  02:11        0.40
  MH130                JUNCTION       0.05     0.80    85.84     0  02:11        0.79
  MH132                JUNCTION       0.06     0.86    85.35     0  02:10        0.86
  MH134                JUNCTION       0.05     0.75    85.19     0  02:10        0.75
  MH136                JUNCTION       0.01     0.19    86.83     0  02:05        0.19
  MH138                JUNCTION       0.01     0.11    90.79     0  02:10        0.11
  MH140                JUNCTION       0.05     0.80    84.25     0  02:11        0.80
  MH142                JUNCTION       0.05     0.70    83.62     0  02:11        0.70
  MH144                JUNCTION       2.36     2.64    84.86     0  02:14        2.64
  MH144_A              JUNCTION       0.36     1.12    83.34     0  02:11        1.12
  MH146                JUNCTION       0.00     0.00    87.09     0  00:00        0.00
  MH148                JUNCTION       0.00     0.00    90.92     0  00:00        0.00
  MH150                JUNCTION       0.01     0.37    83.37     0  02:13        0.37
  RYCB01               JUNCTION       0.07     1.94    88.99     0  02:10        1.94
  RYCB02               JUNCTION       0.06     1.64    88.39     0  02:10        1.64
```

```
  RYCB03               JUNCTION       0.06     1.61    88.31     0  02:10        1.61
  220_(STM)            OUTFALL        0.00     0.00    80.00     0  00:00        0.00
  MudC                 OUTFALL        0.02     0.06    81.03     0  03:37        0.06
  RVD                  OUTFALL        0.00     0.00     0.00     0  00:00        0.00
  WC_Drain             OUTFALL        0.00     0.00     0.00     0  00:00        0.00


  *******************
  Node Inflow Summary
  *******************

  -------------------------------------------------------------------------------------------------
                                  Maximum  Maximum                  Lateral       Total        Flow
                                  Lateral    Total  Time of Max      Inflow      Inflow     Balance
                                   Inflow   Inflow  Occurrence       Volume      Volume       Error
  Node                 Type           LPS      LPS  days hr:min    10^6 ltr    10^6 ltr     Percent
  -------------------------------------------------------------------------------------------------
  CB01                 JUNCTION    153.75   153.75     0  02:10       0.221       0.221      -0.131
  CB02                 JUNCTION    131.46   224.84     0  02:10       0.191       0.267       0.076
  CB03                 JUNCTION    127.26   280.86     0  02:10       0.184       0.288      -0.038
  CB04                 JUNCTION    144.36   352.77     0  02:10       0.209       0.344      -0.217
  CB05                 JUNCTION     80.94   171.51     0  02:10       0.117       0.174      -0.118
  CB06                 JUNCTION    144.15   144.15     0  02:10       0.214       0.214      -0.018
  CB07                 JUNCTION     99.55   560.90     0  02:10       0.144       0.402      -0.103
  CB08                 JUNCTION    114.82   114.82     0  02:10       0.166       0.166      -0.049
  CB09                 JUNCTION    152.26   216.24     0  02:10       0.218       0.281      -0.095
  CB10                 JUNCTION    102.20   229.53     0  02:10       0.148       0.263      -0.090
  CB11                 JUNCTION     53.82   760.13     0  02:10      0.0778       0.602      -0.028
  CB12                 JUNCTION    161.95   161.95     0  02:10       0.233       0.233      -0.039
  CB13                 JUNCTION    155.52   252.81     0  02:10       0.224       0.333      -0.140
  CB14                 JUNCTION     98.59    98.59     0  02:10       0.141       0.141      -0.146
  CB15                 JUNCTION     78.14   176.13     0  02:10       0.112       0.253      -0.592
  CB16                 JUNCTION     85.41  1100.38     0  02:10       0.123       0.969       0.014
  CB17                 JUNCTION     77.75   420.93     0  02:10       0.112       0.343       0.295
  J1                   JUNCTION    181.67   181.67     0  02:10       0.316       0.316      -0.773
  J10                  JUNCTION      0.00   272.28     0  02:14           0       0.185       0.002
  J2                   JUNCTION      0.00   178.65     0  02:10           0       0.318      24.334
  J3                   JUNCTION      0.00   161.90     0  02:10           0       0.256      29.979
  J4                   JUNCTION      0.00   101.44     0  02:11           0       0.197      58.943
  J5                   JUNCTION      0.00    39.67     0  02:11           0       0.124      40.872
  J6                   JUNCTION      0.00     5.16     0  03:34           0       0.088       0.423
```

```
J7                   JUNCTION      0.00   368.17     0  02:10           0       0.231       0.197
J8                   JUNCTION      0.00   610.98     0  02:10           0       0.445       0.183
J9                   JUNCTION      0.00   812.58     0  02:10           0       0.535       0.030
MH100                JUNCTION      0.00    49.70     0  02:07           0       0.104       0.002
MH102                JUNCTION      0.00    49.70     0  02:10           0       0.104      -0.109
MH104                JUNCTION      0.00   134.19     0  02:10           0        0.27       0.181
MH106                JUNCTION      0.00   224.87     0  02:10           0       0.454      -0.040
MH108                JUNCTION      0.00   224.86     0  02:10           0       0.454       0.116
MH110                JUNCTION      0.00   826.01     0  02:11           0        1.96       0.056
MH112                JUNCTION      0.00     0.00     0  00:00           0           0       0.000 ltr
MH114                JUNCTION      0.00     2.38     0  02:02           0    0.000277       0.506
MH116                JUNCTION      0.00    58.97     0  02:03           0       0.145       0.109
MH118                JUNCTION      0.00   124.26     0  02:05           0       0.308      -0.033
MH120                JUNCTION      0.00   189.85     0  02:05           0        0.46       0.149
MH122                JUNCTION      0.00   282.41     0  02:09           0       0.647      -0.083
MH124                JUNCTION      0.00   386.96     0  02:13           0       0.906       0.053
MH126                JUNCTION      0.00    65.57     0  02:16           0       0.157      -0.278
MH128                JUNCTION      0.00    49.70     0  02:01           0       0.158       0.202
MH130                JUNCTION      0.00   512.49     0  02:13           0        1.28      -0.113
MH132                JUNCTION      0.00   918.96     0  02:12           0        2.19      -0.052
MH134                JUNCTION      0.00  1060.50     0  02:10           0        2.49      -0.023
MH136                JUNCTION      0.00   147.11     0  02:10           0       0.294       0.332
MH138                JUNCTION      0.00    63.68     0  02:10           0       0.125      -0.181
MH140                JUNCTION      0.00  1317.82     0  02:10           0         3.2       0.000
MH142                JUNCTION      0.00  1317.83     0  02:11           0         3.2      -0.000
MH144                JUNCTION      0.00   403.00     0  02:14           0        0.92       0.304
MH144_A              JUNCTION      0.00  1708.14     0  02:11           0        3.65       0.007
MH146                JUNCTION      0.00     0.00     0  00:00           0           0       0.000 ltr
MH148                JUNCTION      0.00     0.00     0  00:00           0           0       0.000 ltr
MH150                JUNCTION     42.59   292.55     0  02:14      0.0569       0.242       0.010
RYCB01               JUNCTION    221.84   221.84     0  02:10        0.29        0.29      -0.002
RYCB02               JUNCTION    221.70   221.70     0  02:10       0.273       0.273      -0.059
RYCB03               JUNCTION    158.69   158.69     0  02:10       0.194       0.194       0.500
220_(STM)            OUTFALL       0.00  1710.00     0  02:12           0        3.65       0.000
MudC                 OUTFALL       0.00     5.15     0  03:37           0      0.0876       0.000
RVD                  OUTFALL      40.15    40.15     0  02:10      0.0504      0.0504       0.000
WC_Drain             OUTFALL     338.98   338.98     0  02:10       0.459       0.459       0.000


**********************
Node Surcharge Summary
```

```
**********************

Surcharging occurs when water rises above the top of the highest conduit.
---------------------------------------------------------------------
                                             Max. Height   Min. Depth
                                 Hours       Above Crown    Below Rim
Node                 Type      Surcharged         Meters       Meters
---------------------------------------------------------------------
CB16                 JUNCTION        0.03          0.004        0.000
MH104                JUNCTION        0.08          0.391        1.754
MH106                JUNCTION        0.10          0.151        1.954
MH108                JUNCTION        0.12          0.117        2.006
MH110                JUNCTION        0.10          0.060        2.202
MH124                JUNCTION        0.14          0.141        2.209
MH126                JUNCTION        0.11          0.089        2.214
MH130                JUNCTION        0.13          0.111        2.113


*********************
Node Flooding Summary
*********************

No nodes were flooded.


***********************
Outfall Loading Summary
***********************

-----------------------------------------------------------
                       Flow       Avg       Max       Total
                       Freq      Flow      Flow      Volume
Outfall Node           Pcnt       LPS       LPS    10^6 ltr
-----------------------------------------------------------
220_(STM)             44.33    129.48   1710.00       3.646
MudC                  88.21      1.13      5.15       0.088
RVD                    7.00     12.27     40.15       0.050
WC_Drain              28.50     25.88    338.98       0.459
-----------------------------------------------------------
System                42.01    168.77   1956.24       4.243
```

```
********************
Link Flow Summary
********************

---------------------------------------------------------------------------
                              Maximum  Time of Max   Maximum    Max/    Max/
                               |Flow|   Occurrence   |Veloc|    Full    Full
Link                Type          LPS  days hr:min     m/sec    Flow   Depth
---------------------------------------------------------------------------
100-102             CONDUIT     49.70     0  02:10      1.51    0.70    0.62
102-104             CONDUIT     50.88     0  02:11      1.29    0.72    0.85
104-106             CONDUIT    134.17     0  02:10      1.91    1.08    1.00
106-108             CONDUIT    224.86     0  02:10      2.09    1.00    1.00
108-110             CONDUIT    224.83     0  02:10      1.56    0.76    1.00
110-132             CONDUIT    828.26     0  02:12      1.51    1.08    0.99
112-114             CONDUIT      0.00     0  00:00      0.00    0.00    0.05
114-116             CONDUIT      3.39     0  02:13      0.19    0.04    0.35
116-118             CONDUIT     58.66     0  02:05      1.29    0.67    0.60
118-120             CONDUIT    124.25     0  02:05      1.53    0.78    0.67
120-122             CONDUIT    191.71     0  02:09      1.72    0.49    0.63
122-124             CONDUIT    285.35     0  02:13      1.35    0.51    0.89
124-130             CONDUIT    387.14     0  02:13      1.32    1.17    1.00
126-124             CONDUIT     91.55     0  02:15      0.65    0.62    1.00
128-126             CONDUIT     65.57     0  02:16      0.88    0.44    0.94
130-110             CONDUIT    513.03     0  02:13      1.47    1.14    1.00
132-134             CONDUIT    919.64     0  02:12      1.74    1.16    0.94
134-140             CONDUIT   1059.24     0  02:10      2.38    0.71    0.76
136-134             CONDUIT    147.68     0  02:10      2.50    0.47    0.59
138-136             CONDUIT     63.57     0  02:10      2.56    0.30    0.38
140-142             CONDUIT   1317.83     0  02:11      2.51    0.32    0.57
142-144A            CONDUIT   1317.89     0  02:11      2.40    0.54    0.59
144-140             CONDUIT    260.28     0  02:14      2.66    0.11    0.28
144A-220            CONDUIT   1710.00     0  02:12      2.21    1.35    0.63
146-144             CONDUIT      0.00     0  00:00      0.00    0.00    0.00
148-146             CONDUIT      0.00     0  00:00      0.00    0.00    0.00
225-144A            CONDUIT    293.38     0  02:14      0.98    0.16    0.49
C1                  CHANNEL     90.82     0  02:10      0.33    0.03    0.26
C1_1                CHANNEL    178.65     0  02:10      0.34    0.00    0.08
C1_2                CHANNEL    101.44     0  02:11      0.30    0.00    0.17
C1_3                CHANNEL    161.90     0  02:10      0.34    0.00    0.11
```

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C1_4                CHANNEL     39.67     0  02:11      0.05    0.00    0.39
C1_5                CHANNEL      5.16     0  03:34      0.01    0.00    0.15
C1_7                CHANNEL      5.15     0  03:37      0.36    0.00    0.03
C10                 CHANNEL    591.82     0  02:11      0.65    0.15    0.48
C11                 CHANNEL    659.75     0  02:10      0.71    0.17    0.49
C12                 CHANNEL    124.40     0  02:09      0.25    0.03    0.38
C13                 CHANNEL    127.55     0  02:10      0.61    0.01    0.24
C14                 CHANNEL     64.04     0  02:10      0.53    0.01    0.17
C15                 CHANNEL     97.34     0  02:10      0.83    0.01    0.17
C16                 CHANNEL    166.93     0  02:10      0.77    0.01    0.33
C17                 CHANNEL     98.03     0  02:10      0.88    0.01    0.18
C18                 CHANNEL    172.66     0  02:10      0.90    0.01    0.58
C19                 CHANNEL    780.33     0  02:11      0.62    0.16    0.72
C21                 CONDUIT    145.72     0  02:10      0.93    0.53    0.76
C22                 CONDUIT    158.62     0  02:10      1.29    0.48    0.68
C23                 CONDUIT    113.58     0  02:10      0.85    0.38    0.82
C24_1               CONDUIT    272.28     0  02:14      1.34    1.63    0.87
C24_2               CONDUIT    272.00     0  02:14      1.85    0.44    0.74
C3                  CHANNEL     93.47     0  02:10      0.43    0.01    0.23
C4                  CHANNEL    154.24     0  02:10      0.57    0.03    0.26
C5                  CHANNEL    209.23     0  02:10      0.70    0.03    0.27
C6                  CHANNEL    250.38     0  02:10      0.51    0.04    0.35
C7                  CHANNEL    117.88     0  02:10      0.30    0.02    0.31
C8_1                CHANNEL    358.87     0  02:10      0.54    0.10    0.41
C8_2                CHANNEL    343.50     0  02:11      0.47    0.09    0.44
C9                  CHANNEL    500.23     0  02:10      0.58    0.14    0.47
OL16                ORIFICE    180.44     0  02:14                      1.00
OR1                 ORIFICE    180.44     0  02:14                      1.00
OR10                ORIFICE     43.26     0  02:10                      1.00
OR8                 ORIFICE     68.08     0  02:10                      1.00
OR9                 ORIFICE     62.24     0  02:10                      1.00
W1                  WEIR       142.73     0  02:14                      0.08
OL1                 DUMMY       49.70     0  02:01
OL10                DUMMY       65.60     0  02:02
OL11                DUMMY       65.60     0  02:02
OL12                DUMMY       63.68     0  02:10
OL13                DUMMY       83.54     0  02:10
OL14                DUMMY        0.00     0  00:00
OL15                DUMMY        0.00     0  00:00
OL17                DUMMY       58.50     0  02:03
OL2                 DUMMY       49.70     0  02:03
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  OL3                  DUMMY        90.70      0  02:05
  OL4                  DUMMY        49.70      0  02:07
  OL5                  DUMMY        90.70      0  02:03
  OL6                  DUMMY        90.70      0  02:05
  OL7                  DUMMY        85.14      0  02:10
  OL8                  DUMMY        49.70      0  02:07
  OL9                  DUMMY        58.50      0  02:02


  ***************************
  Flow Classification Summary
  ***************************

  -------------------------------------------------------------------------------------
                      Adjusted    ---------- Fraction of Time in Flow Class ----------
                       /Actual         Up    Down  Sub   Sup   Up    Down  Norm  Inlet
  Conduit               Length    Dry  Dry   Dry   Crit  Crit  Crit  Crit  Ltd   Ctrl
  -------------------------------------------------------------------------------------
  100-102                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  102-104                 1.00   0.01  0.00  0.00  0.01  0.01  0.00  0.98  0.01  0.00
  104-106                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  106-108                 1.00   0.01  0.00  0.00  0.01  0.00  0.00  0.98  0.00  0.00
  108-110                 1.00   0.01  0.00  0.00  0.01  0.00  0.00  0.98  0.00  0.00
  110-132                 1.00   0.01  0.00  0.00  0.10  0.00  0.00  0.89  0.00  0.00
  112-114                 1.00   0.99  0.01  0.00  0.00  0.00  0.00  0.00  0.00  0.00
  114-116                 1.00   0.81  0.15  0.00  0.04  0.00  0.00  0.00  0.90  0.00
  116-118                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  118-120                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  120-122                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  122-124                 1.00   0.01  0.00  0.00  0.06  0.12  0.00  0.82  0.04  0.00
  124-130                 1.00   0.01  0.00  0.00  0.02  0.00  0.00  0.97  0.00  0.00
  126-124                 1.00   0.01  0.00  0.00  0.03  0.00  0.00  0.97  0.00  0.00
  128-126                 1.00   0.01  0.00  0.00  0.03  0.00  0.00  0.96  0.01  0.00
  130-110                 1.00   0.01  0.00  0.00  0.02  0.00  0.00  0.98  0.00  0.00
  132-134                 1.00   0.01  0.00  0.00  0.02  0.00  0.00  0.98  0.00  0.00
  134-140                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  136-134                 1.00   0.01  0.00  0.00  0.00  0.01  0.00  0.98  0.00  0.00
  138-136                 1.00   0.01  0.00  0.00  0.00  0.01  0.00  0.99  0.00  0.00
  140-142                 1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  142-144A                1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  144-140                 1.00   0.03  0.00  0.00  0.00  0.00  0.00  0.97  0.00  0.00
  144A-220                1.00   0.01  0.00  0.00  0.00  0.00  0.00  0.99  0.00  0.00
  146-144                 1.00   1.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
  148-146                 1.00   1.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
  225-144A                1.00   0.03  0.00  0.00  0.03  0.00  0.00  0.94  0.01  0.00
  C1                      1.00   0.54  0.02  0.00  0.43  0.00  0.00  0.00  0.00  0.00
  C1_1                    1.00   0.01  0.00  0.00  0.98  0.02  0.00  0.00  0.97  0.00
  C1_2                    1.00   0.02  0.00  0.00  0.96  0.02  0.00  0.00  0.04  0.00
  C1_3                    1.00   0.01  0.00  0.00  0.97  0.02  0.00  0.00  0.00  0.00
  C1_4                    1.00   0.03  0.04  0.00  0.92  0.00  0.00  0.00  0.00  0.00
  C1_5                    1.00   0.08  0.03  0.00  0.90  0.00  0.00  0.00  0.00  0.00
  C1_7                    1.00   0.10  0.00  0.00  0.61  0.28  0.00  0.00  0.01  0.00
  C10                     1.00   0.01  0.00  0.00  0.55  0.44  0.00  0.00  0.01  0.00
  C11                     1.00   0.01  0.47  0.00  0.52  0.01  0.00  0.00  0.96  0.00
  C12                     1.00   0.01  0.62  0.00  0.37  0.00  0.00  0.00  0.98  0.00
  C13                     1.00   0.58  0.04  0.00  0.16  0.23  0.00  0.00  0.06  0.00
  C14                     1.00   0.57  0.05  0.00  0.37  0.01  0.00  0.00  0.99  0.00
  C15                     1.00   0.55  0.05  0.00  0.13  0.26  0.00  0.00  0.99  0.00
  C16                     1.00   0.01  0.60  0.00  0.36  0.03  0.00  0.00  0.97  0.00
  C17                     1.00   0.01  0.00  0.00  0.51  0.49  0.00  0.00  0.98  0.00
  C18                     1.00   0.01  0.00  0.00  0.02  0.00  0.00  0.98  0.02  0.00
  C19                     1.00   0.01  0.00  0.00  0.02  0.00  0.00  0.97  0.02  0.00
  C21                     1.00   0.98  0.00  0.00  0.00  0.00  0.00  0.02  0.00  0.00
  C22                     1.00   0.47  0.51  0.00  0.01  0.01  0.00  0.00  0.90  0.00
  C23                     1.00   0.98  0.00  0.00  0.01  0.00  0.00  0.00  0.91  0.00
  C24_1                   1.00   0.09  0.00  0.00  0.01  0.01  0.00  0.90  0.00  0.00
  C24_2                   1.00   0.09  0.00  0.00  0.00  0.00  0.00  0.91  0.00  0.00
  C3                      1.00   0.56  0.06  0.00  0.37  0.01  0.00  0.00  0.96  0.00
  C4                      1.00   0.56  0.05  0.00  0.38  0.01  0.00  0.00  0.01  0.00
  C5                      1.00   0.56  0.06  0.00  0.37  0.01  0.00  0.00  0.99  0.00
  C6                      1.00   0.01  0.60  0.00  0.37  0.02  0.00  0.00  0.96  0.00
  C7                      1.00   0.01  0.64  0.00  0.34  0.02  0.00  0.00  0.97  0.00
  C8_1                    1.00   0.01  0.01  0.00  0.50  0.49  0.00  0.00  0.02  0.00
  C8_2                    1.00   0.01  0.42  0.00  0.57  0.00  0.00  0.00  0.99  0.00
  C9                      1.00   0.01  0.00  0.00  0.99  0.01  0.00  0.00  0.97  0.00


  *************************
  Conduit Surcharge Summary
  *************************

  ----------------------------------------------------------------------------
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                                                         Hours        Hours
                       --------- Hours Full --------   Above Full   Capacity
Conduit                Both Ends  Upstream  Dnstream   Normal Flow   Limited
----------------------------------------------------------------------------
102-104                     0.01      0.01      0.08      0.01         0.01
104-106                     0.08      0.08      0.10      0.11         0.08
106-108                     0.11      0.11      0.13      0.01         0.09
108-110                     0.10      0.12      0.11      0.01         0.09
110-132                     0.01      0.10      0.01      0.16         0.01
122-124                     0.01      0.01      0.14      0.01         0.01
124-130                     0.13      0.15      0.13      0.20         0.13
126-124                     0.13      0.13      0.15      0.01         0.01
128-126                     0.01      0.01      0.11      0.01         0.01
130-110                     0.11      0.13      0.11      0.20         0.11
132-134                     0.01      0.07      0.01      0.20         0.01
144A-220                    0.01      0.01      0.01      0.26         0.01
C18                         0.01      0.01      0.03      0.01         0.01
C19                         0.01      0.01      0.03      0.01         0.01
C23                         0.01      0.01      0.14      0.01         0.01
C24_1                       0.01      0.01      0.14      0.16         0.01


Analysis begun on:  Thu Mar 28 09:40:06 2024
Analysis ended on:  Thu Mar 28 09:40:08 2024
Total elapsed time: 00:00:02
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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] = **(6.12 hectares) x (0.67) x (2.775)** / **(14.3 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 $l/s/m^2$.

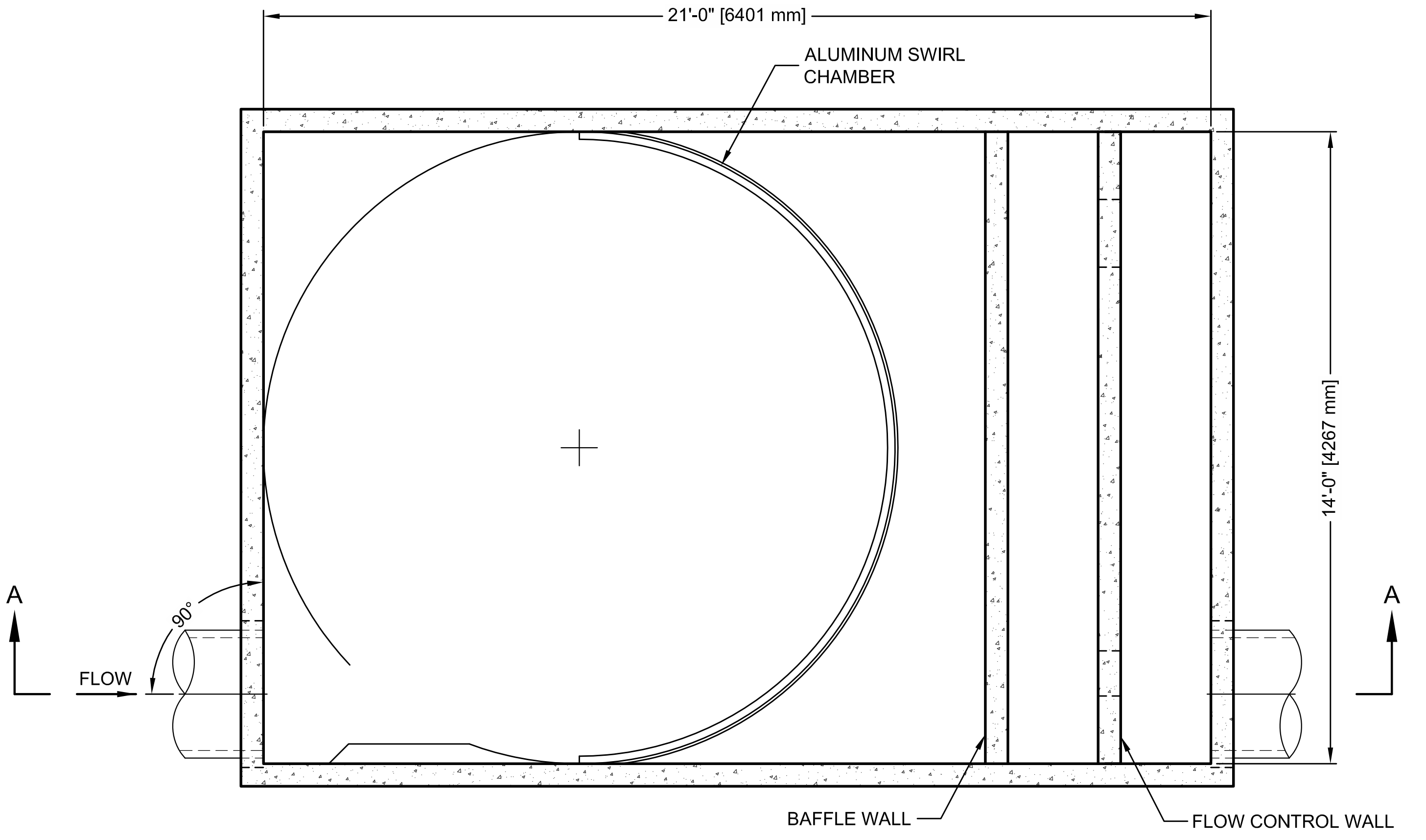
3 - Based on 42 years of hourly rainfall data from Canadian Station 6105976, Ottawa CDA, ON

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

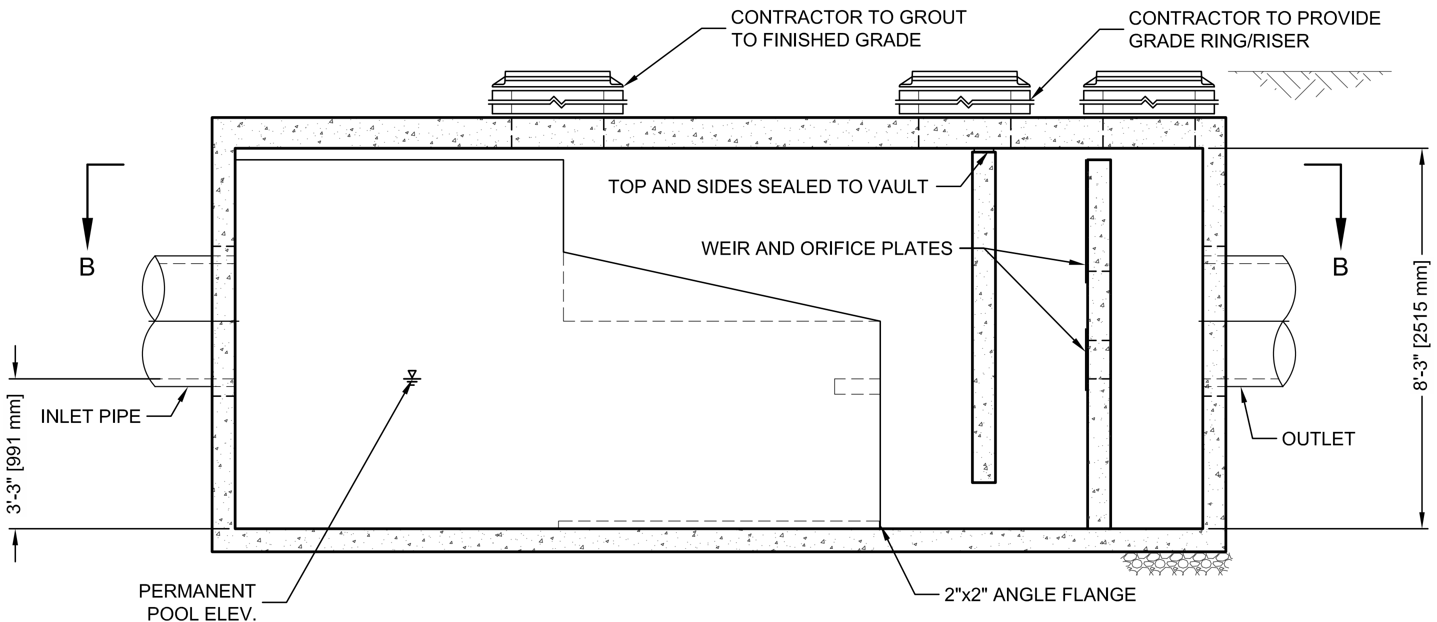
5- Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.

Calculated by: JAK 7/26

Checked by:



SECTION B-B



SECTION A-A

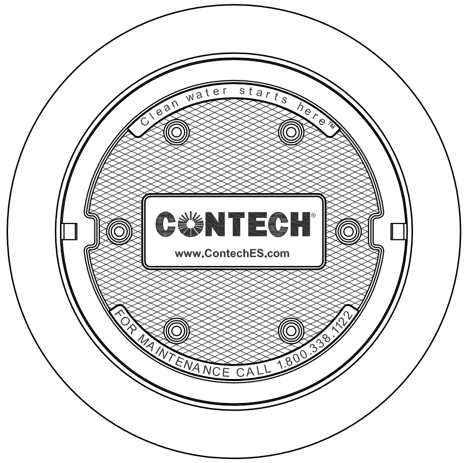
Vortechs®

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

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



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

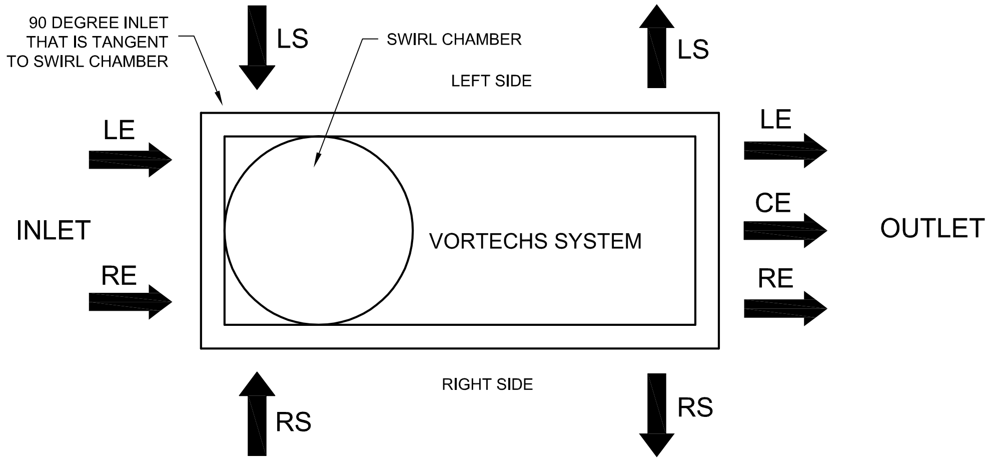
A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE VORTECHS STRUCTURE (LIFTING CLUTCHES PROVIDED).
C. 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.

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

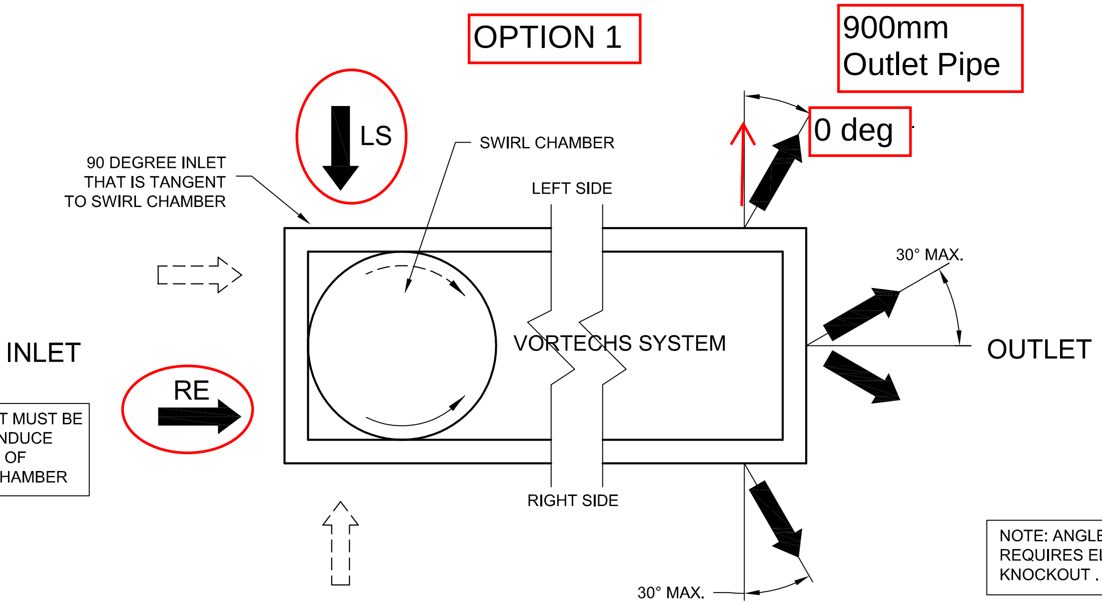
VORTECHS PC1421
STANDARD DETAIL

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



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

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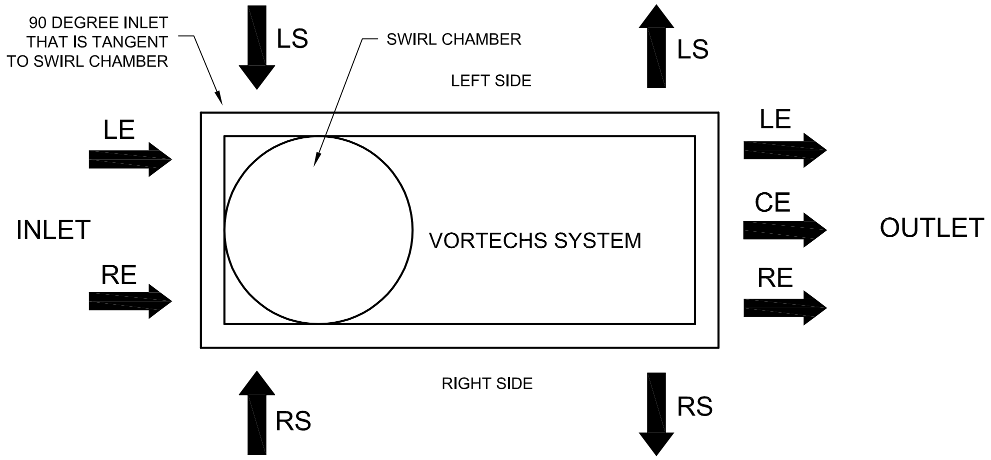
CONTECH® STORMWATER SOLUTIONS INC.
contechstormwater.com

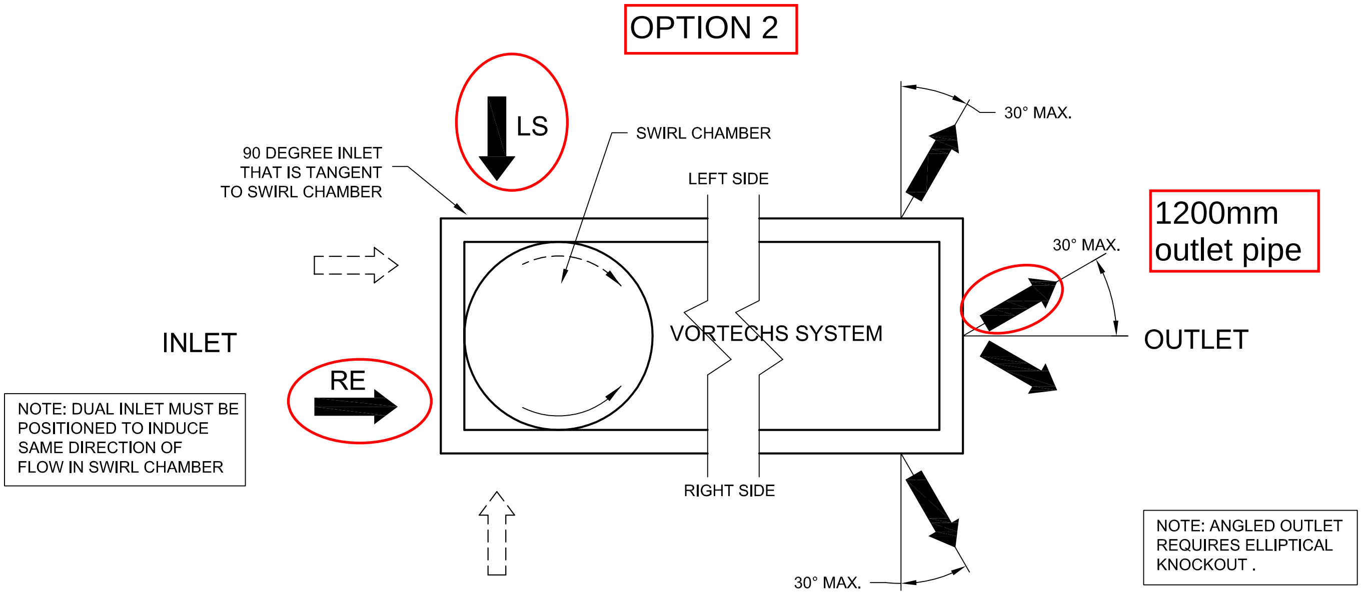
TYPICAL VORTECHS® SYSTEM ORIENTATIONS

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

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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

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.

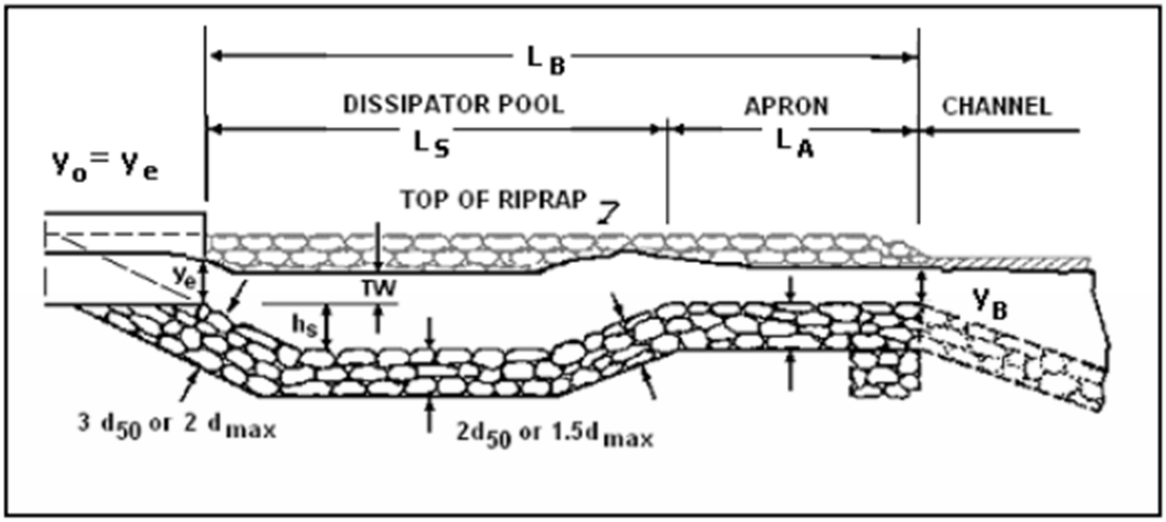


Figure 10.1. Profile of Riprap Basin

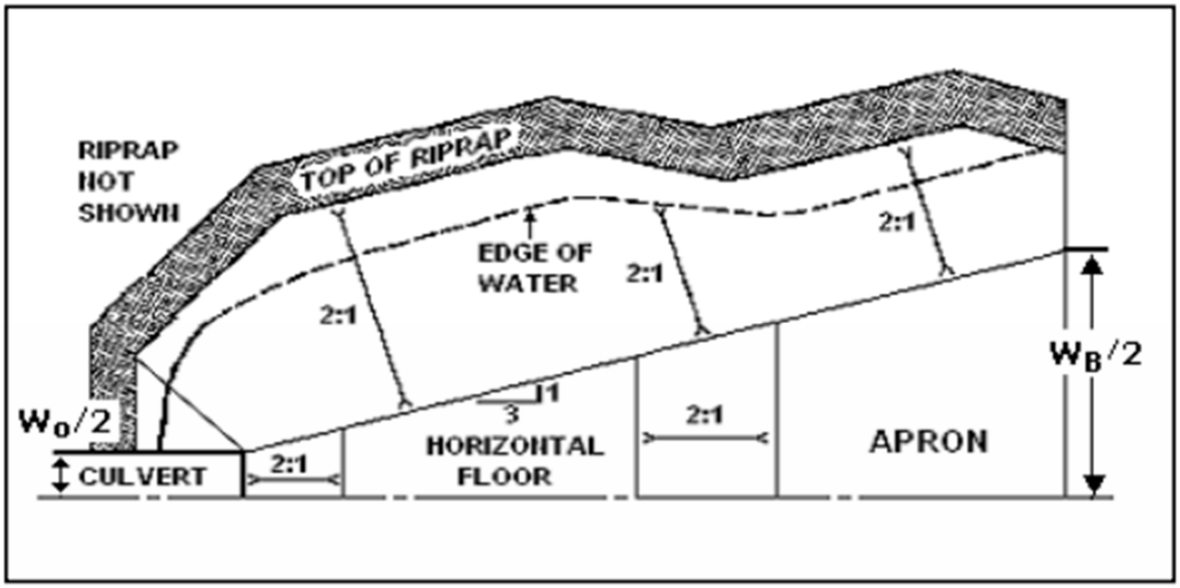


Figure 10.2. Half Plan of Riprap Basin

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 (2.05cms)

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

$$V = \frac{2.13cms}{2.53m^3}$$

$$V = 0.84m/s$$

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.

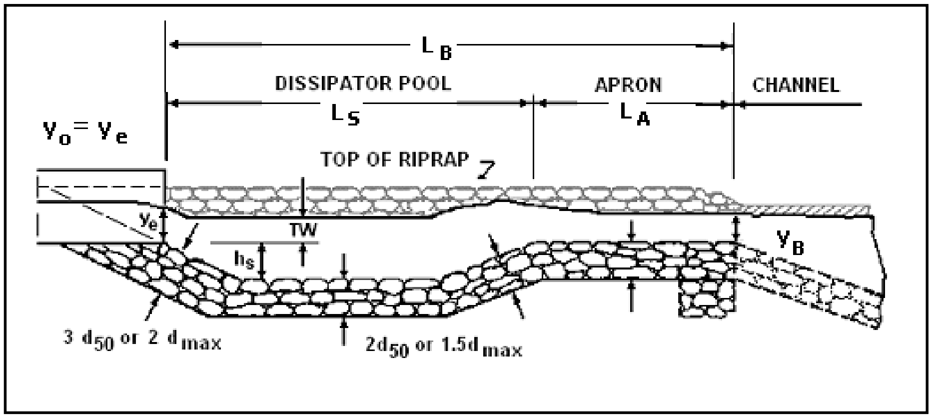


Figure 10.1. Profile of Riprap Basin

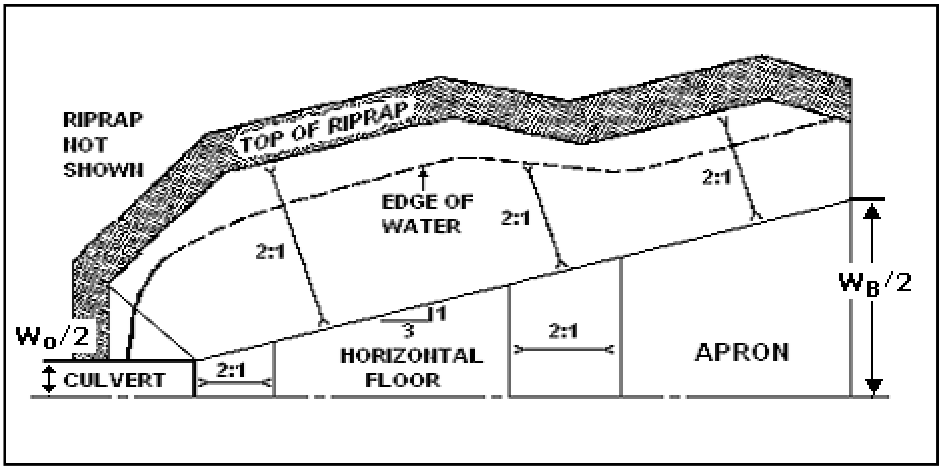


Figure 10.2. Half Plan of Riprap Basin

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.

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

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:

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

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.

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

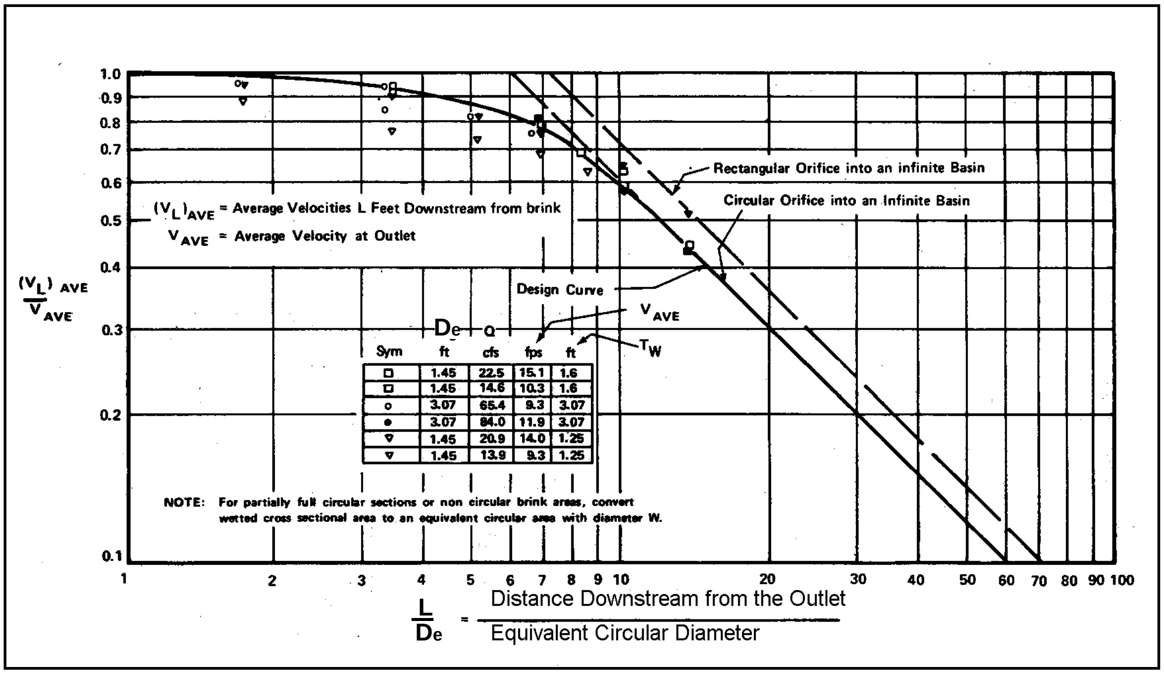
Use of the envelope design relationship (Equations 10.1 and 10.2) is recommended when the consequences of failure at or near the design flow are severe. Use of the best fit design relationship (Equations 10.1 and 10.3) is recommended when basin failure may easily be addressed as part of routine maintenance. 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.



Sym	De ft	Q cfs	V_{AVE} fps	T_W ft
□	1.45	22.5	15.1	1.6
◘	1.45	14.6	10.3	1.6
○	3.07	65.4	9.3	3.07
●	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

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:

$Q = 800 \text{ ft}^3/\text{s}$

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 $TW/y_o \le 0.75$. 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 $V_L \le V_{allow}$. 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)

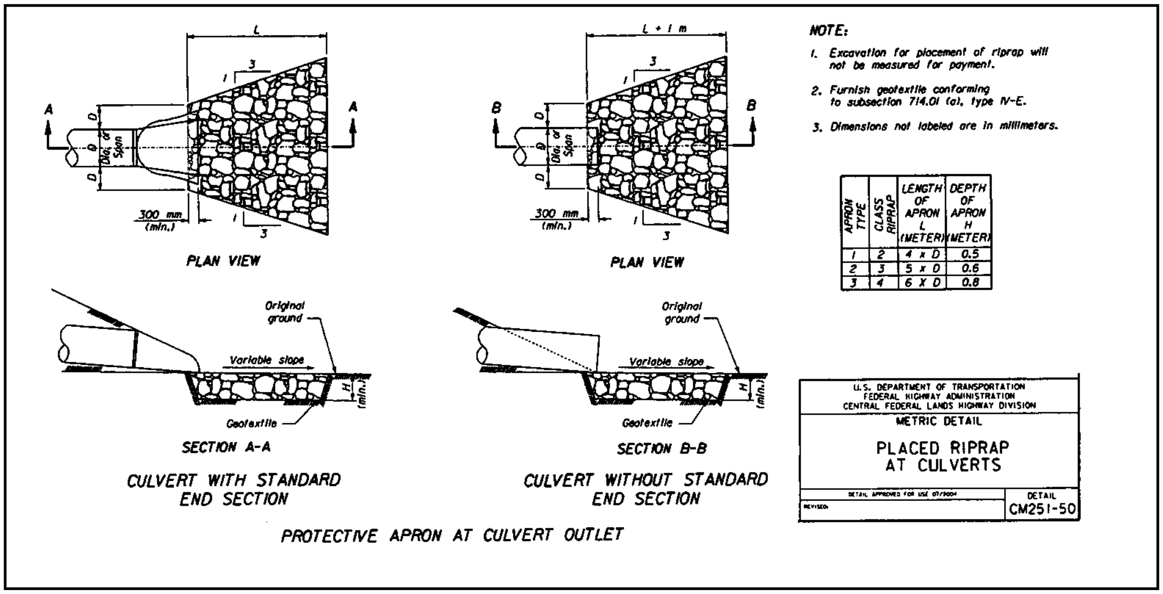
Two feasible options have been identified. First, a 2.3-ft-deep, 23-ft-long pool, with an 11.5-ft-apron using D_{50} = 0.5 ft. Second, a 1.4-ft-deep, 18-ft-long pool, with a 6-ft-apron using D_{50} = 0.83 ft. The choice between these two options will likely depend on the available space and the cost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



NOTE:

1. Excavation for placement of riprap will not be measured for payment.
2. Furnish geotextile conforming to subsection 714.01 (a), type IV-E.
3. Dimensions not labeled are in millimeters.

APRON TYPE	CLASS RIPRAP	LENGTH OF APRON L (METER)	DEPTH OF APRON H (METER)
1	2	4 x D	0.5
2	3	5 x D	0.6
3	4	6 X D	0.8

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 = $2.33 \text{ m}^3/\text{s}$
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

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).

NOVATECH
Engineers, Planners & Landscape Architects

MEMORANDUM

DATE: APRIL 16, 2024

TO: SAM BAHIA & BEN SWEET (NOVATECH)

FROM: OLIVIA RENN & MIKE PETEPIECE (NOVATECH)

RE: 4386 RIDEAU VALLEY DRIVE – STINSON LANDS
OXBOW WATER BALANCE
121153

This memorandum provides an overview of the water balance calculations completed in support of the recommended storm outlet for the Stinson Lands. The water balance was completed to assess the amount of runoff from the site draining to the oxbow under pre- and post-development conditions and evaluate the potential impacts of the proposed development on normal water levels in the oxbow.

Background Documents

The following documents were reviewed in preparation of this memo:

- Stormwater Management Planning and Design Manual (MOE, 2003)
- Groundwater Impact Assessment – Proposed Residential Development – 4386 Rideau Valley Drive, Ottawa, Ontario (Paterson, August 2022)
- Mud Creek Flood Risk Mapping from Prince of Wales Drive to Rideau River (RVCA, July 2019)

Existing (Pre-Development) Drainage Conditions

Under existing conditions, the oxbow receives overland storm runoff from approximately 3.57ha of the Stinson property – refer to Figures 1 and 2. There is an existing berm at the outlet from the oxbow to Mud Creek which creates a permanent water feature in the oxbow by retaining water below the berm elevation of 81.35m. This can be considered as the 'normal' water level in the oxbow. The oxbow has a retention volume (permanent pool) of approximately 1000m^3 at the top of the berm.

Mud Creek has a 2-year water level of 82.22m, which is approximately 0.9m above the top of the berm at the outlet from the oxbow. Water levels in the oxbow will temporarily rise above 81.35m during times when water levels in Mud Creek are elevated. This will occur most often during the spring freshet but can also occur following significant rainfall events at other times of year.

Water levels in the oxbow will fluctuate over the course of the year. Water levels will gradually decrease due to losses from infiltration and evapotranspiration but will be regularly replenished from storm runoff and during periods where the water level in Mud Creek is above the berm.

Refer to the Oxbow Plan and Profile (Drawing 121153-OXBOW) for details.

Historical Photos

GeoOttawa was used to compare aerial photographs of the oxbow over multiple years and at different times of year. Based on the review of the aerial photographs (Figures 4-7), it is evident that the oxbow does retain water year-round. The highest water levels occur during periods where the water levels in Mud Creek are above the berm at the outlet of the oxbow, as seen on Figure 5 (April/May 2017), Figure 6 (April/May 2014), and Figure 7 (2011).

Water Balance Calculations

The water balance calculations were conducted using 30 years of meteorological data. Actual evapotranspiration and water surplus values were calculated using the Thornthwaite-Mather (1957) methodology while the runoff and infiltration values were calculated using the methodology presented in Section 3.2 of the Stormwater Management Planning and Design Manual (MOE, 2003). Pre-development and post-development runoff volumes to the oxbow were estimated based on existing and proposed site conditions (land use, topography, soil characteristics, etc.). The results are summarized in **Table 1** below.

Table 1: Annual Runoff to the Oxbow (Pre vs. Post-Development)

	Area (ha)	Runoff (mm/yr)	Runoff (m^3/yr)
PRE	3.57	294	10,514
POST	0.91	396	3,600

Under post-development conditions, the drainage area from the site to the oxbow will be reduced from 3.57 ha to 0.91 ha. The results of the water balance analysis indicate that annual runoff volumes from the site to the oxbow will decrease from 10,514 m^3/yr to 3,600 m^3/yr, approximately 66% less than pre-development conditions. Refer to attachments for drainage area figures, water balance methodology and results.

Impact to Normal Water Level & Retention Volume

While there will be a reduction in runoff to the oxbow under post-development conditions – refer to Figure 3, additional calculations were completed to determine whether the post-development runoff volumes will be sufficient to maintain normal water levels in the oxbow throughout the year.

Due to the existing berm at the Mud Creek outlet, the oxbow has a total retention volume of approximately 1,000 m^3 at a ‘normal’ water level of 81.35 m. Water levels in the oxbow will periodically drop below this elevation due to infiltration and evaporation and will be replenished either by runoff from the contributing drainage area or by backwater from Mud Creek when water levels are above the outlet berm.

Runoff Volume (Input)

Based on the water balance calculations, 3,600 m^3 of runoff will drain to the oxbow annually under post-development conditions. This is approximately 3.6x the permanent retention volume of the oxbow (1000 m^3).

Infiltration (Loss)

The Groundwater Impact Assessment prepared by Paterson provides a soil hydraulic conductivity of $1x10^{-7}$ m/s for silty clay which is representative of the soils in the area. A daily infiltration volume of 14.7 m^3 was calculated by multiplying the hydraulic conductivity by the 0.17 ha footprint of the oxbow (assumed constant infiltration year-round).

Evaporation (Loss)

Daily evaporation volumes were calculated by multiplying the City of Ottawa's lake evaporation values (mm/day) by the 0.17 ha footprint of the oxbow.

Table 2 provides a summary of the calculated average monthly runoff and infiltration/evaporation volumes to/from the oxbow under post-development conditions. The results of this analysis indicate that the average monthly runoff volume to the oxbow will be greater than the volume lost to infiltration/evaporation.

Table 2: Average Monthly Post-Development Runoff and Infiltration/Evaporation

Month	Runoff[1] (m^3)	Infil./Evap.[2] (m^3)	Net Volume[3] (m^3)
January	368.7	79.1	289.6
February	330.4	79.5	250.9
March	532.5	131.5	401.0
April	457.6	132.7	324.9
May	182.6	80.4	102.2
June	161.9	76.2	85.7
July	131.1	70.9	60.2
August	151.5	65.4	86.2
September	184.6	80.4	104.2
October	377.1	120.2	256.8
November	400.8	122.0	278.8
December	320.8	90.8	230.0
ANNUAL TOTAL	**3,600**	**1,129**	**2,471**

[1] Post-development runoff volume to the oxbow.
[2] Volume of water infiltrated/evaporated from the oxbow.
[3] Volume of runoff retained within the oxbow.

Mud Creek Backwater (Input)

The RVCA's Mud Creek Flood Risk Mapping technical memo indicate that water levels in Mud Creek will periodically rise above the top of the berm at the outlet from the oxbow and contribute to maintaining normal water levels in the oxbow. The proposed development will have negligible impact on flows and water levels in Mud Creek, so the frequency and duration of backwater from Mud Creek into the oxbow will not change under post-development conditions. The contributions from Mud Creek to the oxbow have not been included in the water balance analysis and should not be required to maintain the retention volume in the oxbow below 81.35m.

Conclusions

Based on long-term climate data, the water balance analysis demonstrates that the proposed post-development drainage area to the oxbow (0.91 ha) will generate sufficient runoff to maintain the 'normal' water level and retention volume.

Monthly average runoff volumes to the oxbow will exceed the calculated losses from infiltration/evaporation, and the net annual water contribution to the oxbow (2,471 m^3) is greater than the retention volume in the oxbow (1,000 m^3) at the normal water level. Based on this analysis, it can be concluded that the proposed development will provide a net surplus of water to the oxbow and should be sufficient to maintain the oxbow as a permanent water feature.

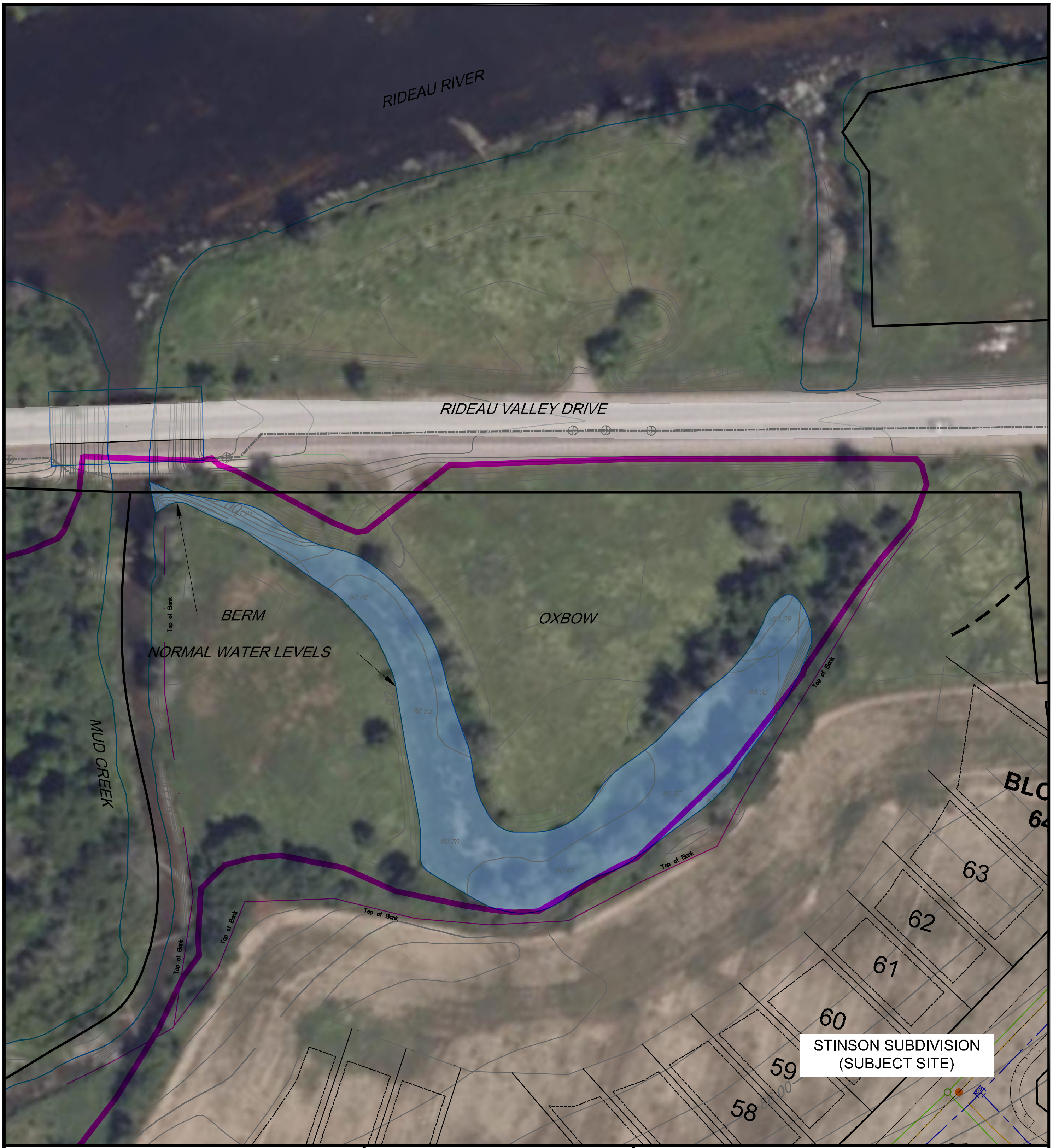
The oxbow will continue to be periodically inundated by backwater from Mud Creek under post-development conditions. This will occur most often during the spring freshet but can also occur during larger storm events over the course of the year. During these periods, the backwater from Mud Creek will result in water levels in the oxbow above 81.35m, but this excess water will quickly drain back into Mud Creek once water levels in the creek drop below the height of the berm. The water balance analysis indicates that the additional volume from backwater is not required to maintain the normal water level.

Attachments

Figure 1:	Oxbow Existing Conditions
Figure 2:	Pre-Development Drainage Area
Figure 3:	Post-Development Drainage Area
Figures 4-7:	Existing Oxbow Ditch Aerial Photos

Oxbow Plan and Profile (Drawing 121153-Oxbow)

Water Balance Methodology
Water Balance Calculations



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CITY of OTTAWA
STINSON SUBDIVISION
4386 RIDEAU VALLEY DRIVE

OXBOW
EXISTING CONDITIONS

SCALE 1 : 1000 0 10 20 30 40

DATE	JOB	FIGURE
APR 2024	121153	1

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Figure 2: Pre-Development Drainage Area

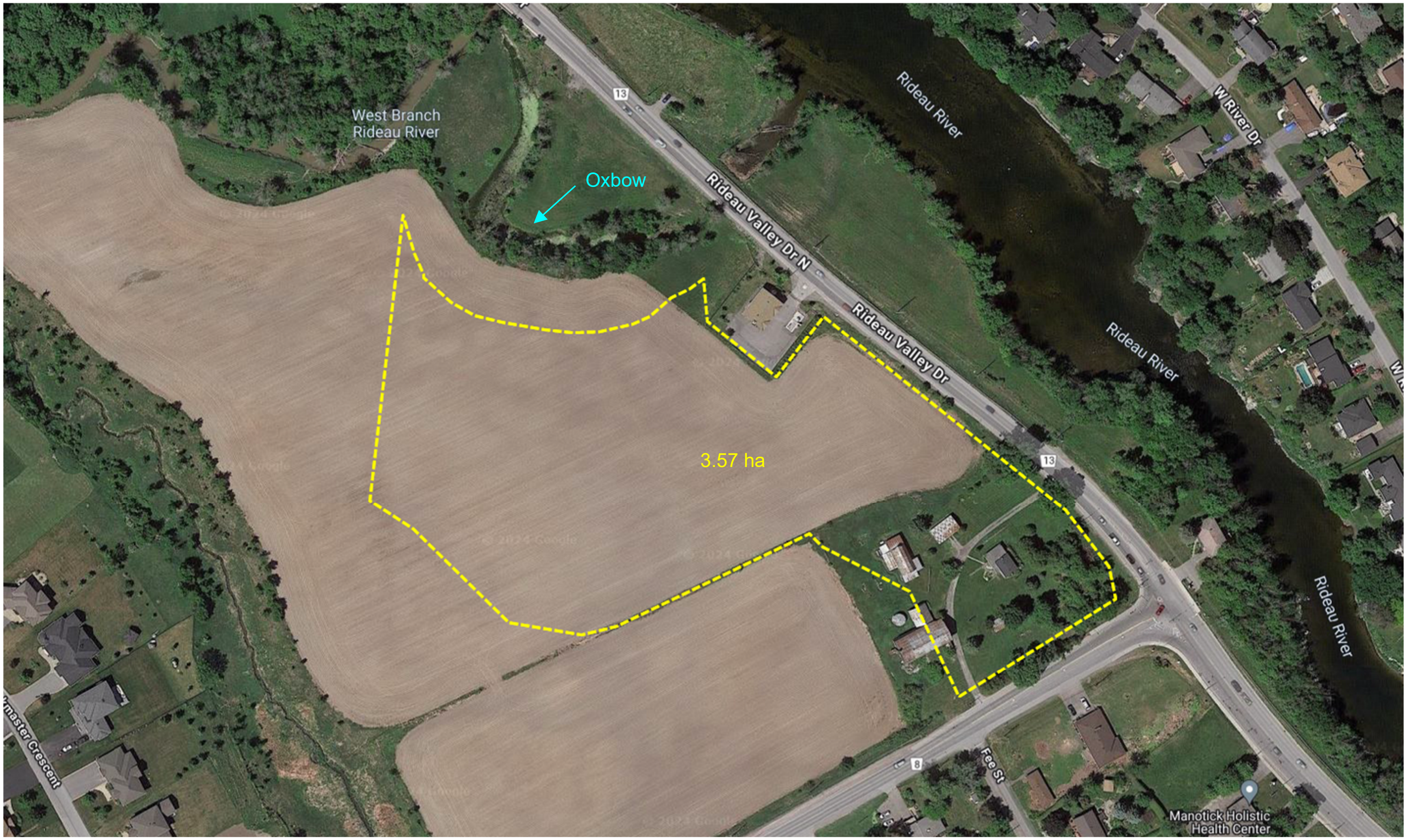
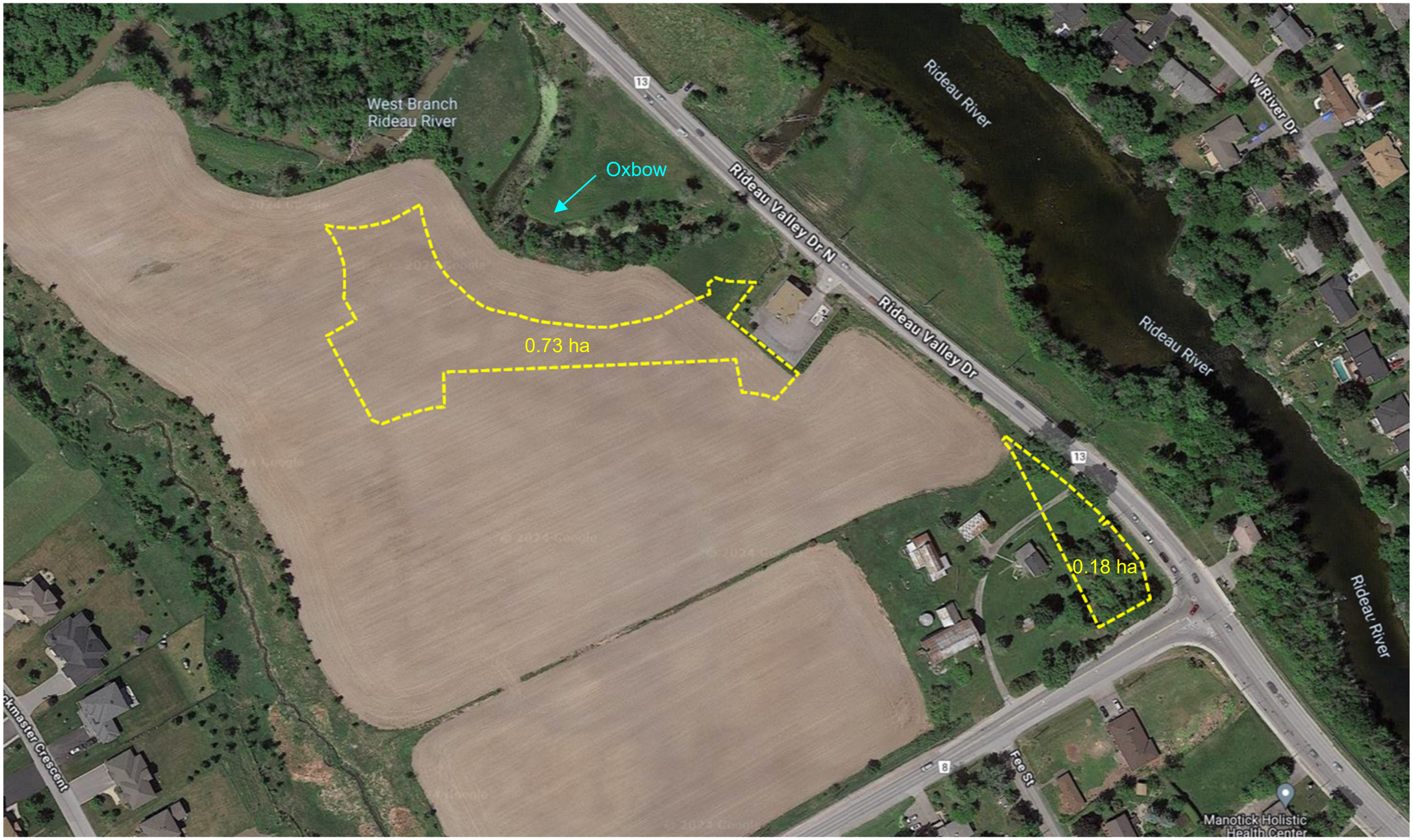
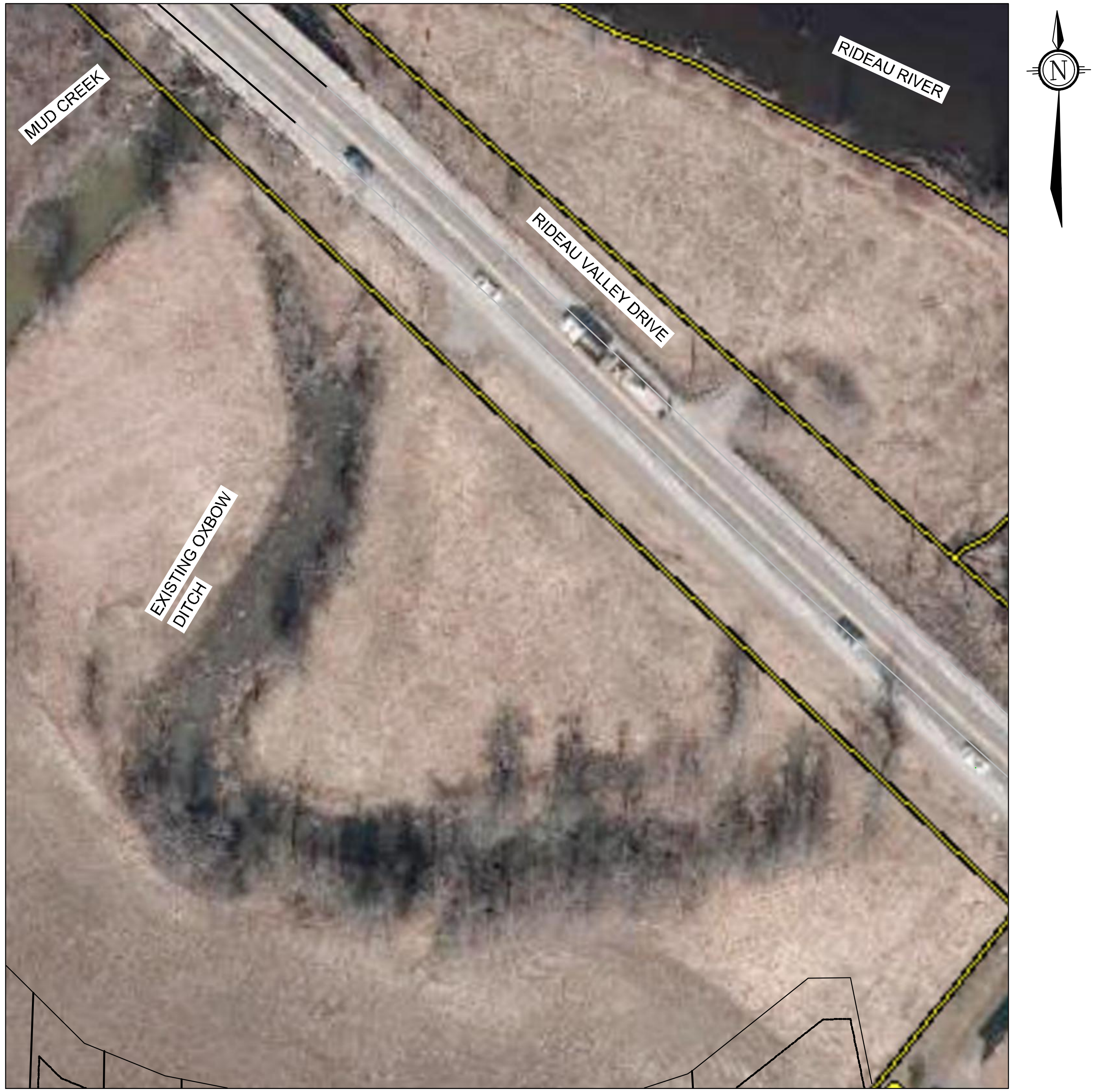


Figure 3: Post-Development Drainage Areas



JULY/AUGUST 2022 AERIAL

MARCH/APRIL 2021 AERIAL



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CITY OF OTTAWA
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4386 RIDEAU VALLEY DRIVE

EXISTING OXBOW DITCH
AERIAL

SCALE
NOT TO SCALE

DATE
APR 2024

JOB
121153

FIGURE
4

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AUGUST/SEPTEMBER 2019 AERIAL

APRIL/MAY 2017 AERIAL

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EXISTING OXBOW DITCH
AERIAL

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DATE	JOB	FIGURE
APR 2024	121153	5

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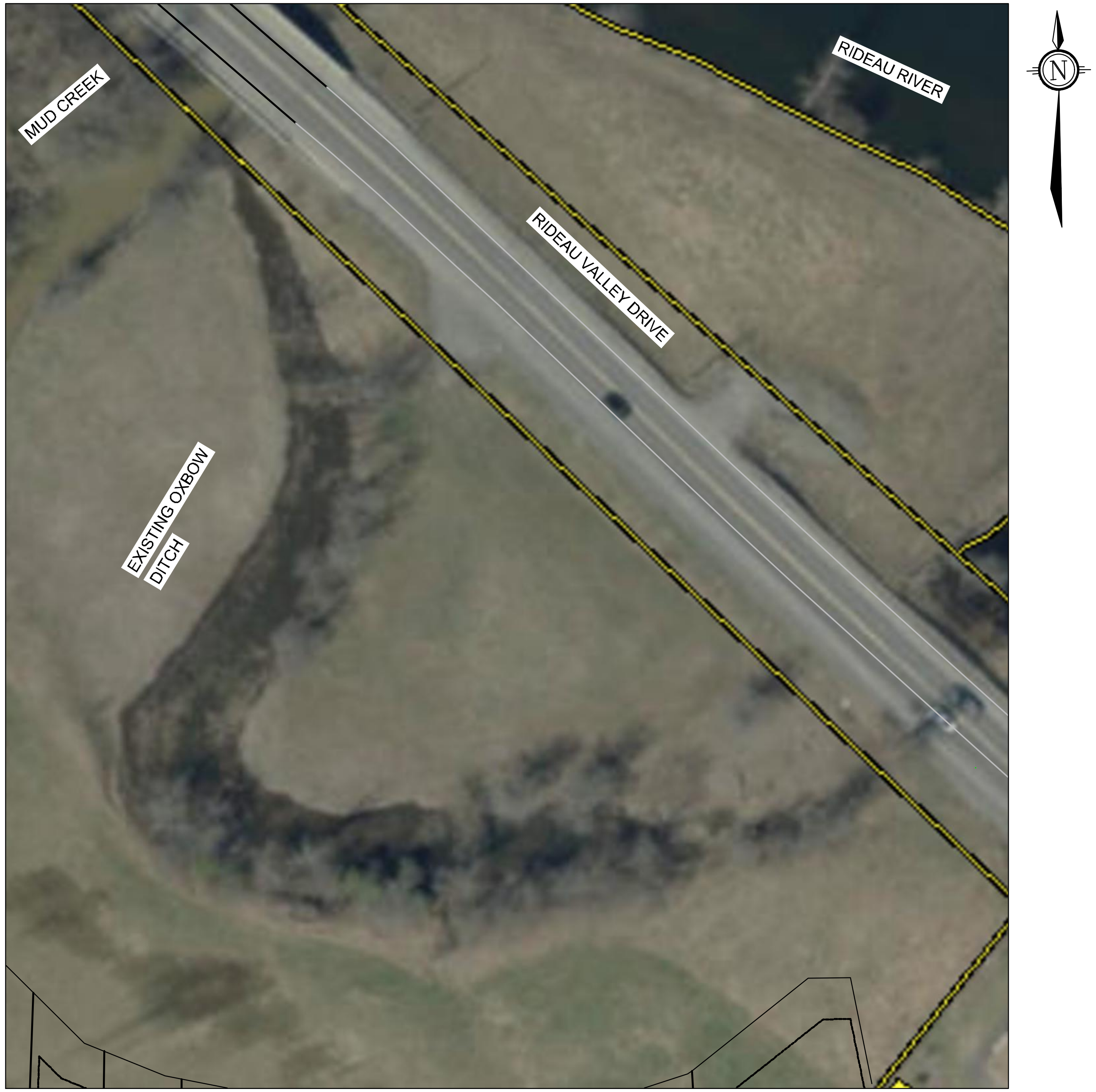
JULY/AUGUST 2015 AERIAL

APRIL/MAY 2014 AERIAL



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EXISTING OXBOW DITCH AERIAL

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DATE	JOB	FIGURE
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2011 AERIAL

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EXISTING OXBOW DITCH AERIAL

SCALE
NOT TO SCALE

DATE	JOB	FIGURE
APR 2024	121153	7

SHT11X17.DWG - 279mmX432mm

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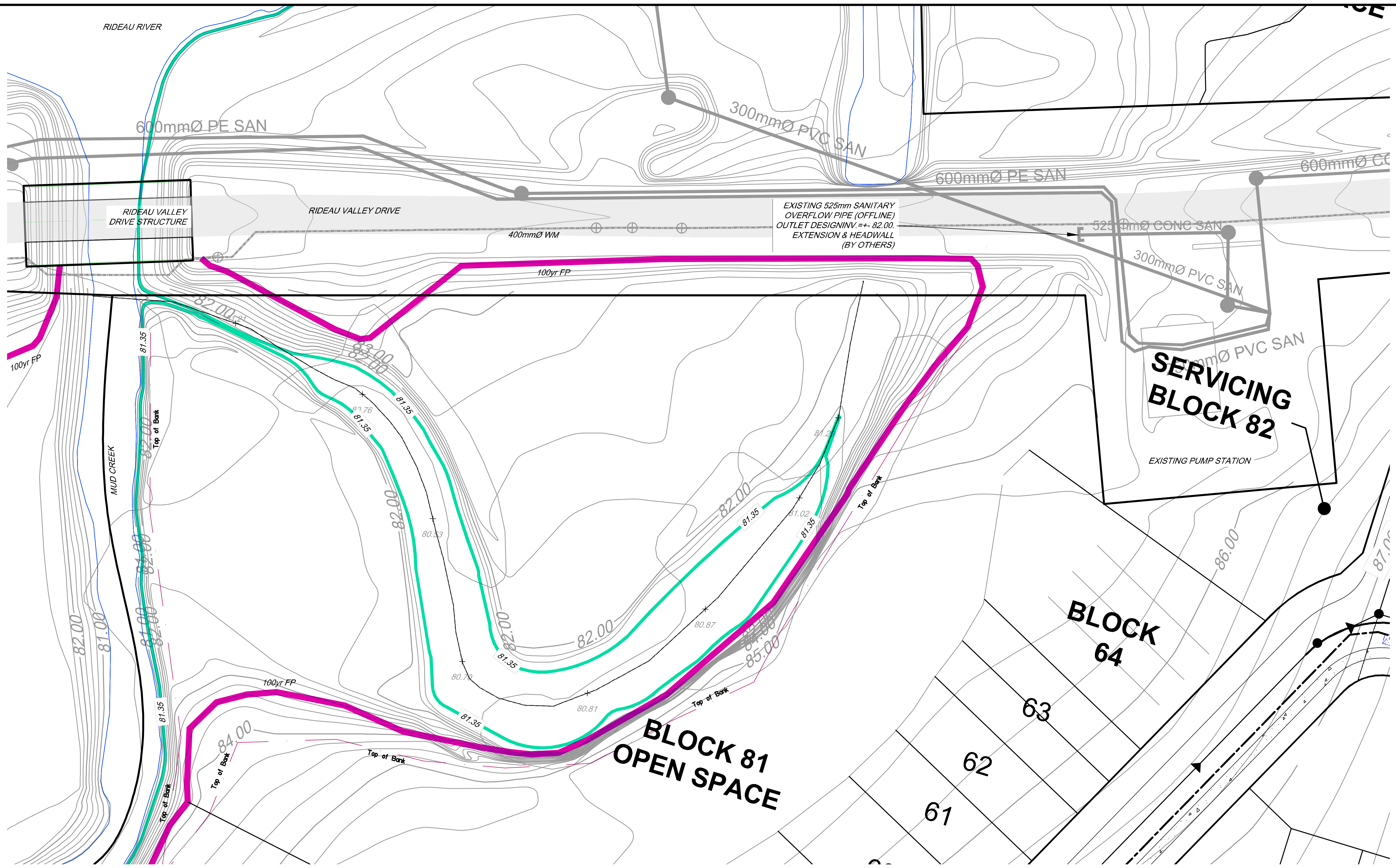
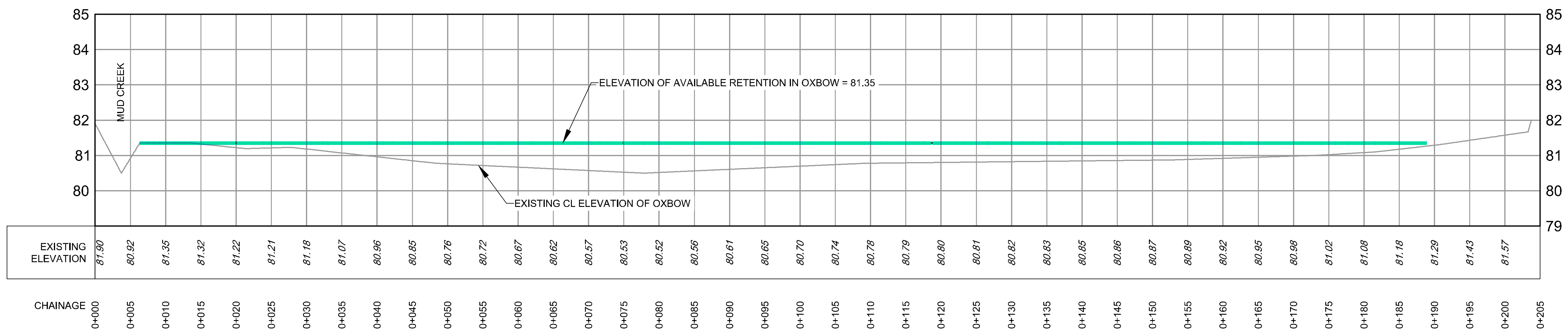


OXBOW - PLAN VIEW
SCALE: 1:500

LEGEND

🟩	AVAILABLE RETENTION STORAGE IN OXBOW = ~1000m³
🟥	100 YEAR FLOODPLAIN LIMIT
—	EXISTING WATERCOURSE



CHAINAGE	EXISTING ELEVATION
0+000	81.90
0+005	80.92
0+010	81.35
0+015	81.32
0+020	81.22
0+025	81.21
0+030	81.18
0+035	81.07
0+040	80.96
0+045	80.85
0+050	80.76
0+055	80.72
0+060	80.67
0+065	80.62
0+070	80.57
0+075	80.53
0+080	80.52
0+085	80.56
0+090	80.61
0+095	80.65
0+100	80.70
0+105	80.74
0+110	80.78
0+115	80.79
0+120	80.80
0+125	80.81
0+130	80.82
0+135	80.83
0+140	80.85
0+145	80.86
0+150	80.87
0+155	80.89
0+160	80.92
0+165	80.95
0+170	80.98
0+175	81.02
0+180	81.08
0+185	81.18
0+190	81.29
0+195	81.43
0+200	81.57
0+205	

OXBOW - PROFILE VIEW
SCALE: 1:500 (H) & 1:25 (V)

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NOTE:
1. OXBOW IS TO GENERALLY REMAIN UNTOUCHED TO MAINTAINT ITS NATURAL FEATURES AND ECOLOGICAL FUNCTION.
2. OWNER AND CONTRACTOR ARE OBTAIN ALL NECESSARY DFO, PARKS CANADA, MECP, RVCA, AND CITY PERMITS.
3. ANY IN-WATER WORKS ARE TO BE COMPLETED BETWEEN JULY 1 AND JANUARY 1.
4. PRIMARY AND SECONDARY CONTAINMENT OF WASTEWATER FLOWS DURING PUMP STATION CATASTROHPIC EVENTS ARE TO BE CLEANED UP BY CITY OPERATIONS, SHOULD A SPILL OCCUR.
5. STORMWATER RUNOFF FROM THE PROPOSED SUBDIVISION IS TO BE TREATED VIA A WATER QULAITY TREATMENT UNIT (WQT) FOR ENHANCED LEVEL TREATMENT PER MECP GUIDELINES (80% TSS REMOVAL).

No.	REVISION	DATE	BY
2.	REVISED & REISSUED FOR OXBOW WATER BALANCE MEMORANDUM	APR 15/24	BHB
1.	PRELIMINARY DESIGN	JAN 9/23	BHB

SCALE
AS SHOWN

DESIGN	BPR
CHECKED	BHB
DRAWN	BPR
CHECKED	BHB
APPROVED	BHB

FOR REVIEW ONLY

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CITY of OTTAWA
STINSON SUBDIVISION
4386 RIDEAU VALLEY DRIVE

DRAWING NAME
OXBOW PLAN AND PROFILE

PROJECT No. 121153
REV REV #2
DRAWING No. 121153-OXBOW

PLAN24x36.DWG - 914.4mmx609.6mm

Overview

The Thornthwaite-Mather (1957) water balance models are conceptual models that are used to simulate steady-state climatic averages or continuous values of precipitation (rain + snow), snowpack, snowmelt, soil moisture, evapotranspiration, and water surplus (infiltration + runoff) (refer to **Figure 1**). Input parameters consist of daily precipitation *(PRECIP),* temperature *(MAX / MIN TEMP),* potential evapotranspiration *(PET)*, and the available water content *(AWC)* that can also be referred to as the water holding capacity of the soil. All water quantities in the model are based on monthly calculations and are represented as depths (volume per unit area) of liquid water over the area being simulated. *All model units are in millimetres (mm)*.

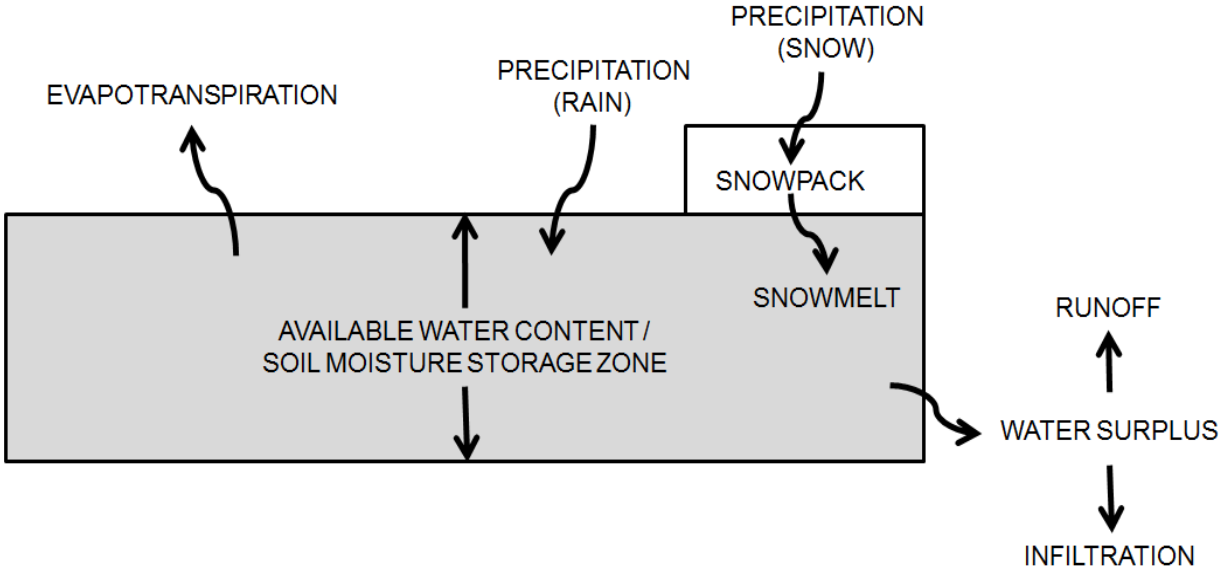


Figure 1: Conceptual Water Balance Model

Available Water Content (Water Holding Capacity)

The available water content (AWC) or water holding capacity of the soil was taken from Table 3.1 from the *Stormwater Management Planning & Design Manual (MOE, 2003)*, which has been reproduced in **Table 1** below. The available water content is the soil-moisture storage zone or the zone between the field capacity and vertical extent of the root zone.

Table 1: Water Holding Capacity Values (MOE, 2003)

Land Use / Soil Type	Hydrologic Soil Group	Water Holding Capacity (mm)
Urban Lawns / Shallow Rooted Crops (spinach, beans, beets, carrots)		
Fine Sand	A	50
Fine Sandy Loam	B	75
Silt Loam	C	125
Clay Loam	CD	100
Clay	D	75

Land Use / Soil Type	Hydrologic Soil Group	Water Holding Capacity (mm)
Moderately Rooted Crops (corn and cereal grains)		
Fine Sand	A	75
Fine Sandy Loam	B	150
Silt Loam	C	200
Clay Loam	CD	200
Clay	D	150
Pasture and Shrubs		
Fine Sand	A	100
Fine Sandy Loam	B	150
Silt Loam	C	250
Clay Loam	CD	250
Clay	D	200
Mature Forests		
Fine Sand	A	250
Fine Sandy Loam	B	300
Silt Loam	C	400
Clay Loam	CD	400
Clay	D	350

Precipitation

Daily precipitation *(PRECIP)* values consist of the total daily rainfall and water equivalent of snowmelt that fell on that day. Based on the mean daily temperature *(MEAN TEMP)* precipitation falls either as rainfall *(RAIN)* or the water equivalent of snowfall *(SNOW)*:

- *RAIN: If (MEAN TEMP >= 0, RAIN, SNOW)*
- *SNOW: If (MEAN TEMP < 0, SNOW, RAIN)*

Snowmelt / Snowpack / Water Input

Snowmelt *(MELT)* occurs if there is available snow (water equivalent) in the snowpack *(SNOWPACK)* and the maximum daily temperature *(MAX TEMP)* is greater than 0. The available snowmelt is limited to the available water in the snowpack.

Snowmelt is computed by a degree-day equation (Haith, 1985):

$$SNOWMELT\ (cm/d) = MELT\ COEFICIENT \times [AIR\ TEMP\ (^{O}C) - MELT\ TEMP(^{O}C)]$$

The melt coefficient is typically 0.45 (cm of depth per degree-day, or cm x C^{-1} x day^{-1}) for northern climates (Haith, 1985). The melt temperature is assumed to be $0^{O}C$. The air temperature is assumed to be the max temperature multiplied by a ratio of the max to min temperatures:

$$AIR\ TEMP = [MAX\ TEMP / (MAX\ TEMP - MIN\ TEMP)]$$

Therefore, the snowmelt equation is:

- *MELT: If (MAX TEMP > 0, IF(SNOWPACK > 0, MIN((0.45cm/°C-day*MAX TEMP*[MAX TEMP/(MAX TEMP – MIN TEMP)]*10mm/cm), SNOWPACK), 0), 0)*

Snow accumulates in the snowpack from the previous day if precipitation falls as snow and there is no snowmelt or the amount of snow that falls in a day exceeds the daily snowmelt:

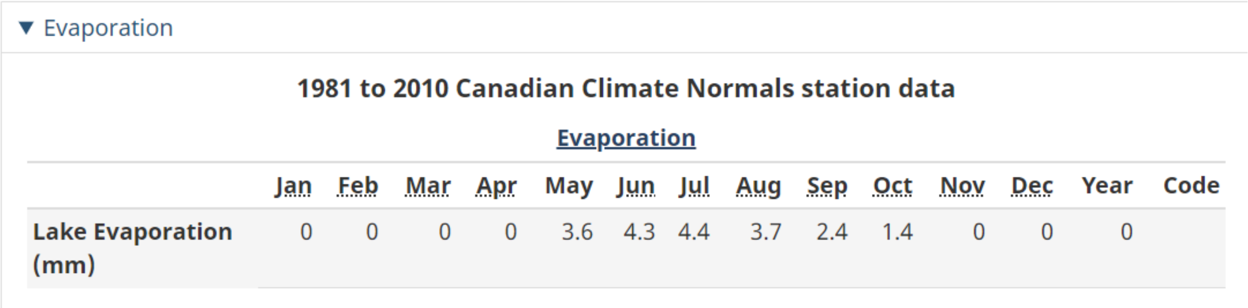
$\mathrm{SNOWPACK}_{N} = \mathrm{SNOWPACK}_{N-1} + \mathrm{SNOW} - \mathrm{MELT}$

The initial snowmelt on day 1 (i.e. January 1) is assumed to be 0. The initial snowpack on day 1 is assumed to be the snowpack on the last day of simulation (i.e. December 31).

The total water input (W) is rain + snowmelt. This is the available water that fills the soil moisture storage zone each day.

Evaporation

Measured potential evaporation (PE) data (i.e. lake evaporation) is provided with the Environment Canada Climate Normals (see example below for Ottawa CDA). The data represents daily averages for each month over a 20+ year period.



▼ Evaporation

1981 to 2010 Canadian Climate Normals station data

Evaporation

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Code
Lake Evaporation (mm)	0	0	0	0	3.6	4.3	4.4	3.7	2.4	1.4	0	0	0	

The daily evaporation data was assumed to represent the middle or 15th of each month and ‘smoothed’ to represent the transition from month to month (see **Figure 2** below). As shown in **Figure 2**, this produces a more realistic curve of potential evapotranspiration.

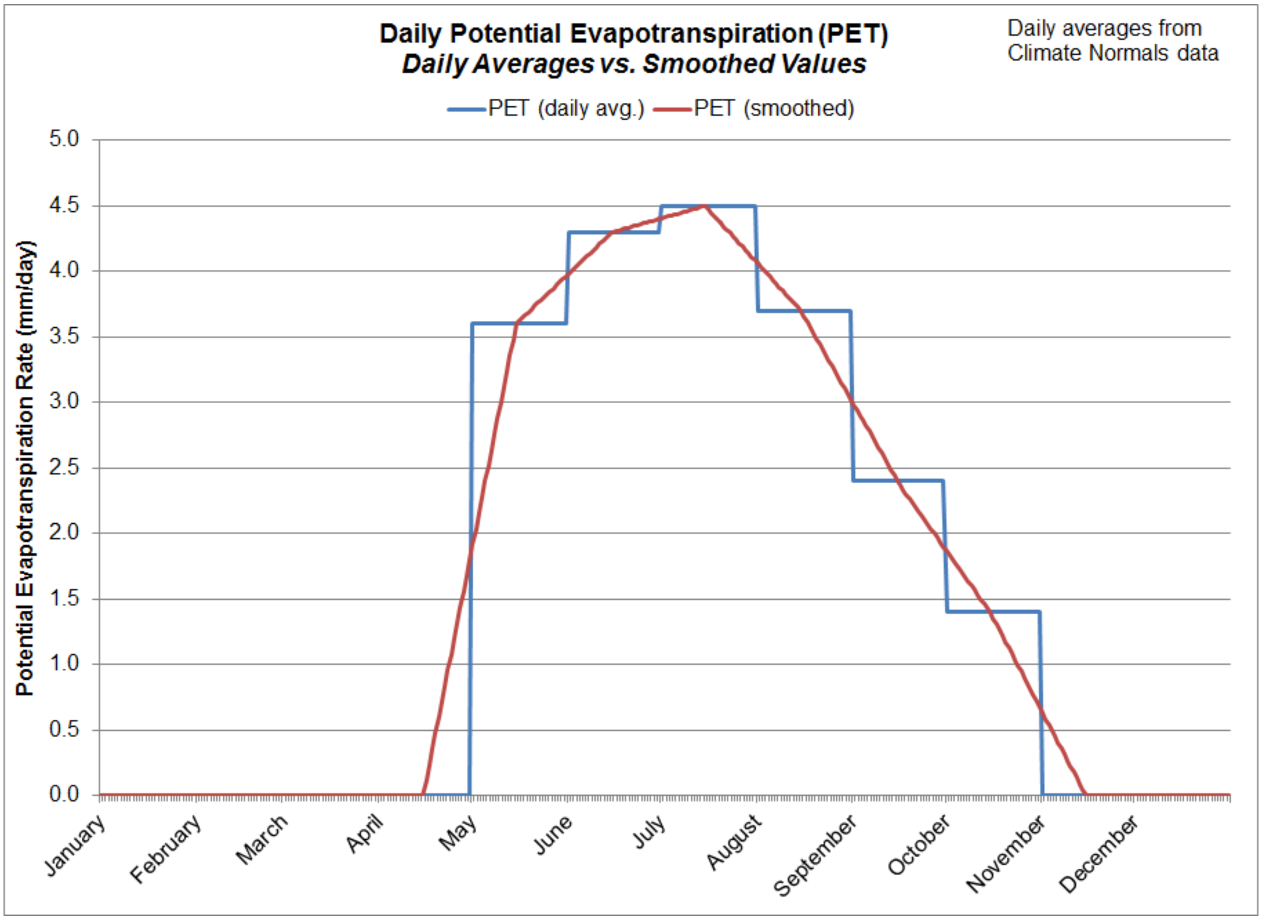


Figure 2: Daily Potential Evapotranspiration Rates (Daily Averages vs. Smoothed Values)

Potential Evapotranspiration

To convert potential evaporation data to potential crop evapotranspiration (PET) data a cover coefficient is applied based on land use and growing / dormant seasons:

PET = PE x Crop Cover Coefficient

Crop cover coefficients are based on the crop growth stages for different crop types (see **Figure 3**). A typical crop coefficient curve is shown in **Figure 4**, which depicts a crop that provides transpiration above the potential evaporation rates during the growing season.

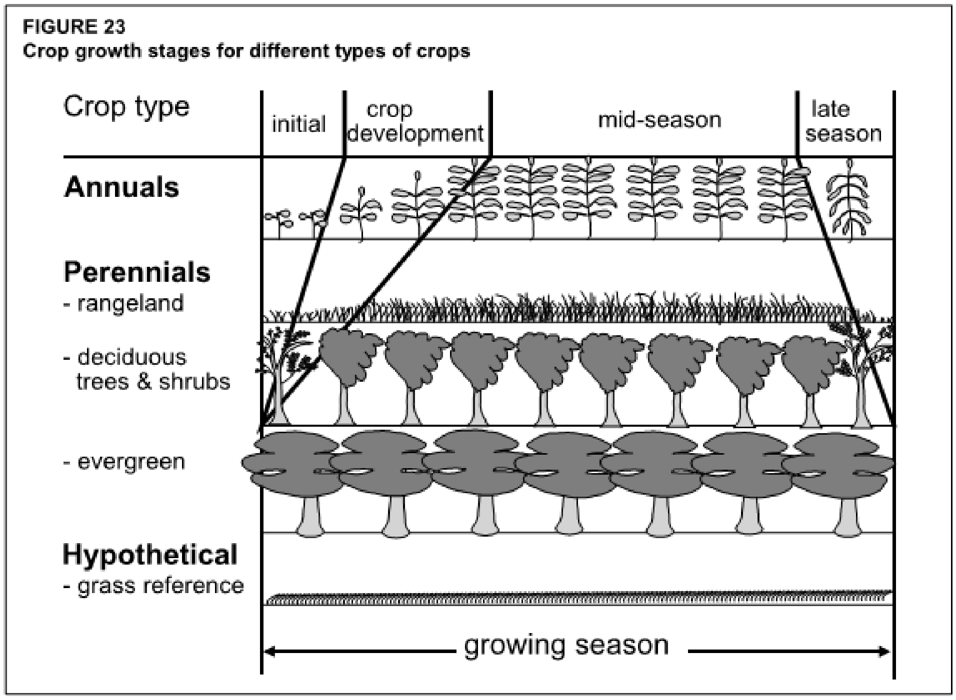


Figure 3: Crop Growth Stages for Different Types of Crops

Source: Food and Agriculture Organization of the United Nations (FAO), 1998, Crop Evapotranspiration - Guidelines for Computing Crop Water Requirements. FAO Irrigation and Drainage paper 56.

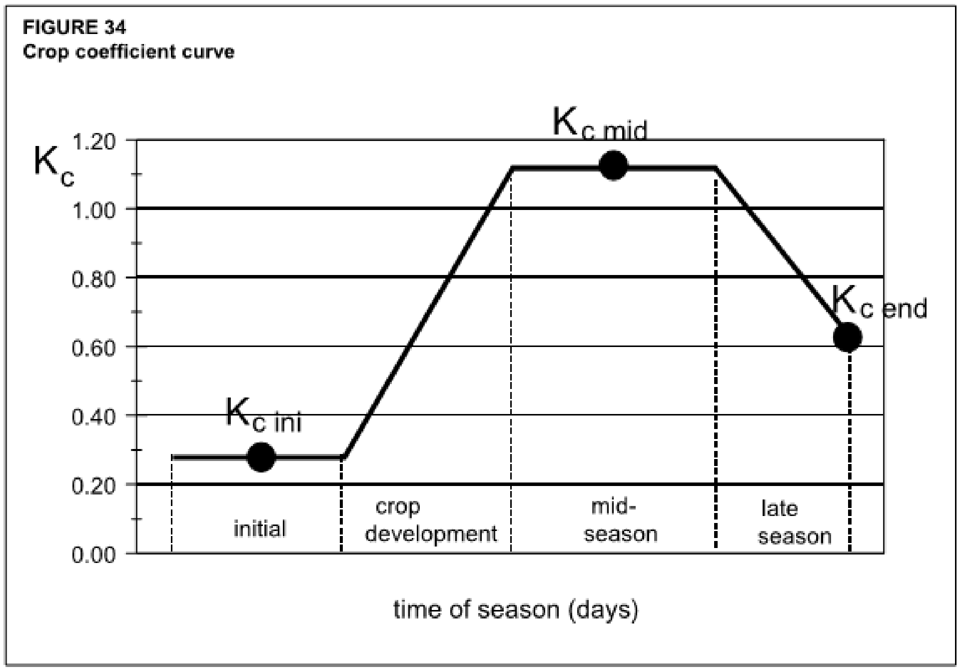


Figure 4: Crop Coefficient Curve

Source: Food and Agriculture Organization of the United Nations (FAO), 1998, Crop Evapotranspiration - Guidelines for Computing Crop Water Requirements. FAO Irrigation and Drainage paper 56.

The crop cover coefficients used in the water budget model for the various land use types is shown in **Table 2**. The growing / dormant seasons are shown in **Table 3**. The crop cover coefficients for the initial growing season are based on the average value of the dormant and middle of the growing season.

Table 2: Crop Cover Coefficients

Land Use	Dormant Season	Initial Growing Season	Middle of Growing Season	End of Growing Season
Urban Lawns / Shallow Rooted Crops*	0.40	0.78	1.15	0.55
Moderately Rooted Crops**	0.30	0.73	1.15	0.40
Pasture and Shrubs***	0.40	0.68	0.95	0.90
Mature Forest****	0.30	0.75	1.20	0.30
Impervious Areas	1.00	1.00	1.00	1.00

Reference: Data is based on Table 12 from the Food and Agriculture Organization of the United Nations (FAO), 1998, Crop Evapotranspiration - Guidelines for Computing Crop Water Requirements. FAO Irrigation and Drainage paper 56.

**Table 12, e. Legumes*
***Table 12, i. Cereals*
****Table 12, j. Forages (Alfalfa)*
*****Table 12, o. Wetlands*

Table 3: Crop Growing Season

Month(s)	Crop Growing Season
January – April	Dormant Season
May	Initial Growing Season
June - August	Middle of Growing Season
September	End of Growing Season
October - December	Dormant Season (harvest in October)

Reference: Food and Agriculture Organization of the United Nations (FAO), 1977, Crop Water Requirements. FAO Irrigation and Drainage paper 24.

Actual Evapotranspiration

Following Alley (1984), if the monthly water input (i.e. rain + snowmelt) is greater than the potential evapotranspiration (PET) rate, the actual evapotranspiration (AET) rate takes place at the potential evapotranspiration rate:

$IF\ W > PET,\ then\ AET = PET$

If the monthly water input is less than the potential evapotranspiration rate (i.e. W < PET) then the actual evapotranspiration rate is the sum of the water input and an increment removed from the available water in the soil moisture storage zone (SOIL WATER):

$IF\ W < PET,\ then\ AET = W + \Delta SOIL\ WATER$

$WHERE:\ \Delta SOIL\ WATER = SOIL\ WATER_{N-1} - SOIL\ WATER_{N}$

Figure 5 shows a comparison of the average monthly potential evapotranspiration and actual evapotranspiration rates.

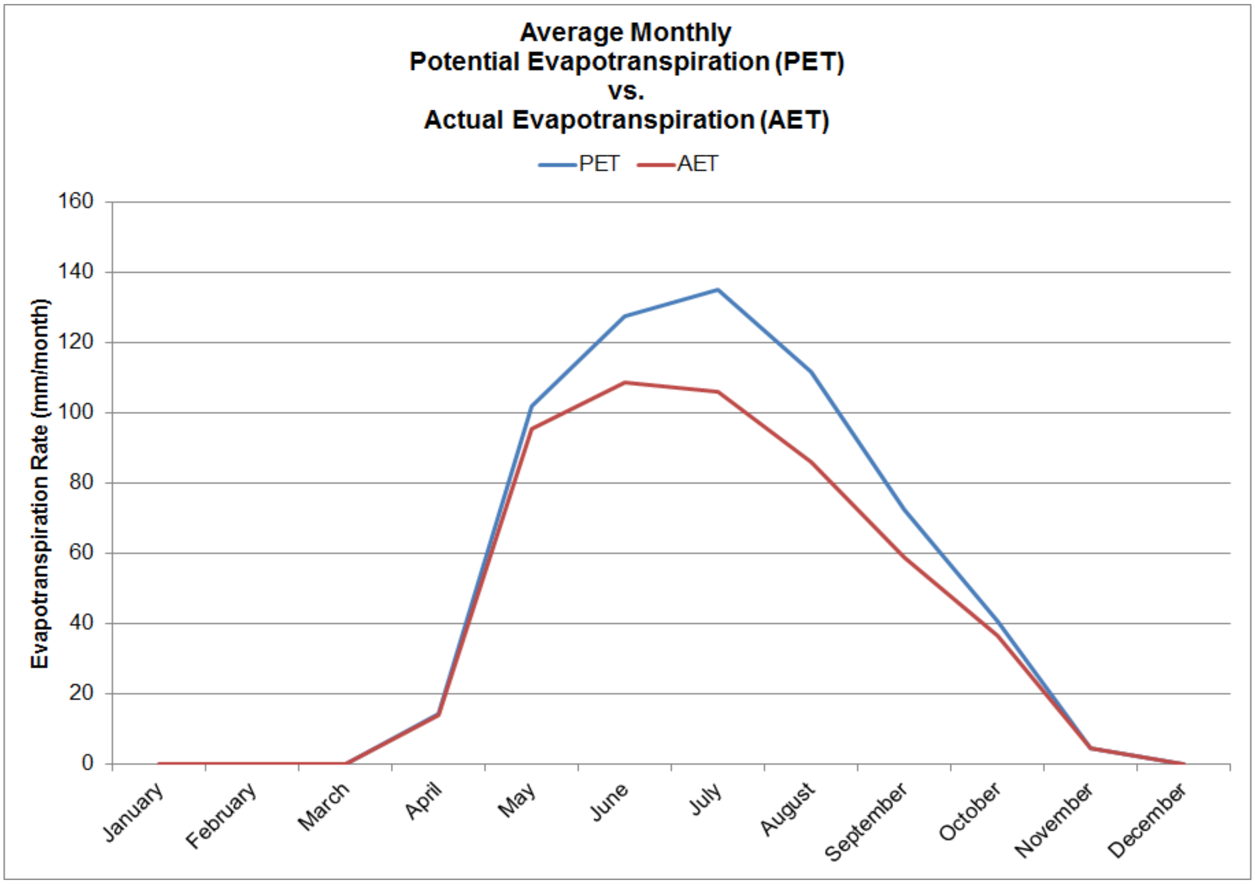


Figure 5: Average Monthly Potential Evapotranspiration vs. Actual Evapotranspiration

Soil Moisture

The soil moisture storage zone (SOIL WATER) is the amount of water available for actual evapotranspiration, but actual evapotranspiration is limited by the potential evapotranspiration rate.

The decrease / change in the soil moisture storage zone (ΔSOIL WATER) is based on the following relationship (Thornthwaite,1948), where AWC represents the available water content:

$\Delta SOIL\ WATER = SOIL\ WATER_{N-1}\ x\ [1-exp(-((PET - W) / AWC))]$

The soil moisture storage zone is replenished with rainwater and snowmelt (i.e. the water input) to the maximum value of the available water content (AWC):

$SOIL\ WATER_N = min[(W - PET) + SOIL\ WATER_{N-1}), AWC]$

Water Surplus

The water surplus (SURPLUS) is defined as the excess water that is greater than the available water content (AWC).

$SURPLUS = W - AET - \Delta SOIL\ WATER$

The water surplus represents the difference between precipitation and evapotranspiration. It is an estimate of the water that is available to contribute to infiltration and runoff (i.e. streamflow).

Infiltration / Runoff

The amount of water surplus that is infiltrated is determined by summing the infiltration factors (IF) based on topography, soils, and land cover. Since the water surplus represents infiltration and runoff; direct runoff is the amount of water surplus remaining after taking into account infiltration: (1.0 – infiltration factor = runoff factor). The infiltration and runoff factors were applied to the average monthly water surplus values:

$INFILTRATION = IF \times SURPLUS$

$RUNOFF = (1.0 - IF) \times SURPLUS$

The infiltration factors are shown in **Table 4**, which was reproduced from Table 3.1 in the *Stormwater Management Planning & Design Manual (MOE, 2003)*. These infiltration factors were initially presented in the document *"Hydrogeological Technical Information Requirements for Land Development Applications" (MOE, 1995)*.

Table 4: Infiltration Factors (MOE, 2003)

Description	Value of Infiltration Factor
Topography	
Flat Land, average slope < 0.6 m/km	0.3
Rolling Land, average slope 2.8 m/km to 3.8 m/km	0.2
Hilly Land, average slope 28 m/km to 47 m/km	0.1
Surficial Soils	
Tight impervious clay	0.1
Medium combination of clay and loam	0.2
Open sandy loam	0.4
Land Cover	
Cultivated Land	0.1
Woodland	0.2

Each soil type been assigned a corresponding infiltration factor as per Table 3.1 in the *Stormwater Management Planning & Design Manual (MOE, 2003),* as shown in **Table 5** below.

Table 5: Soils Infiltration Factors

Soil Type	Hydrologic Soil Group	Infiltration Factor
Coarse Sand	A	0.40
Fine Sand	AB	0.40
Fine Sandy Loam	B	0.40
Loam	BC	0.30
Silt Loam	C	0.20
Clay Loam	CD	0.15
Clay	D	0.10

The land use was combined into five (5) main categories (mature forest, row crops, pasture / meadow, urban lawns, and impervious areas) to be consistent with Table 3.1 in the *Stormwater Management Planning & Design Manual (MOE, 2003)*. The land use infiltration factors are shown in **Table 6** below.

Table 6: Land Use Infiltration Factor

Land Use	Infiltration Factor
Urban Lawns	0.10
Row Crops	0.10
Pasture / Meadow	0.10
Mature Forest	0.20
Impervious Areas	0.00

Land Use / Soils / Topography

The available water content (AWC), infiltration factors (IF), and crop cover coefficients (CROP COEF) are determined based on the combination of land use, soils and topography, as shown in **Table 7**.

Table 7: Model Parameters based on Land Use / Soils (existing areas)

Land Use	Soils (HSG)	AWC (mm)	IF (Land Use)	IF (Soils)	Crop Cover Coefficient			
					Dormant Season	Initial Growing Season	Middle of Growing Season	End of Growing Season
Urban Lawns	A	50	0.10	0.40	0.40	0.78	1.15	0.55
	AB	62.5		0.40				
	B	75		0.40				
	BC	100		0.30				
	C	125		0.20				
	CD	100		0.15				
	D	75		0.10				
Row Crops	A	75	0.10	0.40	0.30	0.73	1.15	0.40
	AB	112.5		0.40				
	B	150		0.40				
	BC	175		0.30				
	C	200		0.20				
	CD	200		0.15				
	D	150		0.10				
Pasture / Meadow	A	100	0.10	0.40	0.40	0.68	0.95	0.90
	AB	125		0.40				
	B	150		0.40				
	BC	200		0.30				
	C	250		0.20				
	CD	250		0.15				
	D	200		0.10				
Mature Forest	A	250	0.20	0.40	0.30	0.75	1.20	0.30
	AB	275		0.40				
	B	300		0.40				
	BC	350		0.30				
	C	400		0.20				
	CD	400		0.15				
	D	350		0.10				
Impervious Areas	A	1.57	0.00	0.00	1.00	1.00	1.00	1.00
	AB	1.57						
	B	1.57						
	BC	1.57						
	C	1.57						
	CD	1.57						
	D	1.57						

**For impervious areas, potential evapotranspiration is equal to potential evaporation (i.e. crop cover coefficient = 1.00).*

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Pre-Development Conditions

Potential Evaporation Rates (AVG. mm/d)

0.0	0.0	0.0	0.0	3.6	4.3	4.4	3.7	2.4	1.4	0.0	0.0

Surface Type	Area ID	Catchment Parameters							Infiltration Factor[1]				Crop Cover Coefficient[2]				Potential Evapotranspiration (AVG. mm/d)											
		AREA (m^2)	AREA (ha)	SOILS (HSG)	LAND USE	SOILS / LAND USE	TOPOGRAPHY	AWC[1]	IF (soils)	IF (cover)	IF (topo)	IF (Total)	Dormant Season	Initial Growing Season	Middle of Growing Season	End of Growing Season	January	February	March	April	May	June	July	August	September	October	November	December
Forest	1	2000	0.20	C/D	FOREST	C/D FOREST	HILLY	400.00	0.20	0.20	0.10	0.50	0.30	0.75	1.20	0.30	0.00	0.00	0.00	0.00	2.70	5.16	5.28	4.44	0.72	0.42	0.00	0.00
Row Crop	2	27980	2.80	C/D	ROW CROP	C/D ROW CROP	HILLY	200.00	0.20	0.10	0.10	0.40	0.30	0.73	1.15	0.40	0.00	0.00	0.00	0.00	2.63	4.95	5.06	4.26	0.96	0.42	0.00	0.00
Lawn	3	4930	0.49	C/D	LAWNS	C/D LAWNS	HILLY	100.00	0.20	0.10	0.10	0.40	0.40	0.78	1.15	0.55	0.00	0.00	0.00	0.00	2.81	4.95	5.06	4.26	1.32	0.56	0.00	0.00
Impervious	4	800	0.08	C/D	IMPERVIOUS	IMPERVIOUS	HILLY	1.57	0.00	0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00	3.60	4.30	4.40	3.70	2.40	1.40	0.00	0.00

[1] Available Water Content (AWC) and Infiltration Factors (IF) for pervious areas based on Table 3.1 from the Stormwater Management Planning and Design Manual (MOE, 2003)

[2] Crop Cover Coefficients based on Table 12 from the Food and Agriculture Organization of the United Nations (FAO), 1998, Crop Evapotranspiration - Guidelines for Computing Crop Water Requirements - FAO Irrigation and Drainage paper 56

[3] Measured Potential Evaporation Data (i.e. Lake Evaporation) from the Environment Canada Canadian Climate Normals (Ottawa CDA, 1981-2010)

Overall Pre-Development Runoff

Area ID	Area (ha)	Runoff (mm/yr)	Runoff (m^3/yr)
1	0.20	216	432
2	2.80	287	8,021
3	0.49	303	1,493
4	0.08	711	569
TOTAL	**3.57**	**294**	**10,514**

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Pre-Development Conditions

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Water Balance for Area 1: Forest

Average Monthly Results											
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.4	0.0	57.7	28.8	28.8
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	26.4	26.4
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	43.2	43.2
April	76.6	10.8	73.1	3.5	6.7	79.8	69.0	-3.8	10.8	72.9	36.5	36.5
May	78.2	85.0	78.2	0.0	0.0	78.2	-6.8	-23.2	82.4	19.0	9.5	9.5
June	96.0	146.9	96.0	0.0	0.0	96.0	-50.9	-43.5	132.9	6.7	3.3	3.3
July	91.1	159.6	91.1	0.0	0.0	91.1	-68.4	-41.4	131.0	1.6	0.8	0.8
August	87.2	124.2	87.2	0.0	0.0	87.2	-37.0	-9.8	97.0	0.0	0.0	0.0
September	88.2	33.0	88.2	0.0	0.0	88.2	55.2	57.8	27.1	3.3	1.6	1.6
October	88.7	12.2	87.8	0.9	0.6	88.4	76.1	50.1	11.5	26.7	13.4	13.4
November	73.9	1.4	58.3	15.5	12.9	71.2	69.8	12.8	1.4	57.1	28.5	28.5
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.8	0.0	48.0	24.0	24.0
ANNUAL TOTAL	**926.1**	**573.2**	**726.2**	**199.8**	**199.8**	**926.0**	**352.9**	**0.0**	**494.0**	**432.0**	**216.0**	**216.0**

Total Number of Years = *30*

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	573.2	713.0	123.1	133.9	846.9	273.7	0.0	480.7	366.2	183.1	183.1
1989	817.1	573.2	620.0	197.1	153.8	773.8	200.6	0.0	475.8	298.0	149.0	149.0
1990	976.7	573.2	777.6	199.1	232.7	1010.3	437.1	0.0	478.7	531.6	265.8	265.8
1991	820.2	573.2	619.1	201.1	204.0	823.1	250.0	0.0	445.4	377.8	188.9	188.9
1992	908.3	573.2	651.9	256.4	260.2	912.1	339.0	0.0	501.7	410.4	205.2	205.2
1993	1019.3	573.2	754.0	265.3	266.3	1020.3	447.1	0.0	495.5	524.7	262.4	262.4
1994	909.5	573.2	681.6	227.9	234.2	915.8	342.6	0.0	536.9	378.9	189.5	189.5
1995	1038.4	573.2	809.4	229.0	138.2	947.6	374.5	0.0	499.3	448.3	224.2	224.2
1996	1004.7	573.2	866.9	137.8	213.7	1080.6	507.4	0.0	507.3	573.3	286.6	286.6
1997	773.0	573.2	475.9	297.1	309.5	785.4	212.2	-10.6	435.9	360.1	180.1	180.1
1998	841.6	573.2	630.0	211.6	192.8	822.8	249.6	10.6	486.4	325.9	162.9	162.9
1999	830.5	573.2	623.3	207.2	219.8	843.1	269.9	0.0	465.8	377.3	188.6	188.6
2000	987.4	573.2	783.0	204.4	162.0	945.0	371.8	0.0	528.6	416.5	208.2	208.2
2001	753.6	573.2	580.3	173.3	213.1	793.4	220.3	0.0	462.2	331.3	165.6	165.6
2002	867.9	573.2	687.7	180.2	189.6	877.3	304.2	0.0	495.6	381.7	190.9	190.9
2003	1068.5	573.2	820.4	248.1	255.3	1075.7	502.5	0.0	501.9	573.8	286.9	286.9
2004	919.7	573.2	756.2	163.5	124.4	880.6	307.4	0.0	491.0	389.7	194.8	194.8
2005	939.6	573.2	784.9	154.7	175.8	960.7	387.5	0.0	489.8	470.8	235.4	235.4
2006	1152.0	573.2	970.6	181.4	183.1	1153.7	580.5	0.0	520.5	633.1	316.6	316.6
2007	901.0	573.2	728.8	172.2	170.0	898.8	325.7	0.0	497.1	401.7	200.9	200.9
2008	1057.6	573.2	681.6	376.0	391.5	1073.1	499.9	0.0	520.1	553.0	276.5	276.5
2009	946.5	573.2	800.3	146.2	93.4	893.7	320.6	0.0	532.3	361.4	180.7	180.7
2010	970.2	573.2	867.0	103.2	159.0	1026.0	452.8	0.0	494.2	531.7	265.9	265.9
2011	878.2	573.2	676.6	201.6	179.8	856.4	283.3	0.0	479.3	377.2	188.6	188.6
2012	807.5	573.2	596.6	210.9	147.0	743.6	170.4	0.0	459.9	283.7	141.8	141.8
2013	881.4	573.2	704.2	177.2	217.5	921.7	348.5	0.0	514.5	407.2	203.6	203.6
2014	903.1	573.2	759.5	143.6	189.0	948.5	375.3	0.0	520.6	427.9	213.9	213.9
2015	785.7	573.2	648.3	137.4	108.6	756.9	183.7	0.0	493.6	263.3	131.6	131.6
2016	917.9	573.2	656.4	261.5	262.2	918.6	345.5	0.0	464.1	454.5	227.2	227.2
2017	1268.5	573.2	1061.5	207.0	214.0	1275.5	702.3	0.0	545.6	729.9	364.9	364.9
AVERAGE	**926.1**	**573.2**	**726.2**	**199.8**	**199.8**	**926.0**	**352.9**	**0.0**	**494.0**	**432.0**	**216.0**	**216.0**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Pre-Development Conditions

Water Balance for Area 2: Row Crop

Average Monthly Results											
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.0	0.0	58.0	23.2	34.8
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	21.1	31.6
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	34.5	51.8
April	76.6	10.5	73.1	3.5	6.7	79.8	69.3	-3.7	10.4	73.1	29.2	43.9
May	78.2	82.4	78.2	0.0	0.0	78.2	-4.2	-20.0	77.8	20.4	8.2	12.3
June	96.0	141.0	96.0	0.0	0.0	96.0	-45.0	-31.5	118.8	8.7	3.5	5.2
July	91.1	152.9	91.1	0.0	0.0	91.1	-61.8	-23.8	112.4	2.5	1.0	1.5
August	87.2	120.2	87.2	0.0	0.0	87.2	-33.0	0.8	85.1	1.3	0.5	0.8
September	88.2	37.8	88.2	0.0	0.0	88.2	50.4	49.9	29.8	8.5	3.4	5.1
October	88.7	13.1	87.8	0.9	0.6	88.4	75.3	26.9	12.5	49.0	19.6	29.4
November	73.9	1.4	58.3	15.5	12.9	71.2	69.8	1.4	1.4	68.4	27.3	41.0
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.0	0.0	48.8	19.5	29.3
ANNUAL TOTAL	**926.1**	**559.3**	**726.2**	**199.8**	**199.8**	**926.0**	**366.7**	**0.0**	**448.3**	**477.8**	**191.1**	**286.7**

Total Number of Years = 30

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	559.3	713.0	123.1	133.9	846.9	287.6	0.0	438.7	408.1	163.3	244.9
1989	817.1	559.3	620.0	197.1	153.8	773.8	214.5	0.0	424.4	349.4	139.8	209.6
1990	976.7	559.3	777.6	199.1	232.7	1010.3	451.0	0.0	432.2	578.1	231.2	346.9
1991	820.2	559.3	619.1	201.1	204.0	823.1	263.8	0.0	378.6	444.5	177.8	266.7
1992	908.3	559.3	651.9	256.4	260.2	912.1	352.8	0.0	466.6	445.5	178.2	267.3
1993	1019.3	559.3	754.0	265.3	266.3	1020.3	461.0	0.0	445.8	574.5	229.8	344.7
1994	909.5	559.3	681.6	227.9	234.2	915.8	356.5	0.0	504.1	411.7	164.7	247.0
1995	1038.4	559.3	809.4	229.0	138.2	947.6	388.3	0.0	457.0	490.7	196.3	294.4
1996	1004.7	559.3	866.9	137.8	213.7	1080.6	521.3	0.0	468.8	611.8	244.7	367.1
1997	773.0	559.3	475.9	297.1	309.5	785.4	226.1	0.0	366.4	419.0	167.6	251.4
1998	841.6	559.3	630.0	211.6	192.8	822.8	263.5	0.0	437.8	385.0	154.0	231.0
1999	830.5	559.3	623.3	207.2	219.8	843.1	283.8	0.0	411.1	431.9	172.8	259.2
2000	987.4	559.3	783.0	204.4	162.0	945.0	385.7	0.0	493.2	451.8	180.7	271.1
2001	753.6	559.3	580.3	173.3	213.1	793.4	234.1	0.0	396.9	396.5	158.6	237.9
2002	867.9	559.3	687.7	180.2	189.6	877.3	318.0	0.0	441.9	435.5	174.2	261.3
2003	1068.5	559.3	820.4	248.1	255.3	1075.7	516.4	0.0	459.6	616.1	246.5	369.7
2004	919.7	559.3	756.2	163.5	124.4	880.6	321.3	0.0	441.5	439.2	175.7	263.5
2005	939.6	559.3	784.9	154.7	175.8	960.7	401.4	0.0	445.1	515.6	206.3	309.4
2006	1152.0	559.3	970.6	181.4	183.1	1153.7	594.4	0.0	489.7	664.0	265.6	398.4
2007	901.0	559.3	728.8	172.2	170.0	898.8	339.5	0.0	457.5	441.3	176.5	264.8
2008	1057.6	559.3	681.6	376.0	391.5	1073.1	513.8	0.0	480.8	592.2	236.9	355.3
2009	946.5	559.3	800.3	146.2	93.4	893.7	334.4	0.0	497.6	396.2	158.5	237.7
2010	970.2	559.3	867.0	103.2	159.0	1026.0	466.7	0.0	455.0	570.9	228.4	342.6
2011	878.2	559.3	676.6	201.6	179.8	856.4	297.1	0.0	425.9	430.5	172.2	258.3
2012	807.5	559.3	596.6	210.9	147.0	743.6	184.3	0.0	400.4	343.2	137.3	205.9
2013	881.4	559.3	704.2	177.2	217.5	921.7	362.4	0.0	473.7	448.0	179.2	268.8
2014	903.1	559.3	759.5	143.6	189.0	948.5	389.2	0.0	480.8	467.7	187.1	280.6
2015	785.7	559.3	648.3	137.4	108.6	756.9	197.6	0.0	450.4	306.5	122.6	183.9
2016	917.9	559.3	656.4	261.5	262.2	918.6	359.3	0.0	413.6	505.0	202.0	303.0
2017	1268.5	559.3	1061.5	207.0	214.0	1275.5	716.2	0.0	513.4	762.0	304.8	457.2
AVERAGE	**926.1**	**559.3**	**726.2**	**199.8**	**199.8**	**926.0**	**366.7**	**0.0**	**448.3**	**477.8**	**191.1**	**286.7**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Pre-Development Conditions

Water Balance for Area 3: Lawn

Average Monthly Results												
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.0	0.0	58.0	23.2	34.8
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	21.1	31.6
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	34.5	51.8
April	76.6	11.2	73.1	3.5	6.7	79.8	68.6	-3.9	11.0	72.7	29.1	43.6
May	78.2	86.6	78.2	0.0	0.0	78.2	-8.4	-18.1	76.9	19.4	7.8	11.6
June	96.0	141.6	96.0	0.0	0.0	96.0	-45.6	-19.3	105.0	10.3	4.1	6.2
July	91.1	152.9	91.1	0.0	0.0	91.1	-61.8	-9.7	96.7	4.1	1.7	2.5
August	87.2	121.8	87.2	0.0	0.0	87.2	-34.6	3.8	77.1	6.2	2.5	3.7
September	88.2	46.5	88.2	0.0	0.0	88.2	41.7	36.7	35.7	15.8	6.3	9.5
October	88.7	17.6	87.8	0.9	0.6	88.4	70.8	9.9	17.0	61.4	24.6	36.8
November	73.9	1.9	58.3	15.5	12.9	71.2	69.3	0.6	1.9	68.8	27.5	41.3
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.0	0.0	48.8	19.5	29.3
ANNUAL TOTAL	**926.1**	**580.0**	**726.2**	**199.8**	**199.8**	**926.0**	**346.0**	**0.0**	**421.4**	**504.7**	**201.9**	**302.8**

Total Number of Years = *30*

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	580.0	713.0	123.1	133.9	846.9	266.8	0.0	414.9	432.0	172.8	259.2
1989	817.1	580.0	620.0	197.1	153.8	773.8	193.8	0.0	397.5	376.3	150.5	225.8
1990	976.7	580.0	777.6	199.1	232.7	1010.3	430.2	0.0	417.5	592.8	237.1	355.7
1991	820.2	580.0	619.1	201.1	204.0	823.1	243.1	0.0	337.0	486.1	194.4	291.7
1992	908.3	580.0	651.9	256.4	260.2	912.1	332.1	0.0	451.5	460.6	184.2	276.4
1993	1019.3	580.0	754.0	265.3	266.3	1020.3	440.2	0.0	414.5	605.8	242.3	363.5
1994	909.5	580.0	681.6	227.9	234.2	915.8	335.8	0.0	482.7	433.1	173.2	259.8
1995	1038.4	580.0	809.4	229.0	138.2	947.6	367.6	0.0	422.0	525.6	210.2	315.4
1996	1004.7	580.0	866.9	137.8	213.7	1080.6	500.5	0.0	442.4	638.2	255.3	382.9
1997	773.0	580.0	475.9	297.1	309.5	785.4	205.4	0.0	324.0	461.4	184.5	276.8
1998	841.6	580.0	630.0	211.6	192.8	822.8	242.8	0.0	407.2	415.6	166.3	249.4
1999	830.5	580.0	623.3	207.2	219.8	843.1	263.0	0.0	378.3	464.8	185.9	278.9
2000	987.4	580.0	783.0	204.4	162.0	945.0	365.0	0.0	478.8	466.2	186.5	279.7
2001	753.6	580.0	580.3	173.3	213.1	793.4	213.4	0.0	351.4	442.0	176.8	265.2
2002	867.9	580.0	687.7	180.2	189.6	877.3	297.3	0.0	402.0	475.4	190.1	285.2
2003	1068.5	580.0	820.4	248.1	255.3	1075.7	495.6	0.0	439.9	635.8	254.3	381.5
2004	919.7	580.0	756.2	163.5	124.4	880.6	300.6	0.0	411.4	469.2	187.7	281.5
2005	939.6	580.0	784.9	154.7	175.8	960.7	380.7	0.0	416.9	543.8	217.5	326.3
2006	1152.0	580.0	970.6	181.4	183.1	1153.7	573.6	0.0	468.7	685.0	274.0	411.0
2007	901.0	580.0	728.8	172.2	170.0	898.8	318.8	0.0	421.4	477.4	191.0	286.5
2008	1057.6	580.0	681.6	376.0	391.5	1073.1	493.0	0.0	461.1	612.0	244.8	367.2
2009	946.5	580.0	800.3	146.2	93.4	893.7	313.7	0.0	477.2	416.6	166.6	250.0
2010	970.2	580.0	867.0	103.2	159.0	1026.0	445.9	0.0	434.0	592.0	236.8	355.2
2011	878.2	580.0	676.6	201.6	179.8	856.4	276.4	0.0	396.3	460.2	184.1	276.1
2012	807.5	580.0	596.6	210.9	147.0	743.6	163.5	0.0	363.9	379.7	151.9	227.8
2013	881.4	580.0	704.2	177.2	217.5	921.7	341.7	0.0	454.2	467.5	187.0	280.5
2014	903.1	580.0	759.5	143.6	189.0	948.5	368.4	0.0	461.0	487.5	195.0	292.5
2015	785.7	580.0	648.3	137.4	108.6	756.9	176.9	0.0	424.2	332.7	133.1	199.6
2016	917.9	580.0	656.4	261.5	262.2	918.6	338.6	0.0	389.6	529.0	211.6	317.4
2017	1268.5	580.0	1061.5	207.0	214.0	1275.5	695.4	0.0	500.1	775.4	310.2	465.2
AVERAGE	**926.1**	**580.0**	**726.2**	**199.8**	**199.8**	**926.0**	**346.0**	**0.0**	**421.4**	**504.7**	**201.9**	**302.8**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Pre-Development Conditions

Water Balance for Area 4: Impervious

Average Monthly Results											
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.0	0.0	58.0	0.0	58.0
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	0.0	52.7
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	0.0	86.4
April	76.6	14.4	73.1	3.5	6.7	79.8	65.4	-1.0	8.0	72.9	0.0	72.9
May	78.2	102.1	78.2	0.0	0.0	78.2	-23.9	0.0	35.9	42.4	0.0	42.4
June	96.0	127.0	96.0	0.0	0.0	96.0	-31.0	-0.1	43.3	52.7	0.0	52.7
July	91.1	133.0	91.1	0.0	0.0	91.1	-41.8	-0.2	40.6	50.7	0.0	50.7
August	87.2	111.4	87.2	0.0	0.0	87.2	-24.2	-0.1	33.4	53.9	0.0	53.9
September	88.2	72.4	88.2	0.0	0.0	88.2	15.8	0.5	28.1	59.5	0.0	59.5
October	88.7	40.8	87.8	0.9	0.6	88.4	47.6	0.1	22.2	66.0	0.0	66.0
November	73.9	4.7	58.3	15.5	12.9	71.2	66.5	0.8	3.3	67.1	0.0	67.1
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.0	0.0	48.8	0.0	48.8
ANNUAL TOTAL	**926.1**	**605.8**	**726.2**	**199.8**	**199.8**	**926.0**	**320.3**	**0.0**	**214.9**	**711.2**	**0.0**	**711.2**

Total Number of Years = 30

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	605.8	713.0	123.1	133.9	846.9	241.1	0.0	205.8	641.1	0.0	641.1
1989	817.1	605.8	620.0	197.1	153.8	773.8	168.0	0.0	180.5	593.3	0.0	593.3
1990	976.7	605.8	777.6	199.1	232.7	1010.3	404.5	0.0	207.6	802.7	0.0	802.7
1991	820.2	605.8	619.1	201.1	204.0	823.1	217.4	0.0	191.6	631.5	0.0	631.5
1992	908.3	605.8	651.9	256.4	260.2	912.1	306.4	0.0	211.4	700.8	0.0	700.8
1993	1019.3	605.8	754.0	265.3	266.3	1020.3	414.5	0.0	243.6	776.7	0.0	776.7
1994	909.5	605.8	681.6	227.9	234.2	915.8	310.1	0.0	224.9	690.9	0.0	690.9
1995	1038.4	605.8	809.4	229.0	138.2	947.6	341.9	0.0	197.5	750.2	0.0	750.2
1996	1004.7	605.8	866.9	137.8	213.7	1080.6	474.8	0.0	220.2	860.4	0.0	860.4
1997	773.0	605.8	475.9	297.1	309.5	785.4	179.7	0.0	178.1	607.3	0.0	607.3
1998	841.6	605.8	630.0	211.6	192.8	822.8	217.1	0.0	209.4	613.4	0.0	613.4
1999	830.5	605.8	623.3	207.2	219.8	843.1	237.3	0.0	192.7	650.4	0.0	650.4
2000	987.4	605.8	783.0	204.4	162.0	945.0	339.3	0.0	240.8	704.2	0.0	704.2
2001	753.6	605.8	580.3	173.3	213.1	793.4	187.7	0.0	195.0	598.5	0.0	598.5
2002	867.9	605.8	687.7	180.2	189.6	877.3	271.6	0.0	194.6	682.8	0.0	682.8
2003	1068.5	605.8	820.4	248.1	255.3	1075.7	469.9	0.0	233.9	841.8	0.0	841.8
2004	919.7	605.8	756.2	163.5	124.4	880.6	274.9	0.0	220.1	660.5	0.0	660.5
2005	939.6	605.8	784.9	154.7	175.8	960.7	354.9	0.0	218.2	742.5	0.0	742.5
2006	1152.0	605.8	970.6	181.4	183.1	1153.7	547.9	0.0	241.1	912.6	0.0	912.6
2007	901.0	605.8	728.8	172.2	170.0	898.8	293.1	0.0	205.7	693.1	0.0	693.1
2008	1057.6	605.8	681.6	376.0	391.5	1073.1	467.3	0.0	234.1	838.9	0.0	838.9
2009	946.5	605.8	800.3	146.2	93.4	893.7	288.0	0.0	256.2	637.5	0.0	637.5
2010	970.2	605.8	867.0	103.2	159.0	1026.0	420.2	0.0	245.4	780.5	0.0	780.5
2011	878.2	605.8	676.6	201.6	179.8	856.4	250.7	0.0	217.9	638.6	0.0	638.6
2012	807.5	605.8	596.6	210.9	147.0	743.6	137.8	0.0	208.6	535.0	0.0	535.0
2013	881.4	605.8	704.2	177.2	217.5	921.7	316.0	0.0	231.7	690.0	0.0	690.0
2014	903.1	605.8	759.5	143.6	189.0	948.5	342.7	0.0	230.4	718.0	0.0	718.0
2015	785.7	605.8	648.3	137.4	108.6	756.9	151.2	0.0	200.5	556.4	0.0	556.4
2016	917.9	605.8	656.4	261.5	262.2	918.6	312.9	0.0	171.9	746.8	0.0	746.8
2017	1268.5	605.8	1061.5	207.0	214.0	1275.5	669.7	0.0	236.8	1038.7	0.0	1038.7
AVERAGE	**926.1**	**605.8**	**726.2**	**199.8**	**199.8**	**926.0**	**320.3**	**0.0**	**214.9**	**711.2**	**0.0**	**711.2**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Post-Development Conditions

Potential Evaporation Rates (AVG. mm/d)

0.0	0.0	0.0	0.0	3.6	4.3	4.4	3.7	2.4	1.4	0.0	0.0

Surface Type	Area ID	Catchment Parameters							Infiltration Factor[1]				Crop Cover Coefficient[2]				Potential Evapotranspiration (AVG. mm/d)											
		AREA (m^2)	AREA (ha)	SOILS (HSG)	LAND USE	SOILS / LAND USE	TOPOGRAPHY	AWC[1]	IF (soils)	IF (cover)	IF (topo)	IF (Total)	Dormant Season	Initial Growing Season	Middle of Growing Season	End of Growing Season	January	February	March	April	May	June	July	August	September	October	November	December
Impervious	1	2322	0.23	C/D	IMPERVIOUS	IMPERVIOUS	HILLY	1.57	0.00	0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00	3.60	4.30	4.40	3.70	2.40	1.40	0.00	0.00
Lawn	2	5578	0.56	C/D	LAWNS	C/D LAWNS	HILLY	100.00	0.20	0.10	0.10	0.40	0.40	0.78	1.15	0.55	0.00	0.00	0.00	0.00	2.81	4.95	5.06	4.26	1.32	0.56	0.00	0.00
Forest	3	1200	0.12	C/D	FOREST	C/D FOREST	HILLY	400.00	0.20	0.20	0.10	0.50	0.30	0.75	1.20	0.30	0.00	0.00	0.00	0.00	2.70	5.16	5.28	4.44	0.72	0.42	0.00	0.00

[1] Available Water Content (AWC) and Infiltration Factors (IF) for pervious areas based on Table 3.1 from the Stormwater Management Planning and Design Manual (MOE, 2003)

[2] Crop Cover Coefficients based on Table 12 from the Food and Agriculture Organization of the United Nations (FAO), 1998, Crop Evapotranspiration - Guidelines for Computing Crop Water Requirements - FAO Irrigation and Drainage paper 56

[3] Measured Potential Evaporation Data (i.e. Lake Evaporation) from the Environment Canada Canadian Climate Normals (Ottawa CDA, 1981-2010)

Overall Post-Development Runoff

Area ID	Area (ha)	Runoff (mm/yr)	Runoff (m^3/yr)
1	0.23	711	1,651
2	0.56	303	1,689
3	0.12	216	259
TOTAL	**0.91**	**396**	**3,600**

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Post-Development Conditions

Water Balance for Area 1: Impervious

Average Monthly Results											
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.0	0.0	58.0	0.0	58.0
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	0.0	52.7
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	0.0	86.4
April	76.6	14.4	73.1	3.5	6.7	79.8	65.4	-1.0	8.0	72.9	0.0	72.9
May	78.2	102.1	78.2	0.0	0.0	78.2	-23.9	0.0	35.9	42.4	0.0	42.4
June	96.0	127.0	96.0	0.0	0.0	96.0	-31.0	-0.1	43.3	52.7	0.0	52.7
July	91.1	133.0	91.1	0.0	0.0	91.1	-41.8	-0.2	40.6	50.7	0.0	50.7
August	87.2	111.4	87.2	0.0	0.0	87.2	-24.2	-0.1	33.4	53.9	0.0	53.9
September	88.2	72.4	88.2	0.0	0.0	88.2	15.8	0.5	28.1	59.5	0.0	59.5
October	88.7	40.8	87.8	0.9	0.6	88.4	47.6	0.1	22.2	66.0	0.0	66.0
November	73.9	4.7	58.3	15.5	12.9	71.2	66.5	0.8	3.3	67.1	0.0	67.1
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.0	0.0	48.8	0.0	48.8
ANNUAL TOTAL	**926.1**	**605.8**	**726.2**	**199.8**	**199.8**	**926.0**	**320.3**	**0.0**	**214.9**	**711.2**	**0.0**	**711.2**

Total Number of Years = 30

**Based on capturing the first 18 mm of runoff from May - October*

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	605.8	713.0	123.1	133.9	846.9	241.1	0.0	205.8	641.1	0.0	641.1
1989	817.1	605.8	620.0	197.1	153.8	773.8	168.0	0.0	180.5	593.3	0.0	593.3
1990	976.7	605.8	777.6	199.1	232.7	1010.3	404.5	0.0	207.6	802.7	0.0	802.7
1991	820.2	605.8	619.1	201.1	204.0	823.1	217.4	0.0	191.6	631.5	0.0	631.5
1992	908.3	605.8	651.9	256.4	260.2	912.1	306.4	0.0	211.4	700.8	0.0	700.8
1993	1019.3	605.8	754.0	265.3	266.3	1020.3	414.5	0.0	243.6	776.7	0.0	776.7
1994	909.5	605.8	681.6	227.9	234.2	915.8	310.1	0.0	224.9	690.9	0.0	690.9
1995	1038.4	605.8	809.4	229.0	138.2	947.6	341.9	0.0	197.5	750.2	0.0	750.2
1996	1004.7	605.8	866.9	137.8	213.7	1080.6	474.8	0.0	220.2	860.4	0.0	860.4
1997	773.0	605.8	475.9	297.1	309.5	785.4	179.7	0.0	178.1	607.3	0.0	607.3
1998	841.6	605.8	630.0	211.6	192.8	822.8	217.1	0.0	209.4	613.4	0.0	613.4
1999	830.5	605.8	623.3	207.2	219.8	843.1	237.3	0.0	192.7	650.4	0.0	650.4
2000	987.4	605.8	783.0	204.4	162.0	945.0	339.3	0.0	240.8	704.2	0.0	704.2
2001	753.6	605.8	580.3	173.3	213.1	793.4	187.7	0.0	195.0	598.5	0.0	598.5
2002	867.9	605.8	687.7	180.2	189.6	877.3	271.6	0.0	194.6	682.8	0.0	682.8
2003	1068.5	605.8	820.4	248.1	255.3	1075.7	469.9	0.0	233.9	841.8	0.0	841.8
2004	919.7	605.8	756.2	163.5	124.4	880.6	274.9	0.0	220.1	660.5	0.0	660.5
2005	939.6	605.8	784.9	154.7	175.8	960.7	354.9	0.0	218.2	742.5	0.0	742.5
2006	1152.0	605.8	970.6	181.4	183.1	1153.7	547.9	0.0	241.1	912.6	0.0	912.6
2007	901.0	605.8	728.8	172.2	170.0	898.8	293.1	0.0	205.7	693.1	0.0	693.1
2008	1057.6	605.8	681.6	376.0	391.5	1073.1	467.3	0.0	234.1	838.9	0.0	838.9
2009	946.5	605.8	800.3	146.2	93.4	893.7	288.0	0.0	256.2	637.5	0.0	637.5
2010	970.2	605.8	867.0	103.2	159.0	1026.0	420.2	0.0	245.4	780.5	0.0	780.5
2011	878.2	605.8	676.6	201.6	179.8	856.4	250.7	0.0	217.9	638.6	0.0	638.6
2012	807.5	605.8	596.6	210.9	147.0	743.6	137.8	0.0	208.6	535.0	0.0	535.0
2013	881.4	605.8	704.2	177.2	217.5	921.7	316.0	0.0	231.7	690.0	0.0	690.0
2014	903.1	605.8	759.5	143.6	189.0	948.5	342.7	0.0	230.4	718.0	0.0	718.0
2015	785.7	605.8	648.3	137.4	108.6	756.9	151.2	0.0	200.5	556.4	0.0	556.4
2016	917.9	605.8	656.4	261.5	262.2	918.6	312.9	0.0	171.9	746.8	0.0	746.8
2017	1268.5	605.8	1061.5	207.0	214.0	1275.5	669.7	0.0	236.8	1038.7	0.0	1038.7
AVERAGE	**926.1**	**605.8**	**726.2**	**199.8**	**199.8**	**926.0**	**320.3**	**0.0**	**214.9**	**711.2**	**0.0**	**711.2**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Post-Development Conditions

Water Balance for Area 2: Lawn

Average Monthly Results											
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.0	0.0	58.0	23.2	34.8
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	21.1	31.6
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	34.5	51.8
April	76.6	11.2	73.1	3.5	6.7	79.8	68.6	-3.9	11.0	72.7	29.1	43.6
May	78.2	86.6	78.2	0.0	0.0	78.2	-8.4	-18.1	76.9	19.4	7.8	11.6
June	96.0	141.6	96.0	0.0	0.0	96.0	-45.6	-19.3	105.0	10.3	4.1	6.2
July	91.1	152.9	91.1	0.0	0.0	91.1	-61.8	-9.7	96.7	4.1	1.7	2.5
August	87.2	121.8	87.2	0.0	0.0	87.2	-34.6	3.8	77.1	6.2	2.5	3.7
September	88.2	46.5	88.2	0.0	0.0	88.2	41.7	36.7	35.7	15.8	6.3	9.5
October	88.7	17.6	87.8	0.9	0.6	88.4	70.8	9.9	17.0	61.4	24.6	36.8
November	73.9	1.9	58.3	15.5	12.9	71.2	69.3	0.6	1.9	68.8	27.5	41.3
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.0	0.0	48.8	19.5	29.3
ANNUAL TOTAL	**926.1**	**580.0**	**726.2**	**199.8**	**199.8**	**926.0**	**346.0**	**0.0**	**421.4**	**504.7**	**201.9**	**302.8**

Total Number of Years = 30

**Based on capturing the first 18 mm of runoff from May - October*

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	580.0	713.0	123.1	133.9	846.9	266.8	0.0	414.9	432.0	172.8	259.2
1989	817.1	580.0	620.0	197.1	153.8	773.8	193.8	0.0	397.5	376.3	150.5	225.8
1990	976.7	580.0	777.6	199.1	232.7	1010.3	430.2	0.0	417.5	592.8	237.1	355.7
1991	820.2	580.0	619.1	201.1	204.0	823.1	243.1	0.0	337.0	486.1	194.4	291.7
1992	908.3	580.0	651.9	256.4	260.2	912.1	332.1	0.0	451.5	460.6	184.2	276.4
1993	1019.3	580.0	754.0	265.3	266.3	1020.3	440.2	0.0	414.5	605.8	242.3	363.5
1994	909.5	580.0	681.6	227.9	234.2	915.8	335.8	0.0	482.7	433.1	173.2	259.8
1995	1038.4	580.0	809.4	229.0	138.2	947.6	367.6	0.0	422.0	525.6	210.2	315.4
1996	1004.7	580.0	866.9	137.8	213.7	1080.6	500.5	0.0	442.4	638.2	255.3	382.9
1997	773.0	580.0	475.9	297.1	309.5	785.4	205.4	0.0	324.0	461.4	184.5	276.8
1998	841.6	580.0	630.0	211.6	192.8	822.8	242.8	0.0	407.2	415.6	166.3	249.4
1999	830.5	580.0	623.3	207.2	219.8	843.1	263.0	0.0	378.3	464.8	185.9	278.9
2000	987.4	580.0	783.0	204.4	162.0	945.0	365.0	0.0	478.8	466.2	186.5	279.7
2001	753.6	580.0	580.3	173.3	213.1	793.4	213.4	0.0	351.4	442.0	176.8	265.2
2002	867.9	580.0	687.7	180.2	189.6	877.3	297.3	0.0	402.0	475.4	190.1	285.2
2003	1068.5	580.0	820.4	248.1	255.3	1075.7	495.6	0.0	439.9	635.8	254.3	381.5
2004	919.7	580.0	756.2	163.5	124.4	880.6	300.6	0.0	411.4	469.2	187.7	281.5
2005	939.6	580.0	784.9	154.7	175.8	960.7	380.7	0.0	416.9	543.8	217.5	326.3
2006	1152.0	580.0	970.6	181.4	183.1	1153.7	573.6	0.0	468.7	685.0	274.0	411.0
2007	901.0	580.0	728.8	172.2	170.0	898.8	318.8	0.0	421.4	477.4	191.0	286.5
2008	1057.6	580.0	681.6	376.0	391.5	1073.1	493.0	0.0	461.1	612.0	244.8	367.2
2009	946.5	580.0	800.3	146.2	93.4	893.7	313.7	0.0	477.2	416.6	166.6	250.0
2010	970.2	580.0	867.0	103.2	159.0	1026.0	445.9	0.0	434.0	592.0	236.8	355.2
2011	878.2	580.0	676.6	201.6	179.8	856.4	276.4	0.0	396.3	460.2	184.1	276.1
2012	807.5	580.0	596.6	210.9	147.0	743.6	163.5	0.0	363.9	379.7	151.9	227.8
2013	881.4	580.0	704.2	177.2	217.5	921.7	341.7	0.0	454.2	467.5	187.0	280.5
2014	903.1	580.0	759.5	143.6	189.0	948.5	368.4	0.0	461.0	487.5	195.0	292.5
2015	785.7	580.0	648.3	137.4	108.6	756.9	176.9	0.0	424.2	332.7	133.1	199.6
2016	917.9	580.0	656.4	261.5	262.2	918.6	338.6	0.0	389.6	529.0	211.6	317.4
2017	1268.5	580.0	1061.5	207.0	214.0	1275.5	695.4	0.0	500.1	775.4	310.2	465.2
AVERAGE	**926.1**	**580.0**	**726.2**	**199.8**	**199.8**	**926.0**	**346.0**	**0.0**	**421.4**	**504.7**	**201.9**	**302.8**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

4386 Rideau Valley Drive - Stinson Lands (121153)
Oxbow Water Balance Calculations
Post-Development Conditions

Water Balance for Area 3: Forest

Average Monthly Results												
Month	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
January	63.3	0.0	10.9	52.4	47.1	58.0	58.0	0.4	0.0	57.7	28.8	28.8
February	51.9	0.0	10.1	41.8	42.7	52.7	52.7	0.0	0.0	52.7	26.4	26.4
March	60.0	0.0	24.8	35.2	61.5	86.4	86.4	0.0	0.0	86.4	43.2	43.2
April	76.6	10.8	73.1	3.5	6.7	79.8	69.0	-3.8	10.8	72.9	36.5	36.5
May	78.2	85.0	78.2	0.0	0.0	78.2	-6.8	-23.2	82.4	19.0	9.5	9.5
June	96.0	146.9	96.0	0.0	0.0	96.0	-50.9	-43.5	132.9	6.7	3.3	3.3
July	91.1	159.6	91.1	0.0	0.0	91.1	-68.4	-41.4	131.0	1.6	0.8	0.8
August	87.2	124.2	87.2	0.0	0.0	87.2	-37.0	-9.8	97.0	0.0	0.0	0.0
September	88.2	33.0	88.2	0.0	0.0	88.2	55.2	57.8	27.1	3.3	1.6	1.6
October	88.7	12.2	87.8	0.9	0.6	88.4	76.1	50.1	11.5	26.7	13.4	13.4
November	73.9	1.4	58.3	15.5	12.9	71.2	69.8	12.8	1.4	57.1	28.5	28.5
December	71.0	0.0	20.5	50.5	28.3	48.8	48.8	0.8	0.0	48.0	24.0	24.0
ANNUAL TOTAL	**926.1**	**573.2**	**726.2**	**199.8**	**199.8**	**926.0**	**352.9**	**0.0**	**494.0**	**432.0**	**216.0**	**216.0**

Total Number of Years = *30*

Average Annual Results												
Year	**Precip.**	**PET**	**Rain**	**Snow**	**Snowmelt**	**Water Input**	**W-PET**	**ΔSoil Water**	**AET**	**Surplus**	**Infiltration**	**Runoff**
1988	836.1	573.2	713.0	123.1	133.9	846.9	273.7	0.0	480.7	366.2	183.1	183.1
1989	817.1	573.2	620.0	197.1	153.8	773.8	200.6	0.0	475.8	298.0	149.0	149.0
1990	976.7	573.2	777.6	199.1	232.7	1010.3	437.1	0.0	478.7	531.6	265.8	265.8
1991	820.2	573.2	619.1	201.1	204.0	823.1	250.0	0.0	445.4	377.8	188.9	188.9
1992	908.3	573.2	651.9	256.4	260.2	912.1	339.0	0.0	501.7	410.4	205.2	205.2
1993	1019.3	573.2	754.0	265.3	266.3	1020.3	447.1	0.0	495.5	524.7	262.4	262.4
1994	909.5	573.2	681.6	227.9	234.2	915.8	342.6	0.0	536.9	378.9	189.5	189.5
1995	1038.4	573.2	809.4	229.0	138.2	947.6	374.5	0.0	499.3	448.3	224.2	224.2
1996	1004.7	573.2	866.9	137.8	213.7	1080.6	507.4	0.0	507.3	573.3	286.6	286.6
1997	773.0	573.2	475.9	297.1	309.5	785.4	212.2	-10.6	435.9	360.1	180.1	180.1
1998	841.6	573.2	630.0	211.6	192.8	822.8	249.6	10.6	486.4	325.9	162.9	162.9
1999	830.5	573.2	623.3	207.2	219.8	843.1	269.9	0.0	465.8	377.3	188.6	188.6
2000	987.4	573.2	783.0	204.4	162.0	945.0	371.8	0.0	528.6	416.5	208.2	208.2
2001	753.6	573.2	580.3	173.3	213.1	793.4	220.3	0.0	462.2	331.3	165.6	165.6
2002	867.9	573.2	687.7	180.2	189.6	877.3	304.2	0.0	495.6	381.7	190.9	190.9
2003	1068.5	573.2	820.4	248.1	255.3	1075.7	502.5	0.0	501.9	573.8	286.9	286.9
2004	919.7	573.2	756.2	163.5	124.4	880.6	307.4	0.0	491.0	389.7	194.8	194.8
2005	939.6	573.2	784.9	154.7	175.8	960.7	387.5	0.0	489.8	470.8	235.4	235.4
2006	1152.0	573.2	970.6	181.4	183.1	1153.7	580.5	0.0	520.5	633.1	316.6	316.6
2007	901.0	573.2	728.8	172.2	170.0	898.8	325.7	0.0	497.1	401.7	200.9	200.9
2008	1057.6	573.2	681.6	376.0	391.5	1073.1	499.9	0.0	520.1	553.0	276.5	276.5
2009	946.5	573.2	800.3	146.2	93.4	893.7	320.6	0.0	532.3	361.4	180.7	180.7
2010	970.2	573.2	867.0	103.2	159.0	1026.0	452.8	0.0	494.2	531.7	265.9	265.9
2011	878.2	573.2	676.6	201.6	179.8	856.4	283.3	0.0	479.3	377.2	188.6	188.6
2012	807.5	573.2	596.6	210.9	147.0	743.6	170.4	0.0	459.9	283.7	141.8	141.8
2013	881.4	573.2	704.2	177.2	217.5	921.7	348.5	0.0	514.5	407.2	203.6	203.6
2014	903.1	573.2	759.5	143.6	189.0	948.5	375.3	0.0	520.6	427.9	213.9	213.9
2015	785.7	573.2	648.3	137.4	108.6	756.9	183.7	0.0	493.6	263.3	131.6	131.6
2016	917.9	573.2	656.4	261.5	262.2	918.6	345.5	0.0	464.1	454.5	227.2	227.2
2017	1268.5	573.2	1061.5	207.0	214.0	1275.5	702.3	0.0	545.6	729.9	364.9	364.9
AVERAGE	**926.1**	**573.2**	**726.2**	**199.8**	**199.8**	**926.0**	**352.9**	**0.0**	**494.0**	**432.0**	**216.0**	**216.0**

PRECIP	Total Precipitation
PET	Potential Evapotranspiration
W	Water Input (Rain + Snowmelt)
Soil Water (SW)	Available Water in the Soil Moisture Storage Zone
ΔSoil Water	Change in Soil Water
AET	Actual Evapotranspiration

The water balance calculations are conducted on a daily time step
All units in mm

Appendix D
Sanitary Sewer Design Sheets and Sanitary Calculations

SANITARY SEWER DESIGN SHEET (FUTURE GROWTH)

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #:	121153
Project Name:	Stinson Lands Subdivision
Date Prepared:	1/11/2023
Date Revised:	12/10/2024
Input By:	Brendan Rundle
Reviewed By:	Sam Bahia
Drawing Reference:	121153-GP

Legend:
- 🟨 PROJECT SPECIFIC INFO
- USER DESIGN INPUT
- 🟦 CUMILATIVE CELL
- 🟩 CALCULATED DESIGN CELL OUTPUT
- CALCULATED ANNUAL CELL OUTPUT
- 🟪 CALCULATED RARE CELL OUTPUT
- 🟧 USER AS-BUILT INPUT

LOCATION				DEMAND																											DESIGN CAPACITY							
				RESIDENTIAL FLOW													INDUSTRIAL / COMMERICAL / INSTITUTIONAL FLOW							EXTRANOUS FLOW AREA METHOD				TOTAL DESIGN FLOW			PROPOSED SEWER PIPE SIZING / DESIGN							
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.)	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)	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	101	103	4	10			0.041	0.041	3.67	0.13	0.48	3.00	0.28	0.650	0.650		0.000	0.00	1.00	0.000	0.00	0.00	0.650	0.21	0.20	0.36	0.70	0.48	0.639	84.5	200 PVC	0.203	0.013	1.30	39.0	1.20	1.8%
Street 1	2	103	105		12			0.032	0.073	3.62	0.24	0.86	2.97	0.50	0.370	1.020		0.000	0.00	1.00	0.000	0.00	0.00	1.020	0.34	0.31	0.56	1.19	0.81	1.062	46.0	200 PVC	0.203	0.013	1.30	39.0	1.20	3.1%
Street 1	3	105	107	2	3			0.015	0.088	3.61	0.28	1.03	2.96	0.60	0.210	1.230		0.000	0.00	1.00	0.000	0.00	0.00	1.230	0.41	0.37	0.68	1.43	0.97	1.278	29.8	200 PVC	0.203	0.013	1.50	41.9	1.29	3.4%
Street 1	4	109	107	3				0.010	0.098	3.60	0.32	1.14	2.95	0.67	0.200	1.430		0.000	0.00	1.00	0.000	0.00	0.00	1.430	0.47	0.43	0.79	1.62	1.10	1.456	18.5	200 PVC	0.203	0.013	1.50	41.9	1.29	3.9%
Street 1	5	109	111	1				0.003	0.102	3.59	0.33	1.18	2.95	0.69	0.100	1.530		0.000	0.00	1.00	0.000	0.00	0.00	1.530	0.50	0.46	0.84	1.69	1.15	1.533	16.4	200 PVC	0.203	0.013	1.50	41.9	1.29	4.0%
Street 2	6	113	115	5				0.017	0.017	3.71	0.06	0.20	3.03	0.12	0.320	0.320		0.000	0.00	1.00	0.000	0.00	0.00	0.320	0.11	0.10	0.18	0.31	0.22	0.295	59.7	200 PVC	0.203	0.013	0.50	24.2	0.75	1.3%
Street 2	7	115	117	2				0.007	0.024	3.70	0.08	0.29	3.02	0.17	0.190	0.510		0.000	0.00	1.00	0.000	0.00	0.00	0.510	0.17	0.15	0.28	0.45	0.32	0.447	10.9	200 PVC	0.203	0.013	0.50	24.2	0.75	1.9%
Street 2	8	117	119	8				0.027	0.051	3.65	0.17	0.60	2.99	0.35	0.410	0.920		0.000	0.00	1.00	0.000	0.00	0.00	0.920	0.30	0.28	0.51	0.91	0.63	0.859	47.1	200 PVC	0.203	0.013	0.50	24.2	0.75	3.7%
Street 2	9	119	121	5				0.017	0.068	3.63	0.22	0.80	2.97	0.47	0.290	1.210		0.000	0.00	1.00	0.000	0.00	0.00	1.210	0.40	0.36	0.67	1.20	0.83	1.133	30.7	200 PVC	0.203	0.013	0.50	24.2	0.75	5.0%
Street 2	10	121	123	8				0.027	0.095	3.60	0.31	1.11	2.95	0.65	0.440	1.650		0.000	0.00	1.00	0.000	0.00	0.00	1.650	0.54	0.50	0.91	1.66	1.15	1.558	51.7	200 PVC	0.203	0.013	0.50	24.2	0.75	6.8%
Street 2	11	123	125	7				0.024	0.119	3.58	0.39	1.38	2.93	0.81	0.390	2.040		0.000	0.00	1.00	0.000	0.00	0.00	2.040	0.67	0.61	1.12	0.91	0.63	0.859	53.1	200 PVC	0.203	0.013	0.55	25.4	0.78	3.6%
Street 2	12	129	127	9				0.031	0.031	3.68	0.10	0.37	3.01	0.21	0.870	0.870		0.000	0.00	1.00	0.000	0.00	0.00	0.870	0.29	0.26	0.48	0.65	0.47	0.692	65.4	200 PVC	0.203	0.013	0.35	20.2	0.62	3.2%
Street 2	13	127	125	2			0.250	0.008	0.038	3.67	0.12	0.45	3.00	0.27	0.400	1.270		0.000	0.00	1.00	0.000	0.00	0.00	1.270	0.42	0.38	0.70	0.87	0.65	0.964	30.3	200 PVC	0.203	0.013	0.35	20.2	0.62	4.3%
Street 3	14	125	131					0.000	0.157	3.55	0.51	1.81	2.91	1.06	0.050	3.360		0.000	0.00	1.00	0.000	0.00	0.00	3.360	1.11	1.01	1.85	2.92	2.07	2.907	28.4	250 PVC	0.254	0.013	0.30	34.0	0.67	8.6%
Street 3	15	131	111	4	2			0.019	0.176	3.53	0.57	2.02	2.90	1.18	0.370	3.730		0.000	0.00	1.00	0.000	0.00	0.00	3.730	1.23	1.12	2.05	3.25	2.30	3.235	69.6	250 PVC	0.254	0.013	0.25	31.0	0.61	10.5%
Street 3	16	111	133	2	6			0.023	0.301	3.46	0.97	3.37	2.85	1.98	0.400	5.660		0.000	0.00	1.00	0.000	0.00	0.00	5.660	1.87	1.70	3.11	5.24	3.68	5.095	73.6	250 PVC	0.254	0.013	0.25	31.0	0.61	16.9%
Street 3	17	133	135		1			0.003	0.303	3.46	0.98	3.40	2.85	2.00	0.120	5.780		0.000	0.00	1.00	0.000	0.00	0.00	5.780	1.91	1.73	3.18	5.31	3.73	5.178	11.7	250 PVC	0.254	0.013	0.50	43.9	0.87	12.1%
Street 3	18	139	137		25			0.068	0.068	3.63	0.22	0.79	2.97	0.46	0.760	0.760		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	88.3	200 PVC	0.203	0.013	5.00	76.5	2.36	1.4%
Street 3	19	137	135		10			0.027	0.095	3.60	0.31	1.10	2.95	0.65	0.300	1.060		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	57.3	200 PVC	0.203	0.013	4.00	68.4	2.11	2.1%
Street 3	20	135	141		6			0.016	0.414	3.41	1.34	4.58	2.81	2.69	0.280	7.120		0.000	0.00	1.00	0.000	0.00	0.00	7.120	2.35	2.14	3.92	6.93	4.83	6.609	51.2	250 PVC	0.254	0.013	0.25	31.0	0.61	22.3%
Street 3	21	141	143					0.000	0.414	3.41	1.34	4.58	2.81	2.69	0.010	7.130		0.000	0.00	1.00	0.000	0.00	0.00	7.130	2.35	2.14	3.92	6.93	4.83	6.614	4.8	250 PVC	0.254	0.013	3.00	107.5	2.12	6.5%
Street 3	20	147	145		12			0.032	0.032	3.68	0.11	0.39	3.01	0.23	0.660	0.660		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	73.8	200 PVC	0.203	0.013	5.50	80.2	2.47	0.8%
Street 3	19	145	143					0.000	0.032	3.68	0.11	0.39	3.01	0.23	0.090	0.750		0.000	0.00	1.00	0.000	0.00	0.00	0.750	0.25	0.23	0.41	0.63	0.45	0.638	63.5	200 PVC	0.203	0.013	4.00	68.4	2.11	0.9%
Offsite		143	149					0.000	0.447	3.40	1.45	4.92	2.80	2.89	0.000	7.880		0.000	0.00	1.00	0.000	0.00	0.00	7.880	2.60	2.36	4.33	7.52	5.26	7.228	25.2	250 PVC	0.254	0.013	5.50	145.5	2.87	5.2%
TOTALS				62	87	0	0.250	0.447	0.447	3.40	1.45	4.92	2.80	2.89	7.880	7.880		0.000	0.00	1.00	0.000	0.00	0.00	7.880	2.60	2.36	4.33	7.52	5.26	7.228								

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

SANITARY SEWER DESIGN SHEET

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands
Date: 9/4/2024
Input By: Brendan Rundle
Reviewed By: Sam Bahia
Drawing Reference: 2024828 Stinson Phasing

Location				Demand																			
Street	Area ID	From MH	To MH	Residential Flow											Industrial / Commercial / Institutional (ICI) Flow						Extraneous Flow Area Method		Total Design Flow
				Singles	Semis / Towns	Apts	Park Area	Population (in 1000's)	Cumulative Population (in 1000's)	Average Pop. Flow Q(q) (L/s)	Design Peaking Factor M	Peak Design Pop. Flow Q(p) (L/s)	Res. Drainage Area (ha.)	Cumulative Res. Drainage Area (ha.)	Commercial / Institutional Area (ha.)	Cumulative Commercial / Institutional Area (ha.)	Average Design Commercial / Institutional Flow (L/s)	Commercial / Institutional Peaking Factor	Cumulative ICI Area (ha.)	Peak Design ICI Flow Q (ici) (L/s)	Cumulative Extraneous Drainage Area (ha.)	Design Extraneous Flow Q(e) (L/s)	Total Peak Design Flow Q(D) (L/s)
Phase 1	PH1			41	14		0.247	0.178	0.178	0.58	3.53	2.04	3.583	3.583	0.000	0.000	0.00	1.00	0.000	0.00	3.583	1.18	3.22
Phase 1 + Phase 2	PH1&2			21	73			0.269	0.447	1.45	3.40	4.92	4.134	7.717	0.000	0.000	0.00	1.00	0.000	0.00	7.717	2.55	7.47
Totals				**62**	**87**	**0**	**0.247**	**0.447**	**0.447**	**1.45**	**3.40**	**4.92**	**7.717**	**7.717**	**0.000**	**0.000**	**0.00**	**1.00**	**0.000**	**0.00**	**7.717**	**2.55**	**7.47**

Demand Equation / 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 = 280 L/per person/day (design)
200 L/per person/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 = 4 single unit equivalent / park ha (~ 3,600 L/ha/day)
7. Q(fd) = 0.45 L/s/unit
8. Q(ici) = ICI Area x ICI Flow x ICI Peak
9. Q(e) = 0.33 L/s/ha (design)
0.30 L/s/ha (annual)
0.55 L/s/ha (rare)

Definitions

Q(D) = Peak Design Flow (L/s)
Q(A) = Peak Annual Flow (L/s)
Q(R) = Peak Rare Flow (L/s)
Q(p) = Peak Design Population Flow (L/s)
Q(q) = Average Population Flow (L/s)

	Singles	Semis / Towns	Apts
P = Residential Population =	3.4	2.7	2.1
q = Average Capita Flow			
M = Harmon Formula			
K = Harmon Correction Factor			
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/s)
Q(ici) = Industrial / Commercial / Institutional Flow (L/s)
Q(e) = Extraneous Flow (L/s)

Institutional / Commercial / Industrial	Industrial	Commercial / Institutional	
Design =	35000	28000	L/gross ha/day
Annual / Rare =	10000	17000	L/gross ha/day
ICI Peak *			
Design =	1.0	1.5	* ICI Peak = 1.0 Default, 1.5 if ICI in contributing area is >20% (design only)
Annual / Rare =		1.0	

DESIGN BRIEF
VILLAGE OF MANOTICK
MUNICIPAL SERVICING
MAIN SANITARY SEWAGE PUMP STATION
CITY OF OTTAWA

11931

SEPTEMBER 2008

IBI
GROUP

J:\11931_Manotick\5.9 Drawings\59civil\current\Design Brief-Sept. 2008\Fig 7 - 8 Control Wet Well.dwg Sheet Set: #### Last Saved By: ---- Last Saved At: Sep. 29, 08
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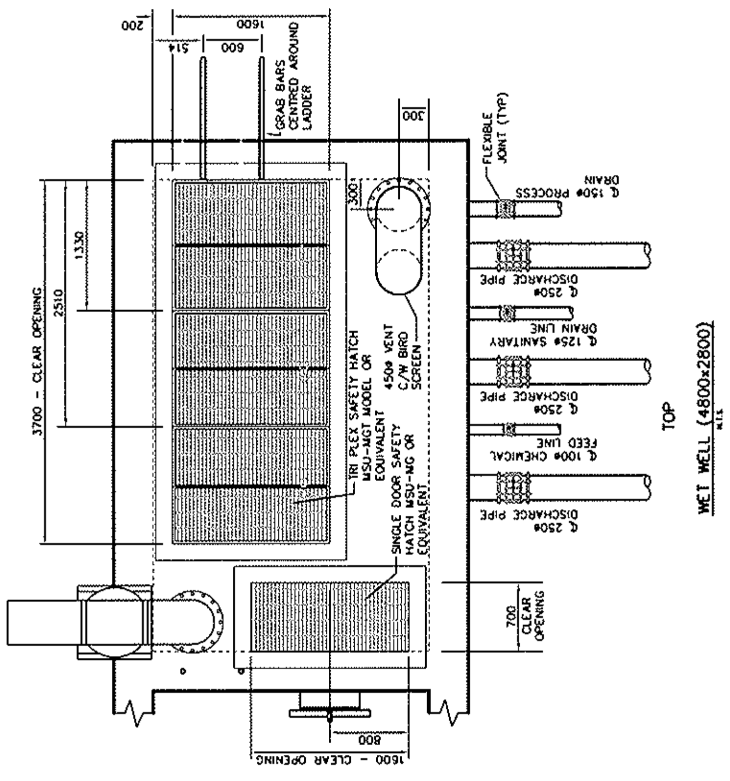

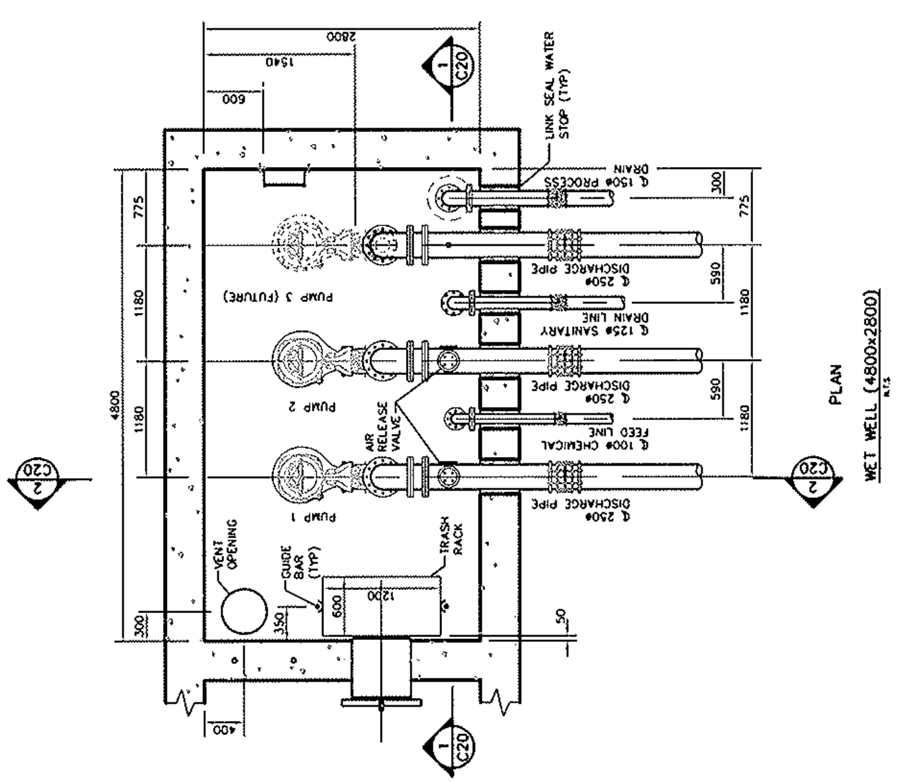

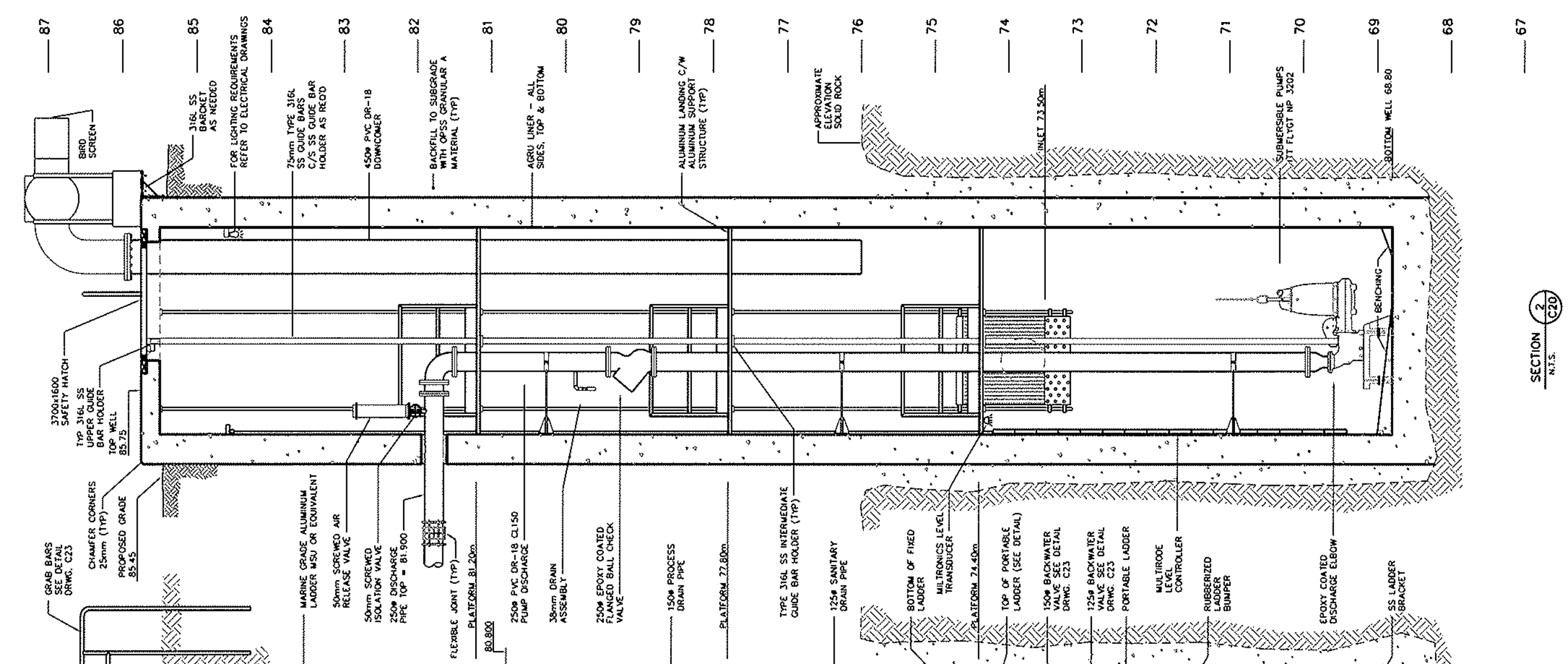

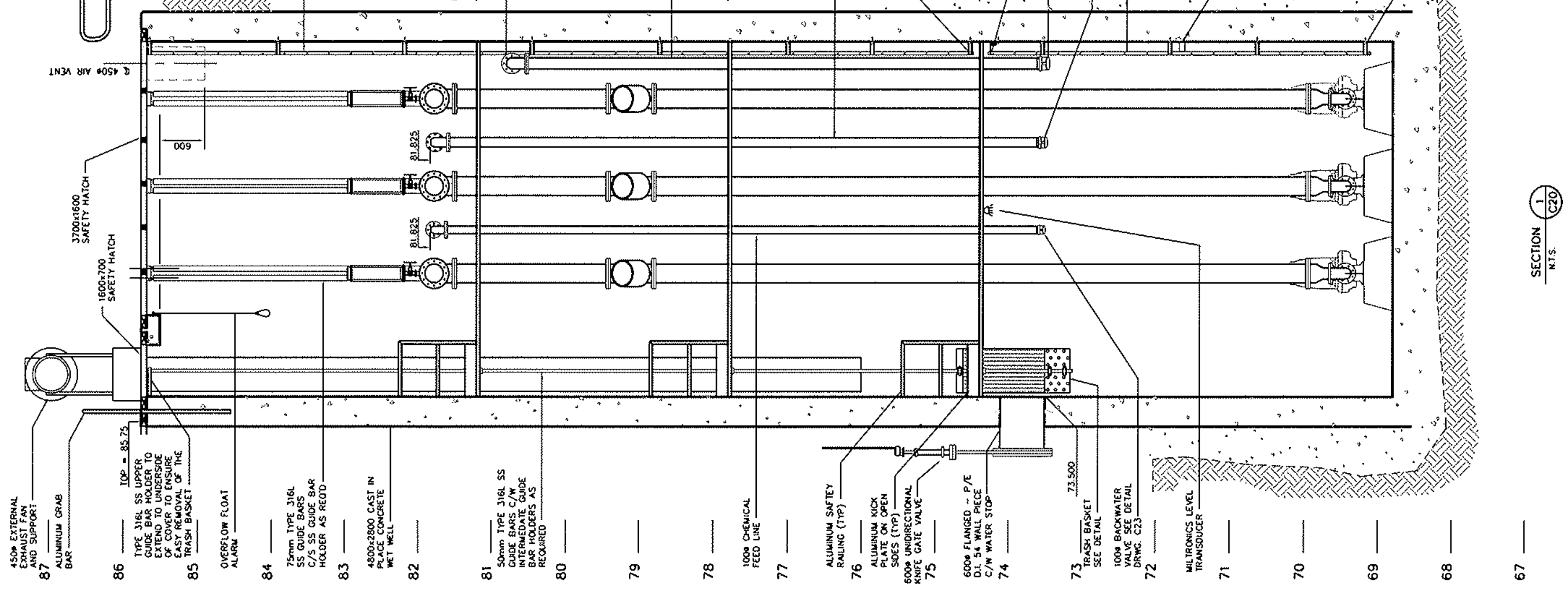

IBI GROUP

Project Title

DESIGN BRIEF
MANOTICK MUNICIPAL SERVICING
MAIN SANITARY SEWAGE PUMP STATION
CITY OF OTTAWA

Drawing Title

WET WELL DETAILS

Project No.
11931

Date
SEPT. 2008

FIGURE 7

J:\11931_Manotick\5.9 Drawings\59civil\current\Design Brief-Sept. 2008\Fig 7 - 8 Control Wet Well.dwg Sheet Set: ####
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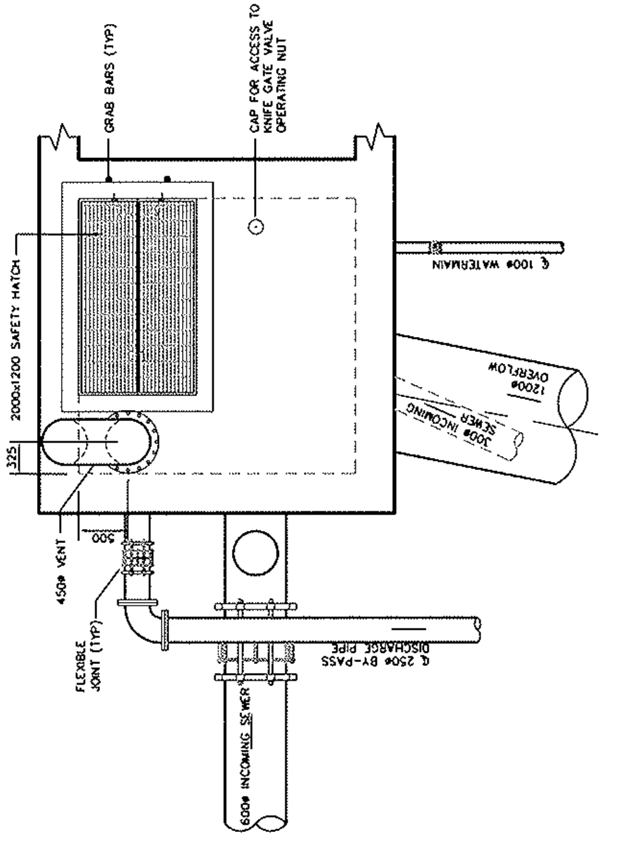
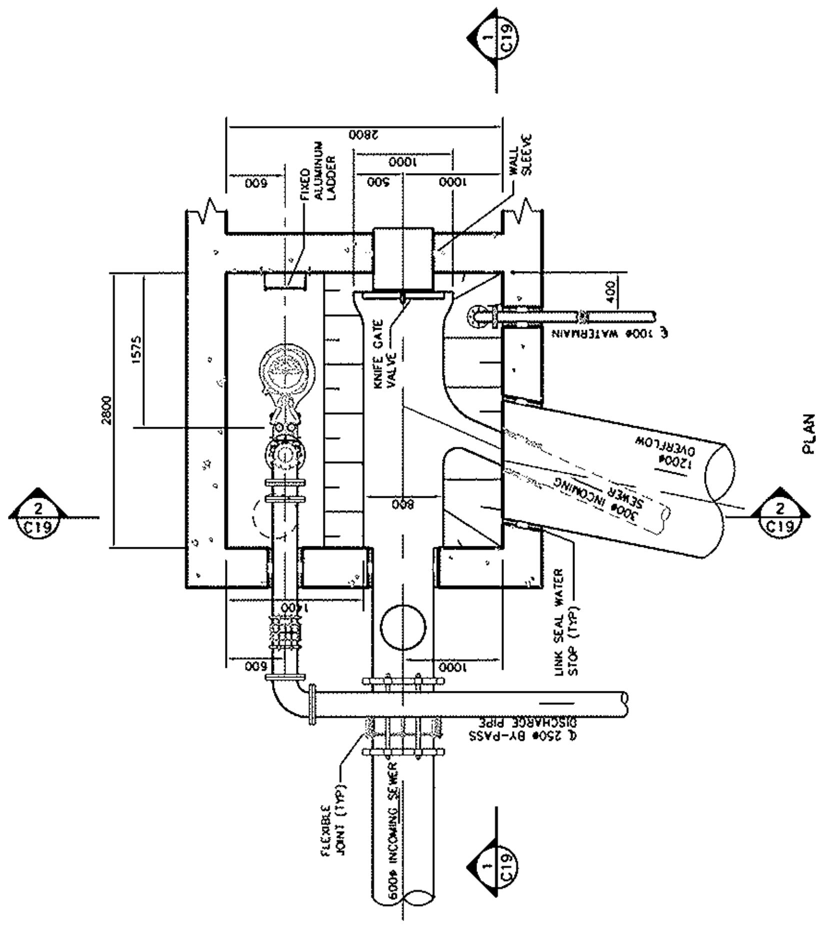
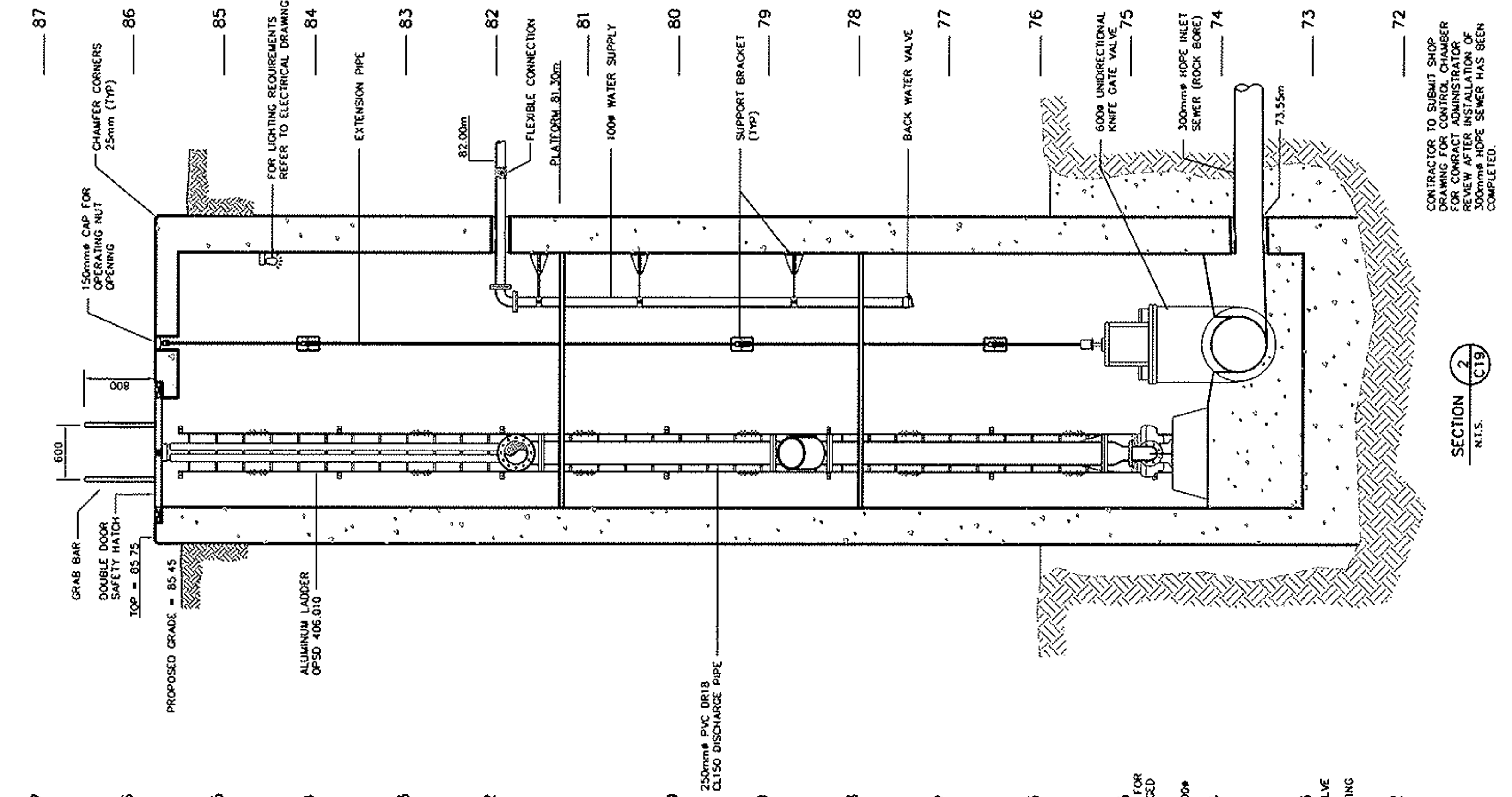


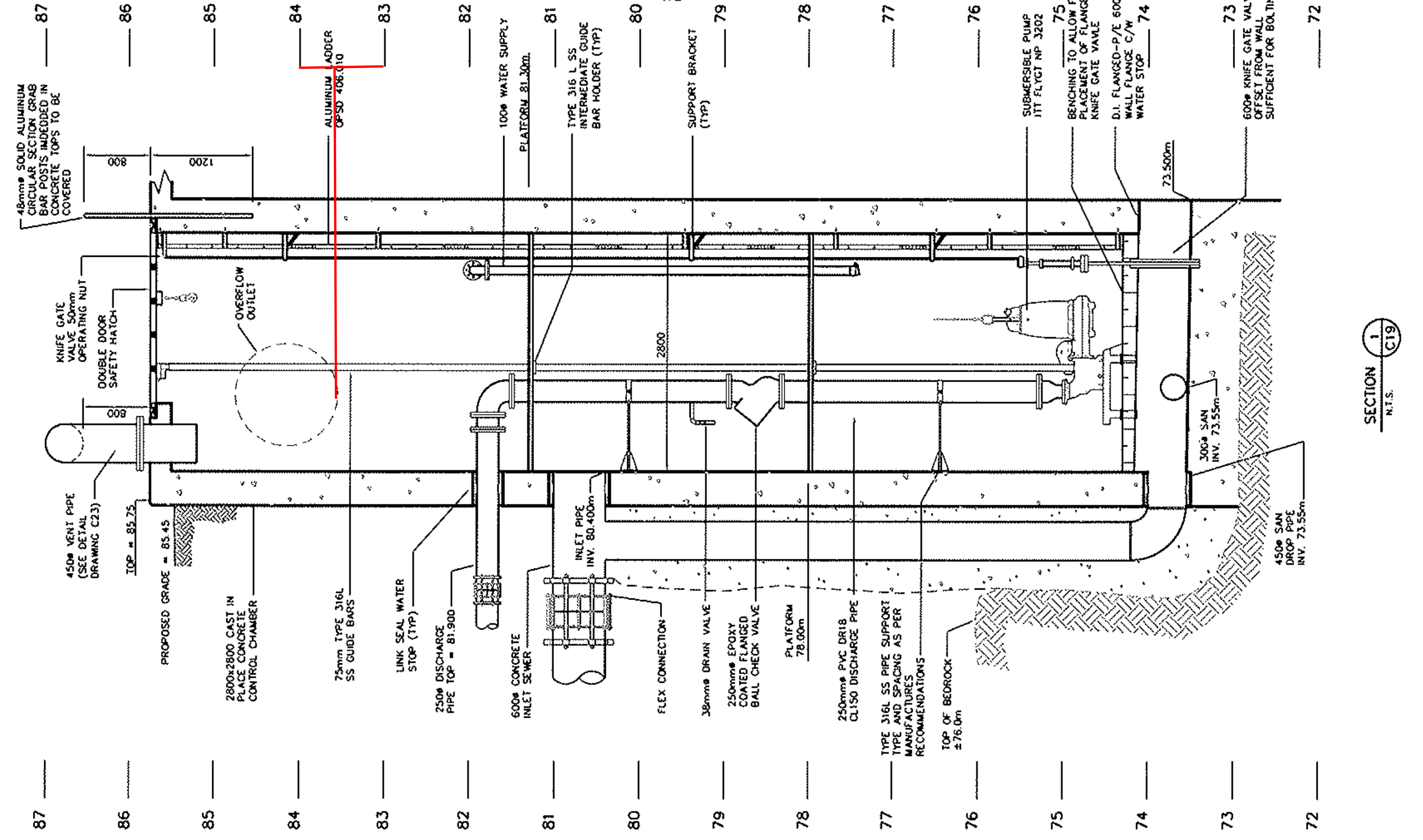
TOP
CONTROL CHAMBER (2800x2800)
N.T.S.



PLAN
CONTROL CHAMBER (2800x2800)
N.T.S.



SECTION 2 C19
N.T.S.



SECTION 1 C19
N.T.S.

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

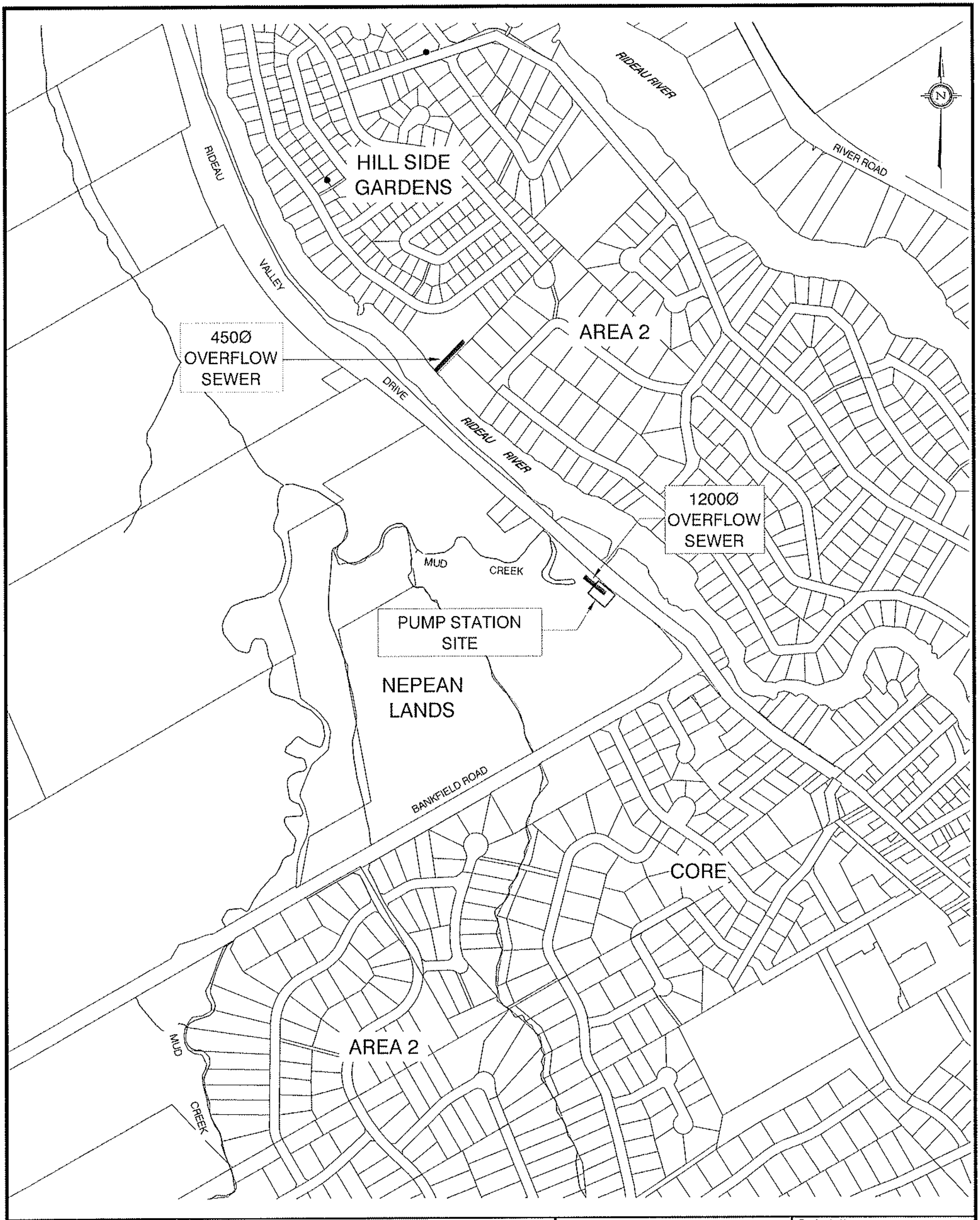
The proposed Main Sanitary Sewage Pump Station in Manotick will receive its power from the Hydro Ottawa power grid. In the event of interruption to that power source, the station will be equipped with a back-up diesel generator which automatically is put into service in the event of a grid power failure. 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.



J:\11931_Manotick\5.9 Drawings\58civil\current\Design Brief-Sept. 2008\FIG 2-5.dwg Sheet Set: ####
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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.

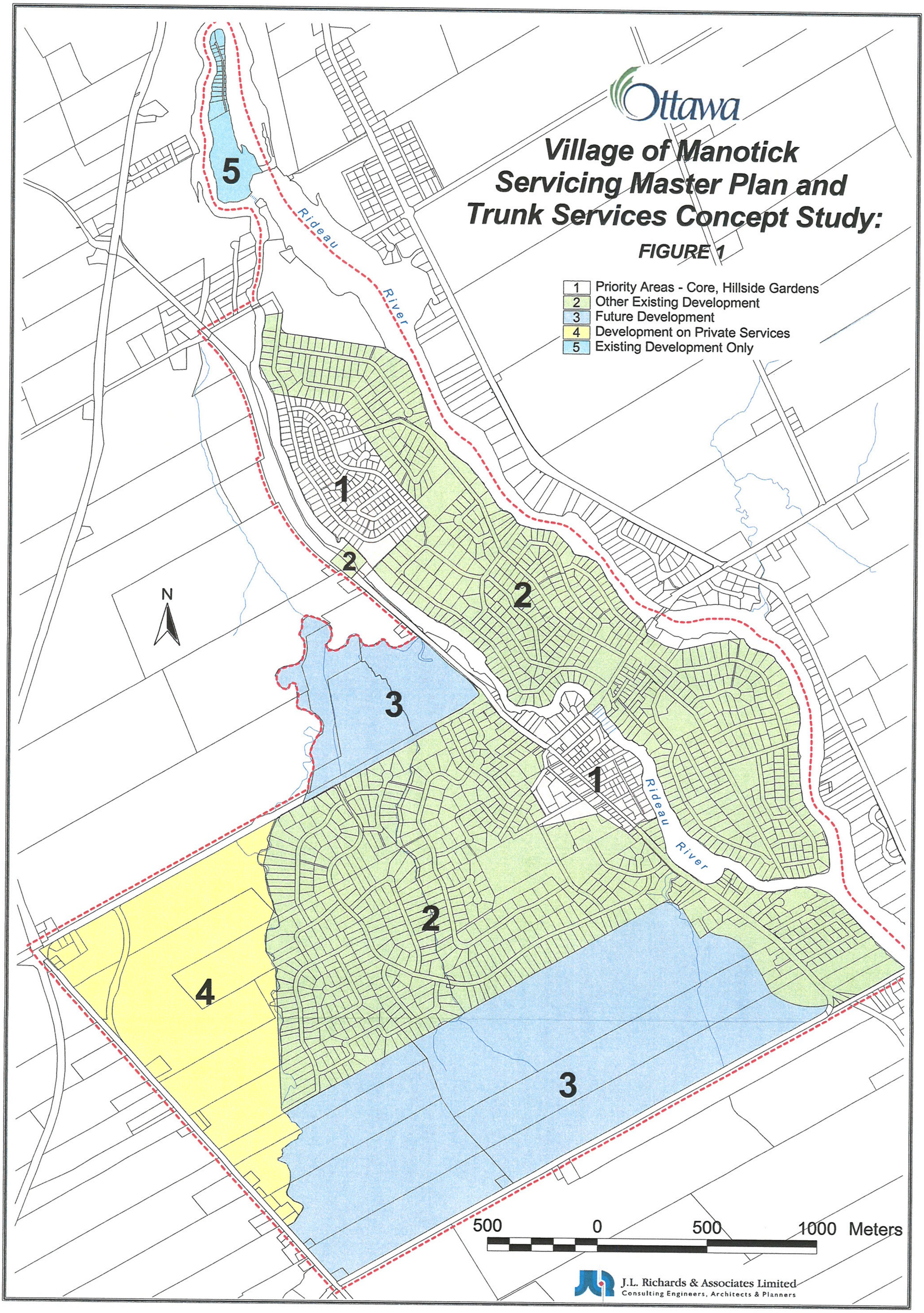
Preliminary discussions with the Hydro Ottawa, the power supply authority, indicate that a 750 KVa supply can be provided to the station. Supply to the station site will be through a pad mount transformer on site.

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



Village of Manotick Servicing Master Plan and Trunk Services Concept Study:

FIGURE 1

APPENDIX B

Sanitary Sewer Design Sheets and Village of Manotick Sanitary Drainage Areas

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% buitl 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	RESID. UNITS Towns Semis	RESID. UNITS 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: ####
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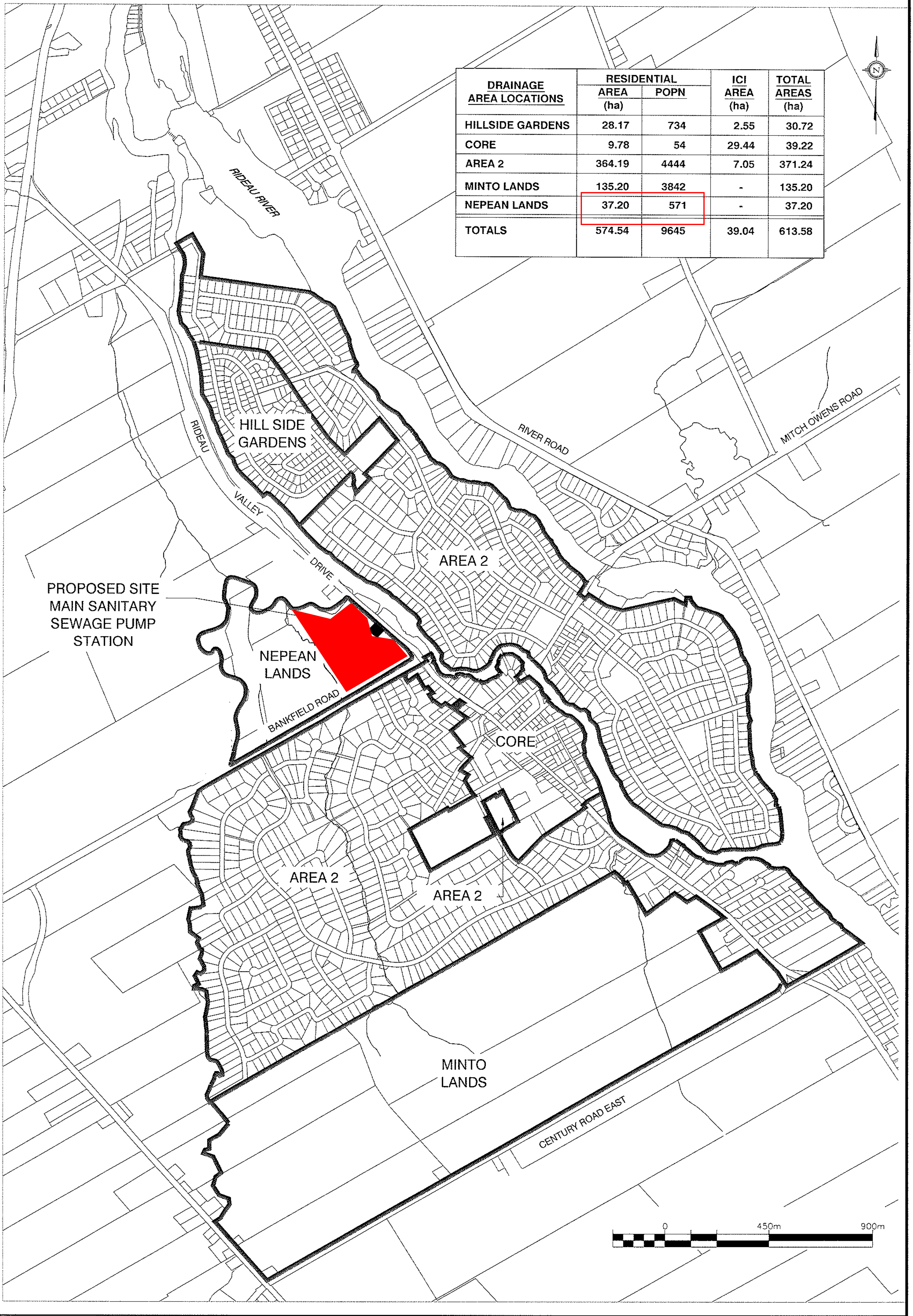

DRAINAGE AREA LOCATIONS	RESIDENTIAL AREA (ha)	RESIDENTIAL POPN	ICI AREA (ha)	TOTAL AREAS (ha)
HILLSIDE GARDENS	28.17	734	2.55	30.72
CORE	9.78	54	29.44	39.22
AREA 2	364.19	4444	7.05	371.24
MINTO LANDS	135.20	3842	-	135.20
NEPEAN LANDS	37.20	571	-	37.20
TOTALS	574.54	9645	39.04	613.58

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: ####
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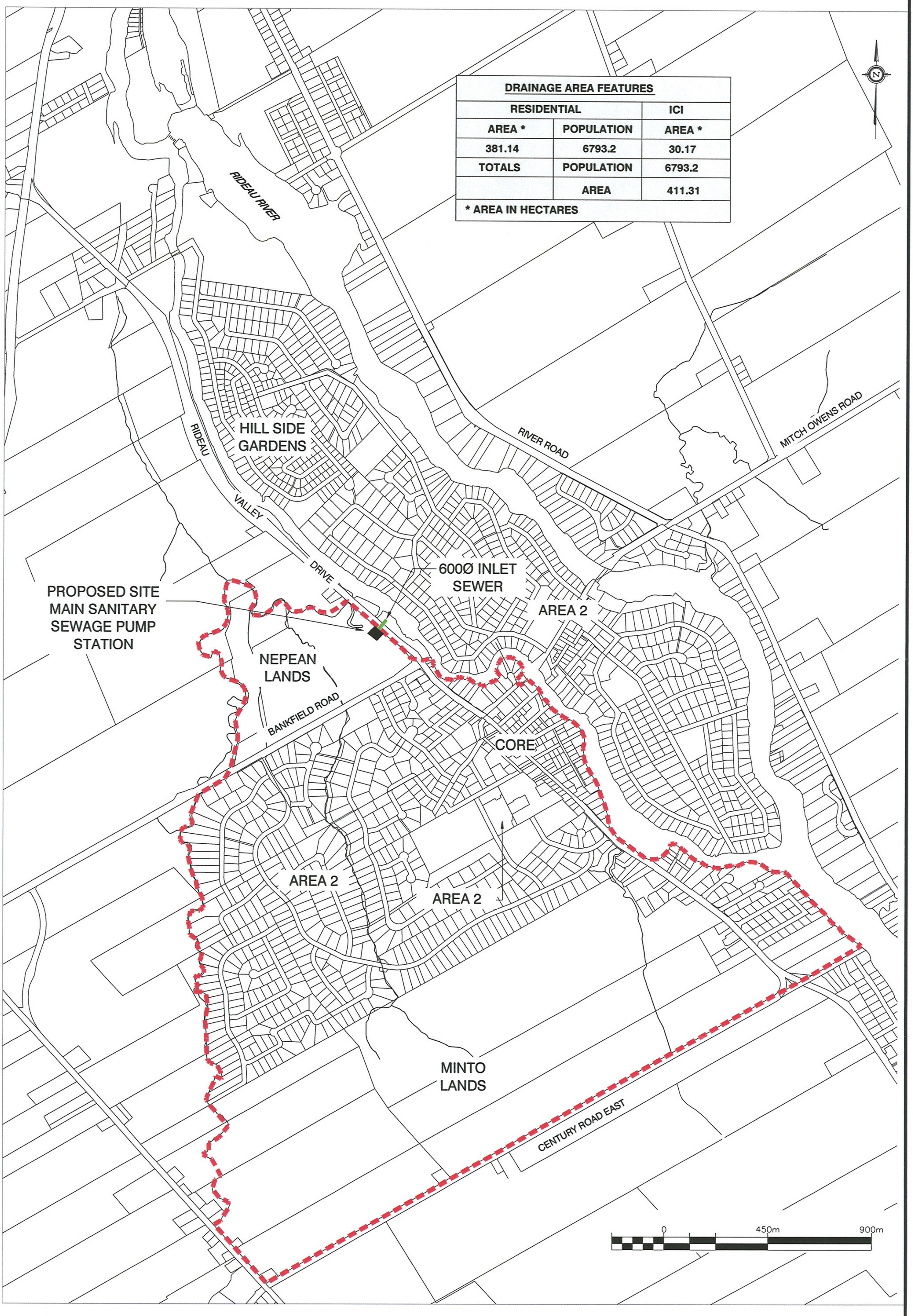


DRAINAGE AREA FEATURES

RESIDENTIAL		ICI
AREA *	POPULATION	AREA *
381.14	6793.2	30.17
TOTALS	POPULATION	6793.2
	AREA	411.31

* AREA IN HECTARES

IBI GROUP

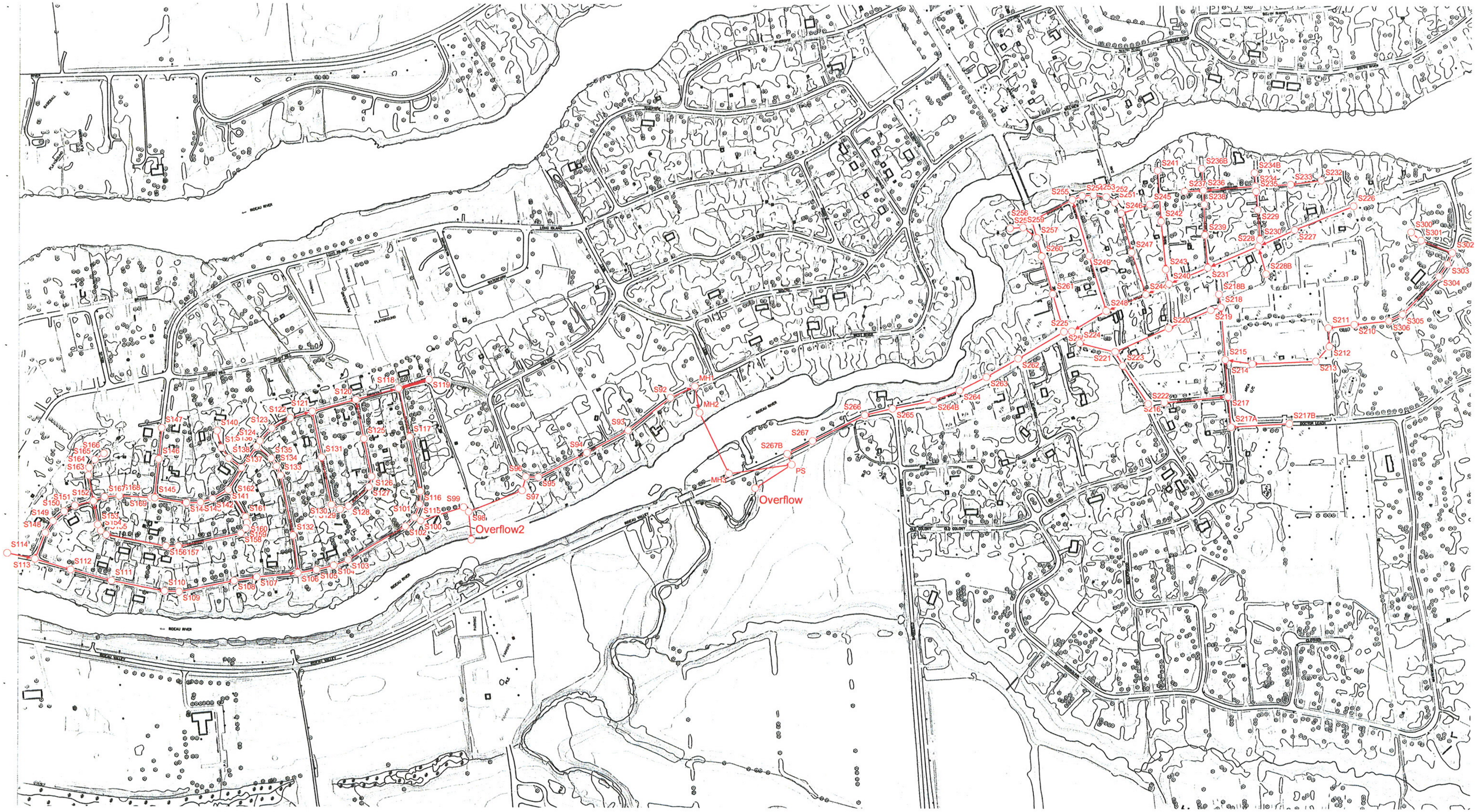
Project Title
DESIGN BRIEF
MANOTICK MUNICIPAL SERVICING
MAIN SANITARY SEWAGE PUMP STATION
CITY OF OTTAWA

Drawing Title
DRAINAGE AREA PLAN
INLET SEWER

Project No. 11931
Date SEPT. 2008
FIGURE 6

APPENDIX D

Emergency Overflow Plan and Profiles



SWMM

Version 9.12

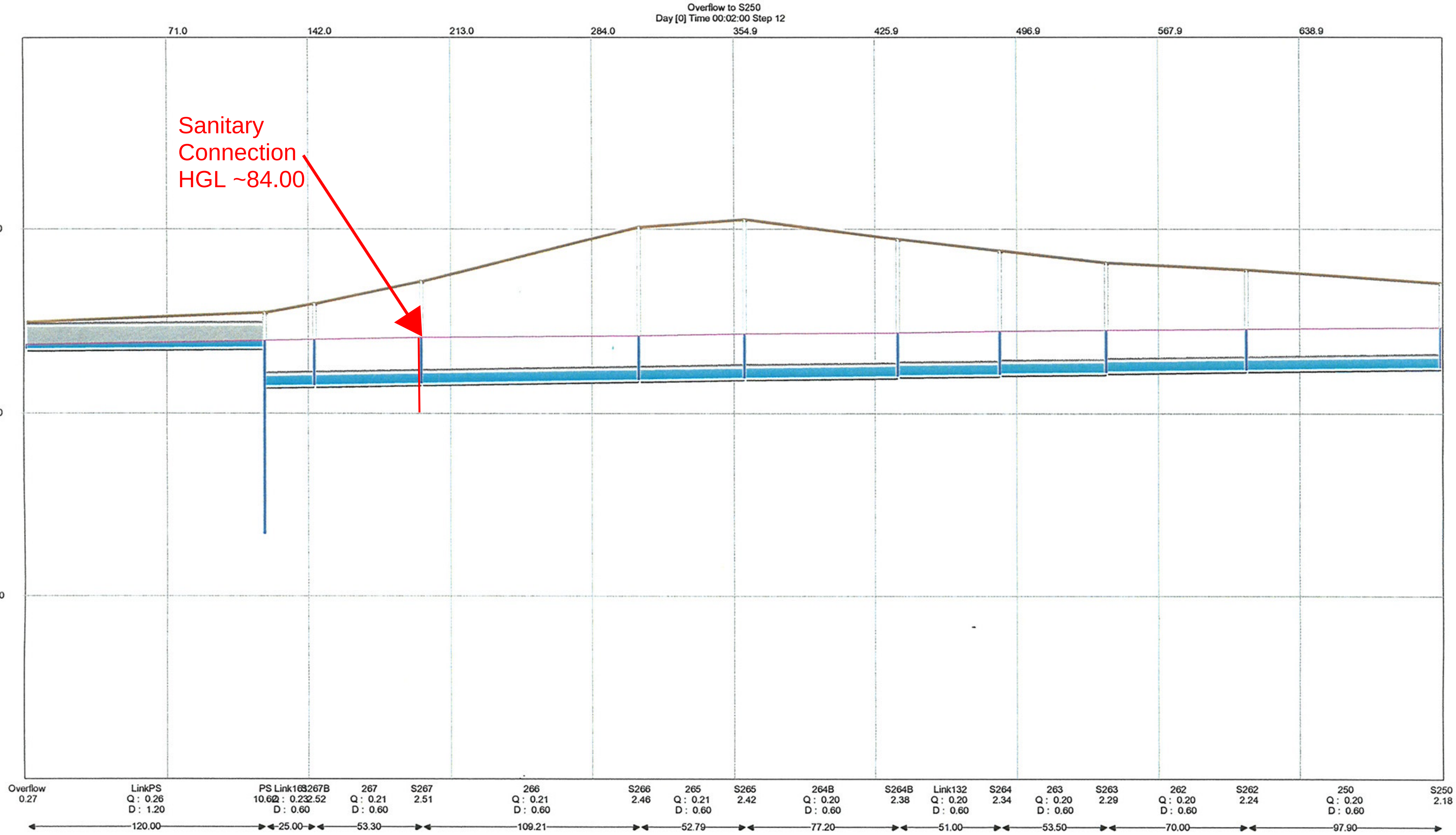
11931 - Manotick Pump Station

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

Sanitary Connection HGL ~84.00

SANITARY SEWER DESIGN SHEET (FUTURE GROWTH)

Novatech Project #: 121153
Project Name: Stinson Lands Subdivision
Date Prepared: 1/11/2023
Date Revised: 1/19/2023
Input By: Brendan Rundle
Reviewed By: Sam Bahia
Drawing Reference: Village of Manotick Servicing Master Plan and Trunk Services Concept Study

NOVATECH
Engineers, Planners & Landscape Architects

LOCATION			DEMAND										
AREA	FROM MH	TO MH	RESIDENTIAL FLOW					INDUSTRIAL / COMMERICAL / INSTITUTIONAL FLOW				EXTRANOUS FLOW AREA METHOD	TOTAL DESIGN FLOW
			POPULATION (in 1000's)	PEAK FACTOR M	AVG POPULATION FLOW Q(q) (L/s)	PEAKED DESIGN POP FLOW Q(p) (L/s)	RESIDENTIAL DRAINAGE AREA (ha.)	COMMERICAL / INSTITUTIONAL AREA (ha.)	AVG DESIGN COMMERICAL / INSTITUTIONAL FLOW Q (ci) (L/s)	COMMERICAL / INSTITUTIONAL PEAK FACTOR	PEAKED DESIGN ICI FLOW Q (CI) (L/s)	DESIGN EXTRAN. FLOW Q(e) (L/s)	TOTAL DESIGN FLOW Q(D) (L/s)
Riverwalk		1670	0.377	3.43	1.22	4.19	15.470	0.000	0.00	1.00	0.00	5.11	9.29
Flows to Mahogany Pumping Station (Mahogany Ph 1-5, Future Minto Lands, Ex Mahogany Estates, Lands E & W of Main St)		1670	6.214	2.72	20.14	54.88	135.200	0.000	0.00	1.00	0.00	44.62	99.49
Servicing Connection (Eastman Ave)	64236	59270	0.034	3.68	0.11	0.41	2.300	0.000	0.00	1.00	0.00	0.76	1.16
Core		1670	0.253	3.49	0.82	2.86	12.530	26.690	8.65	1.50	12.97	12.94	28.78
Servicing Connection (Rideau Valley Dr)	58922	69314	0.003	3.76	0.01	0.04	0.900	0.000	0.00	1.00	0.00	0.30	0.33
Stinson Lands - SUBJECT SITE (Portion of formerly Nepean Lands)		1670	0.447	3.40	1.45	4.92	7.880	0.000	0.00	1.00	0.00	2.60	7.52
Hillside Gardens		1670	0.734	3.31	2.38	7.86	28.170	2.550	0.83	1.00	0.83	10.14	18.83
Servicing Connection (West River Dr)	56426	58900	0.068	3.63	0.22	0.80	4.100	0.000	0.00	1.00	0.00	1.35	2.15
TOTAL FLOW CONTRIBUTION TO MANOTICK PUMPING STATION			8.130	2.63	26.35	69.42	206.550	29.240	9.48	1.00	9.48	77.81	156.70

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(e) = Extraneous Flow (L/sec)
Q(p) = Population Flow (L/sec)
K = Harmon Correction Factor

	Singles
P = Residential Population	3.4
Typ Service Diameter (mm)	135
Typ Service Length (m)	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

Design =
Annual / Rare =

ICI Peak *
Design =
Annual / Rare =

Q(A) = Peak Annual Flow (L/sec)
Q(R) = Peak Rare Flow (L/sec)

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

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands
Date: 4/8/2024
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User
No Input Required
Reference: Fire Underwriter's Survey Guideline (2020)
Formula Method

Building Description: Lots 1-29, 2 Storey Singles
Type V - Wood frame

Step			Choose		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Type V - Wood frame	Yes	1.5	1.5	
		Type IV - Mass Timber		Varies		
		Type III - Ordinary construction		1		
		Type II - Non-combustible construction		0.8		
		Type I - Fire resistive construction (2 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	5655			
		Number of Floors/Storeys	2			
		Protected Openings (1 hr) if C<1.0	No			
		Area of structure considered (m^2)			**11,310**	
	F	**Base fire flow without reductions** $F = 220\ C\ (A)^{0.5}$				**35,000**
Reductions or Surcharges						
3	**Occupancy hazard reduction or surcharge**		**FUS Table 3**	**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**29,750**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**		**FUS Table 4**	**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	No	-30%		**0**
		Standard Water Supply	No	-10%		
		Fully Supervised System	No	-10%		
		Cumulative Sub-Total			**0%**	
		Area of Sprinklered Coverage (m²)	0	0%		
		Cumulative Total			**0%**	
5	**Exposure Surcharge**		**FUS Table 5**		**Surcharge**	
	(3)	North Side	>30m		0%	**2,975**
		East Side	20.1 - 30 m		10%	
		South Side	>30m		0%	
		West Side	>30m		0%	
		Cumulative Total			**10%**	
Results						
6	(1) + (2) + (3)	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**33,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**550**
				or	USGPM	8,719

FUS - Fire Flow Calculations

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands
Date: 4/8/2024
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User
No Input Required
Reference: Fire Underwriter's Survey Guideline (2020)
Formula Method

Building Description: Block 75, 2 Storey Townhomes
Type V - Wood frame

Step			Choose		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
	Construction Material			**Multiplier**		
1	**Coefficient related to type of construction C**	Type V - Wood frame	Yes	1.5	1.5	
		Type IV - Mass Timber		Varies		
		Type III - Ordinary construction		1		
		Type II - Non-combustible construction		0.8		
		Type I - Fire resistive construction (2 hrs)		0.6		
	Floor Area					
2	A	Building Footprint (m²)	2200			
		Number of Floors/Storeys	2			
		Protected Openings (1 hr) if C<1.0	No			
		Area of structure considered (m²)			**4,400**	
	F	**Base fire flow without reductions** $F = 220\ C\ (A)^{0.5}$				**22,000**
Reductions or Surcharges						
	Occupancy hazard reduction or surcharge		**FUS Table 3**	**Reduction/Surcharge**		
3	(1)	Non-combustible		-25%	-15%	**18,700**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
	Sprinkler Reduction		**FUS Table 4**	**Reduction**		
4	(2)	Adequately Designed System (NFPA 13)	No	-30%		**0**
		Standard Water Supply	No	-10%		
		Fully Supervised System	No	-10%		
				Cumulative Sub-Total	**0%**	
		Area of Sprinklered Coverage (m²)	0	0%		
				Cumulative Total	**0%**	
	Exposure Surcharge		**FUS Table 5**		**Surcharge**	
5	(3)	North Side	20.1 - 30 m		10%	**4,675**
		East Side	>30m		0%	
		South Side	>30m		0%	
		West Side	10.1 - 20 m		15%	
				Cumulative Total	**25%**	
Results						
6	(1) + (2) + (3)	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**23,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**383**
				or	USGPM	6,077

Water Demand Design Sheet

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Calculated Cells →
Reference: Ottawa Design Guidelines - Water Distribution (2010 and TBs)
MOE Design Guidelines for Drinking-Water Systems (2008)
Fire Underwriter's Survey Guideline (2020)
Ontario Building Code, Part 3 (2012)

Small System = NO

Location	**Total Water Demand**																				
Node	**Residential Input & Average Demand**							**Industrial / Commercial / Institutional (ICI) Input & Average Demand**						**Maximum Day & Peak Hour Demand**						**Design Fire Demand**	
								Indust. Area						**Maximum Day Demand**			**Peak Hour Demand**			**Required Fire Flow (RFF)**	
	Singles	**Semis / Towns**	**Apts (2-BR)**	**Apts (1-BR)**	**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)**	**FUS (L/min)**	**Max Day + RFF (L/s)**
J01						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J03						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00	10,000	166.67
J04		10				27.00	0.09						0.00	2.50	1.50	0.22	5.50	2.70	0.48	10,000	166.89
J07						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J08	2					6.80	0.02						0.00	2.50	1.50	0.06	5.50	2.70	0.12	10,000	166.72
J11	4	4				24.40	0.08						0.00	2.50	1.50	0.20	5.50	2.70	0.43	10,000	166.86
J15	4					13.60	0.04						0.00	2.50	1.50	0.11	5.50	2.70	0.24	10,000	166.78
J16	12					40.80	0.13						0.00	2.50	1.50	0.33	5.50	2.70	0.73	10,000	167.00
J17	20					68.00	0.22						0.00	2.50	1.50	0.55	5.50	2.70	1.21	10,000	167.22
Totals	**42**	**14**	**0**	**0**	**0**	**180.60**	**0.59**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**2.50**	**1.50**	**1.46**	**5.50**	**2.70**	**3.22**		

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
Dailly Demand	L/per person/day				
Average Demand	280				
Basic Demand	200				

Institutional / Commercial / Industrial				
Indust.		**Comm.**	**Inst.**	**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

Residential Peaking Factors		**Max Day (x Avg Day)**	**Peak Hour (x Avg Day)**
	Pop.		
Small System (If Applicable) *Modified*	0	9.50	14.30
	30	9.50	14.30
	150	4.90	7.40
	300	3.60	*5.50*
	450	3.00	*5.50*
	500	2.90	*5.50*
Large System (Default)	> 500	2.50	5.50

ICI Peaking Factors	**Max Day (x Avg Day)**	**Peak Hour (x Avg Day)**
	1.50	2.70

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**		

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Water Demand Design Sheet

Maximum Pressure During Average Day (AVDY) Conditions

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Acceptable (40psi - 80psi)
Acceptable w/ PRV (81psi - 100psi)
Unacceptable (< 40psi or > 100psi)

Note: Hydraulic modelling completed using EPANET 2.0.

Future Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	148.20	54.70	78
J03	87.70	0.00	148.20	60.50	86
J04	87.60	0.09	148.20	60.60	86
J07	93.60	0.00	148.20	54.60	78
J08	90.00	0.02	148.20	58.20	83
J11	87.80	0.08	148.20	60.40	86
J15	88.20	0.04	148.20	60.00	85
J16	88.40	0.13	148.20	59.80	85
J17	89.10	0.22	148.20	59.10	84

Existing Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	156.60	63.10	90
J03	87.70	0.00	156.60	68.90	98
J04	87.60	0.09	156.60	69.00	98
J07	93.60	0.00	156.60	63.00	90
J08	90.00	0.02	156.60	66.60	95
J11	87.80	0.08	156.60	68.80	98
J15	88.20	0.04	156.60	68.40	97
J16	88.40	0.13	156.60	68.20	97
J17	89.10	0.22	156.60	67.50	96

Water Demand Design Sheet

Minimum Pressure During Peak Hour (PKHR) Conditions

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Acceptable (=> 40psi)
Unacceptable (< 40psi)

Note: Hydraulic modelling completed using EPANET 2.0.

Future Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	141.51	48.01	68
J03	87.70	0.00	141.59	53.89	77
J04	87.60	0.48	141.57	53.97	77
J07	93.60	0.00	141.51	47.91	68
J08	90.00	0.12	141.52	51.52	73
J11	87.80	0.43	141.56	53.76	76
J15	88.20	0.24	141.56	53.36	76
J16	88.40	0.73	141.55	53.15	76
J17	89.10	1.21	141.54	52.44	75

Existing Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	139.60	46.10	66
J03	87.70	0.00	139.60	51.90	74
J04	87.60	0.48	139.60	52.00	74
J07	93.60	0.00	139.60	46.00	65
J08	90.00	0.12	139.60	49.60	71
J11	87.80	0.43	139.60	51.80	74
J15	88.20	0.24	139.60	51.40	73
J16	88.40	0.73	139.60	51.20	73
J17	89.10	1.21	139.60	50.50	72

Water Demand Design Sheet

NOVATECH
Engineers, Planners & Landscape Architects

Minimum Pressure During Max Day Plus Fire Flow (MXDY+FF) Condition

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Acceptable (=> 20psi)
Unacceptable (< 20psi)
Note: Hydraulic modelling completed using EPANET 2.0.

Future Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)	FF Demand (L/min)	FF Available (L/min)
J01	93.50	0.00	118.69	25.19	36	-	-
J03	87.70	0.00	119.66	31.96	45	10000	29100
J04	87.60	0.22	119.55	31.95	45	10000	16800
J07	93.60	0.00	118.82	25.22	36	-	-
J08	90.00	0.06	119.02	29.02	41	10000	10860
J11	87.80	0.20	119.49	31.69	45	10000	14760
J15	88.20	0.11	119.42	31.22	44	10000	13260
J16	88.40	0.33	119.40	31.00	44	10000	12900
J17	89.10	0.55	119.25	30.15	43	10000	11340

Existing Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)	FF Demand (L/min)	FF Available (L/min)
J01	93.50	0.00	123.10	29.60	42	-	-
J03	87.70	0.00	124.15	36.45	52	10000	32880
J04	87.60	0.22	124.03	36.43	52	10000	18960
J07	93.60	0.00	123.24	29.64	42	-	-
J08	90.00	0.06	123.45	33.45	48	10000	12480
J11	87.80	0.20	123.97	36.17	51	10000	16740
J15	88.20	0.11	123.90	35.70	51	10000	15060
J16	88.40	0.33	123.88	35.48	50	10000	14640
J17	89.10	0.55	123.71	34.61	49	10000	12900

Note: FF Available results based on a residual system pressure of 20 psi.

Water Demand Design Sheet

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1 + Ph2
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Calculated Cells →

Reference: Ottawa Design Guidelines - Water Distribution (2010 and TBs)
MOE Design Guidelines for Drinking-Water Systems (2008)
Fire Underwriter's Survey Guideline (2020)
Ontario Building Code, Part 3 (2012)

Small System = NO

Location	**Total Water Demand**																				
Node	**Residential Input & Average Demand**							**Industrial / Commercial / Institutional (ICI) Input & Average Demand**						**Maximum Day & Peak Hour Demand**						**Design Fire Demand**	
	Singles	**Semis / Towns**	**Apts (2-BR)**	**Apts (1-BR)**	**Apts (Avg)**	**Pop. Equiv.**	**Res. Average Day Flow Demand (L/s)**	**Indust. Area**		**Comm. Area (ha.)**	**Inst. Area (ha.)**	**Other Area (m²)**	**ICI Average Day Flow Demand (L/s)**	**Maximum Day Demand**			**Peak Hour Demand**			**Required Fire Flow (RFF)**	**Max Day + RFF (L/s)**
								Light (ha.)	**Heavy (ha.)**					**Res. Peaking Factor**	**ICI Peaking Factor**	**Max Day Flow Demand (L/s)**	**Res. Peaking Factor**	**ICI Peaking Factor**	**Peak Hour Flow Demand (L/s)**	**FUS (L/min)**	
J01						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J02		12				32.40	0.11						0.00	2.50	1.50	0.26	5.50	2.70	0.58	10,000	166.93
J03						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J04						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J05		16				43.20	0.14						0.00	2.50	1.50	0.35	5.50	2.70	0.77	10,000	167.02
J06		30				81.00	0.26						0.00	2.50	1.50	0.66	5.50	2.70	1.44	10,000	167.32
J07						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J08						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J09	6	17				66.30	0.21						0.00	2.50	1.50	0.54	5.50	2.70	1.18	10,000	167.20
J10	10	12				66.40	0.22						0.00	2.50	1.50	0.54	5.50	2.70	1.18	10,000	167.20
J11						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J12						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J13	8					27.20	0.09						0.00	2.50	1.50	0.22	5.50	2.70	0.48	10,000	166.89
J14	7					23.80	0.08						0.00	2.50	1.50	0.19	5.50	2.70	0.42	10,000	166.86
J15						0.00	0.00						0.00	2.50	1.50	0.00	5.50	2.70	0.00		0.00
J16	12					40.80	0.13						0.00	2.50	1.50	0.33	5.50	2.70	0.73	10,000	167.00
J17	20					68.00	0.22						0.00	2.50	1.50	0.55	5.50	2.70	1.21	10,000	167.22
Totals	**63**	**87**	**0**	**0**	**0**	**449.10**	**1.46**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**	**2.50**	**1.50**	**3.64**	**5.50**	**2.70**	**8.00**		

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
Dailly Demand	L/per person/day				
Average Demand	280				
Basic Demand	200				

Institutional / Commercial / Industrial				
Indust.		**Comm.**	**Inst.**	**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)**
	1.50	2.70

Residential Peaking Factors		**Max Day (x Avg Day)**	**Peak Hour (x Avg Day)**
	Pop.		
Small System (If Applicable)	0	9.50	14.30
	30	9.50	14.30
	150	4.90	7.40
Modified	300	3.60	*5.50*
	450	3.00	*5.50*
	500	2.90	*5.50*
Large System (Default)	> 500	2.50	5.50

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.		
	Medium Density		
15,000	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**		

NOVATECH
M:\2021\121153\DATA\Calculations\Sewer Calcs\Water\Second Submission\20240408-Water Demand Sheet.xlsx

Water Demand Design Sheet

NOVATECH
Engineers, Planners & Landscape Architects

Maximum Pressure During Average Day (AVDY) Conditions

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1 + Ph2
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Acceptable (40psi - 80psi)
Acceptable w/ PRV (81psi - 100psi)
Unacceptable (< 40psi or > 100psi)

Note: Hydraulic modelling completed using EPANET 2.0.

Future Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	148.20	54.70	78
J02	91.70	0.11	148.20	56.50	80
J03	87.70	0.00	148.20	60.50	86
J04	87.60	0.00	148.20	60.60	86
J05	87.60	0.14	148.20	60.60	86
J06	91.50	0.26	148.20	56.70	81
J07	93.60	0.00	148.20	54.60	78
J08	90.00	0.00	148.20	58.20	83
J09	89.50	0.21	148.20	58.70	83
J10	88.20	0.22	148.20	60.00	85
J11	87.80	0.00	148.20	60.40	86
J12	87.40	0.00	148.20	60.80	86
J13	88.20	0.09	148.20	60.00	85
J14	88.20	0.08	148.20	60.00	85
J15	88.20	0.00	148.20	60.00	85
J16	88.40	0.13	148.20	59.80	85
J17	89.10	0.22	148.20	59.10	84

Existing Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	156.60	63.10	90
J02	91.70	0.11	156.60	64.90	92
J03	87.70	0.00	156.60	68.90	98
J04	87.60	0.00	156.60	69.00	98
J05	87.60	0.14	156.60	69.00	98
J06	91.50	0.26	156.60	65.10	93
J07	93.60	0.00	156.60	63.00	90
J08	90.00	0.00	156.60	66.60	95
J09	89.50	0.21	156.60	67.10	95
J10	88.20	0.22	156.60	68.40	97
J11	87.80	0.00	156.60	68.80	98
J12	87.40	0.00	156.60	69.20	98
J13	88.20	0.09	156.60	68.40	97
J14	88.20	0.08	156.60	68.40	97
J15	88.20	0.00	156.60	68.40	97
J16	88.40	0.13	156.60	68.20	97
J17	89.10	0.22	156.60	67.50	96

Water Demand Design Sheet

NOVATECH
Engineers, Planners & Landscape Architects

Minimum Pressure During Peak Hour (PKHR) Conditions

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1 + Ph2
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Acceptable (=> 40psi)
Unacceptable (< 40psi)
Note: Hydraulic modelling completed using EPANET 2.0.

Future Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	141.52	48.02	68
J02	91.70	0.58	141.54	49.84	71
J03	87.70	0.00	141.57	53.87	77
J04	87.60	0.00	141.53	53.93	77
J05	87.60	0.77	141.53	53.93	77
J06	91.50	1.44	141.52	50.02	71
J07	93.60	0.00	141.52	47.92	68
J08	90.00	0.00	141.52	51.52	73
J09	89.50	1.18	141.52	52.02	74
J10	88.20	1.18	141.52	53.32	76
J11	87.80	0.00	141.53	53.73	76
J12	87.40	0.00	141.52	54.12	77
J13	88.20	0.48	141.52	53.32	76
J14	88.20	0.42	141.52	53.32	76
J15	88.20	0.00	141.52	53.32	76
J16	88.40	0.73	141.52	53.12	76
J17	89.10	1.21	141.52	52.42	75

Existing Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)
J01	93.50	0.00	139.60	46.10	66
J02	91.70	0.58	139.60	47.90	68
J03	87.70	0.00	139.60	51.90	74
J04	87.60	0.00	139.59	51.99	74
J05	87.60	0.77	139.59	51.99	74
J06	91.50	1.44	139.59	48.09	68
J07	93.60	0.00	139.59	45.99	65
J08	90.00	0.00	139.58	49.58	71
J09	89.50	1.18	139.58	50.08	71
J10	88.20	1.18	139.58	51.38	73
J11	87.80	0.00	139.58	51.78	74
J12	87.40	0.00	139.58	52.18	74
J13	88.20	0.48	139.58	51.38	73
J14	88.20	0.42	139.58	51.38	73
J15	88.20	0.00	139.58	51.38	73
J16	88.40	0.73	139.58	51.18	73
J17	89.10	1.21	139.58	50.48	72

Water Demand Design Sheet

NOVATECH
Engineers, Planners & Landscape Architects

Minimum Pressure During Max Day Plus Fire Flow (MXDY+FF) Condition

Novatech Project #: 121153
Project Name: Stinson Lands - Ph1 + Ph2
Date: 4/8/2024 (rev. 12/19/2024)
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User | No Input Required
Acceptable (=> 20psi)
Unacceptable (< 20psi)
Note: Hydraulic modelling completed using EPANET 2.0.

Future Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)	FF Demand (L/min)	FF Available (L/min)
J01	93.50	0.00	119.07	25.57	36	-	-
J02	91.70	0.26	119.22	27.52	39	10000	18417
J03	87.70	0.00	119.48	31.78	45	-	-
J04	87.60	0.00	119.33	31.73	45	-	-
J05	87.60	0.35	119.32	31.72	45	10000	18779
J06	91.50	0.66	119.26	27.76	39	10000	14131
J07	93.60	0.00	119.22	25.62	36	-	-
J08	90.00	0.00	119.27	29.27	42	-	-
J09	89.50	0.54	119.28	29.78	42	10000	13118
J10	88.20	0.54	119.30	31.10	44	10000	14128
J11	87.80	0.00	119.30	31.50	45	-	-
J12	87.40	0.00	119.30	31.90	45	-	-
J13	88.20	0.22	119.30	31.10	44	10000	9746
J14	88.20	0.19	119.30	31.10	44	10000	12637
J15	88.20	0.00	119.30	31.10	44	-	-
J16	88.40	0.33	119.30	30.90	44	10000	13082
J17	89.10	0.55	119.29	30.19	43	10000	11519

Existing Conditions

Node	Elevation (m)	Demand (L/s)	Total Head (m)	Pressure (m)	Pressure (psi)	FF Demand (L/min)	FF Available (L/min)
J01	93.50	0.00	123.52	30.02	43	-	-
J02	91.70	0.26	123.68	31.98	45	10000	21515
J03	87.70	0.00	123.96	36.26	52	-	-
J04	87.60	0.00	123.80	36.20	51	-	-
J05	87.60	0.35	123.79	36.19	51	10000	21856
J06	91.50	0.66	123.72	32.22	46	10000	16484
J07	93.60	0.00	123.68	30.08	43	-	-
J08	90.00	0.00	123.74	33.74	48	-	-
J09	89.50	0.54	123.74	34.24	49	10000	15041
J10	88.20	0.54	123.76	35.56	51	10000	16046
J11	87.80	0.00	123.77	35.97	51	-	-
J12	87.40	0.00	123.77	36.37	52	-	-
J13	88.20	0.22	123.77	35.57	51	10000	11069
J14	88.20	0.19	123.77	35.57	51	10000	14354
J15	88.20	0.00	123.77	35.57	51	-	-
J16	88.40	0.33	123.76	35.36	50	10000	14879
J17	89.10	0.55	123.75	34.65	49	10000	13167

Note: FF Available results based on a residual system pressure of 20 psi.

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

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[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 except 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

PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

4386 Rideau Valley Drive
Ottawa, Ontario

Prepared for Uniform Developments

Report PG5828-1 Revision 5 dated July 19, 2024

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Uniform Developments to conduct a geotechnical investigation for the proposed industrial building, located at 4386 Rideau Valley Drive, Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.

- Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the conceptual site plan, it is understood that the proposed development will consist of townhouses and single-family residential dwellings. Associated driveways, garages, roadways, and landscaping areas are also anticipated throughout the subject site. It is anticipated the proposed dwellings will be provided basement levels. Further, it is anticipated that the proposed development will be municipally serviced.

It is to be noted that as part of the proposed residential subdivision, it is anticipated that a river park will be constructed on 4386 Rideau Valley Drive.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on May 19 and 20, 2021 and consisted of advancing a total of 9 boreholes to a maximum depth of 6.7 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5828-1 - Test Hole Location Plan included in Appendix 2.

Also, a supplemental field investigation was completed for the proposed river park, which is to be located across 4386 Rideau Valley Drive on June 16, 2022, to assess the slope stability of the proposed park and to delineate the limit of hazard lands. At that time, a total of two boreholes were advanced down to a maximum depth of 5.9 m below existing ground surface. The results of this supplemental field investigation are presented in Appendix 3.

The boreholes were completed using a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The testing procedure consisted of augering and excavating to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at boreholes BH 3-21 and BH 5-21. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Boreholes BH 8-21 and BH 9-21 were fitted with 51 mm diameter PVC groundwater monitoring wells. The other boreholes were fitted with flexible piezometers to allow groundwater level monitoring. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

☐ 3.0 m of slotted 51 mm diameter PVC screen at the base of the boreholes.
☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
☐ No. 3 silica sand backfill within annular space around screen.
☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The borehole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5828-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 1 shrinkage test, 4 grain size distribution analyses and 8 Atterberg limit tests were completed on selected soil samples. The results of the testing are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limits Results sheets presented in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was collected from BH 3-21 and submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of agricultural farmland and is currently occupied by a residential dwelling and associated structures at the southeast property boundary. The ground surface across the subject site slopes downward gradually from south to north and east to west.

The site is intersected by Mud Creek along its center and bordered to the west by Wilson Cowan Drain. The area along the creek is bordered by sloped terrain and valley corridors which were reviewed in the field at the time of completing the field investigation. The slope conditions were observed in the field to carry out a slope stability assessment and are discussed further in Subsection 6.8 of this report.

The site is bordered by a municipal maintenance property to the north, Rideau Valley Drive followed by Rideau River to the east, Bankfield Road to the south, and a residential subdivision to the west.

4.2 Subsurface Profile

Generally, the subsurface soil profile at the test hole locations consists of topsoil underlain by a deposit of silty clay. The topsoil was underlain by sand and further by silty clay at BH 5-21, BH 6-21 and BH 7-21 and by fill underlain by glacial till at BH 8-21.

The silty clay deposit generally consisted of a hard to very stiff brown weathered crust to depths ranging between 1.5 and 5.2 m below ground surface. The brown silty clay was observed to be underlain by a stiff grey silty clay at BH 1-21, BH 3-21, BH 4-21, BH 5-21, BH 6-21, and BH 1-22.

Glacial till was encountered below the clay deposit at BH 2-21, BH 9-21, BH 1-22, and BH 2-22. The glacial till deposit was generally observed to consist of compact to dense brown silty sand with gravel, cobbles and boulders.

Practical refusal to augering was encountered at an approximate depth of 4.4 m at borehole BH7-21. Practical refusal to DCPT was encountered at an approximate depth of 15 m, 8.8 m, and 4.24 at BH 3-21, BH 5-21, and BH 2-22, respectively.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Field vane testing was completed within the silty clay deposits encountered in the test holes at the subject site. The shear strength values, as obtained from the field vane, were generally ranging between 50 to >200 kPa.

The remolded shear strength values as obtained from the field vane testing conducted in the test holes was observed to range between 20 to 80 KPa.

The sensitivity index of the encountered silty clay deposit was calculated based on the ratio between the undisturbed and remolded shear vane test measured in the field, for all the boreholes, and it was found to be generally below 4, indicating a normal sensitivity clay.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth.

Atterberg Limit and Shrinkage Tests

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at selected locations throughout the subject site. Based on the results of the Atterberg limits, the encountered silty clay deposit is classified as clay with high plasticity according to the USCS. The results of the Atterberg limits tests are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH1-SS3	1.5-2.1	54	24	30	35.57	CH
BH2-SS2	0.7-1.3	39	17	22	29.01	CL
BH3-SS4	2.2-2.9	51	20	32	34.52	CH
BH4-SS3	1.5-2.1	49	23	26	36.13	CL
BH5-SS2	0.7-1.3	54	22	31	30.27	CH
BH6-SS3	1.5-2.1	62	27	34	43.76	CH
BH7-SS4	2.2-2.9	65	28	37	55.67	CH
BH9-SS2	0.7-1.3	34	17	17	22.41	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CH: Inorganic Clay of High Plasticity CL: Inorganic Clay of Low Plasticity

The results of the shrinkage limit test indicate a shrinkage limit of 19.9% and a shrinkage ration of 2.05.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on four (4) selected soil samples. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis					
Test Hole	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH1-21	SS4	0.0	2.4	50.0	47.6
BH4-21	SS2	0.0	39.1	30.5	30.4
BH6-21	SS4	1.2	91.3	7.5	
BH9-21	SS3	21.5	52.6	25.9	

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and piezometers installed at the borehole locations on May 26, 2021. The measured groundwater levels noted at that time are presented in Table 3.

Table 3 – Summary of Groundwater Levels				
Test Hole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Dated Recorded**
		Depth (m)	**Elevation (m)**	
BH1-21	88.26	1.72	86.54	May 26, 2021
BH2-21	89.55	Dry	N/A	May 26, 2021
BH3-21	87.89	4.99	82.90	May 26, 2021
BH4-21	88.11	1.90	86.21	May 26, 2021
BH5-21	85.36	2.26	83.10	May 26, 2021
BH6-21	85.35	1.98	83.37	May 26, 2021
BH7-21	87.56	Dry	N/A	May 26, 2021
BH8-21	91.32	3.58	87.74	May 26, 2021
BH9-21	90.52	3.77	86.75	May 26, 2021
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and are referenced to a geodetic datum.				

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed residential development. It is anticipated that the proposed buildings will be supported by shallow foundations placed over very stiff brown silty clay, compact to dense glacial till or an approved engineered fill pad.

Permissible grade raise recommendations are discussed in Subsection 5.3. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements under buildings. However, it should be noted that lightweight fill is not permitted under the ROWs.

Due to the presence of a low to medium sensitivity marine silty clay deposit across the site, the proposed development will be subjected to tree planting setback restrictions, as further detailed under Subsection 6.9.

The above and other considerations are discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant amounts of organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other remnants of construction debris from existing structures should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil (including the plastic sensitive silty clay deposit) could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx

Proof Rolling

For the proposed driveways and roadways, proof rolling of the subgrade is required in areas where the existing fill, free of significant amounts of organics and deleterious materials, is encountered. It is recommended that the subgrade surface be proof rolled **under dry conditions and above freezing temperatures** by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by the geotechnical consultant at the time of construction.

In-Fill Recommendations – Rear Yard of Lot 5 and Lot 6

It is understood that in-filling the face of the slope within the rear yards of Lot 5 and Lot 6 to match the surrounding slope and since the existing drainage swale feature will be in-filled by the proposed development. Based on this, it is recommended the following fill placement recommendations be followed for reinstating the slope throughout the swale footprint.

- All existing topsoil, organic soils and deleterious fill and materials should be stripped from the area that will be in-filled.

- It is recommended fill be placed upon benches excavated throughout the swale area to provide adequately wide surfaces for the placement and compaction of the fill material. The benches are recommended to be shaped to provide a 1.5H:1V profile extending upwards and away from the bottom of the swale and in a stepped fashion with maximum 500 mm high steps.

- It is recommended that the fill consist of a workable, site-generated brown silty clay fill placed in maximum 300 mm thick loose lifts **under dry conditions and in above freezing temperatures** to in-fill the slope. Every lift should be adequately compacted using a vibratory sheepsfoot roller and approved by Paterson personnel during placement.

- The grading along the slope should be provided to match the surrounding slope and to a maximum steepness of 3H:1V. In the even that adjacent grading is steeper than 3H:1V, it is recommended that the steepness of the in-fill be provided as 3H:1V.

- A minimum 300 mm thick layer of clayey topsoil mixed with hardy grass seed or hydroseed (weather permitting). All efforts should be taken to retain all vegetation surrounding the in-fill area throughout the in-fill effort.

- **Inspections During Construction:** Periodic inspections during the backfilling operation should be completed by Paterson personnel to confirm the above noted recommendations are undertaken as recommended at the time of construction.

  Reference should be made to Section 2A and 2B which consider the proposed grading in-fill as described herein.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Spread Foundations)

Based on the subsurface profile encountered, it is anticipated that the residential dwellings will be founded on shallow foundations placed on very stiff, brown silty clay, compact to dense glacial till or approved engineered fill. Using continuously applied loads, footings for the proposed development can be designed using the bearing resistance values presented in Table 4.

Table 4 - Bearing Resistance Values

Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Very Stiff Brown Silty Clay	150	225
Compact to Dense Glacial Till	150	225
Engineered Fill Pad	150	225

Note: Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, can be designed for silty clay bearing mediums using the above noted bearing resistance values.

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, whether in-situ or not, have been removed, prior to placement of concrete for footings. An engineered fill pad may be required where the existing fill is located at the proposed founding elevation for buildings located throughout southeastern portion of the subject site. It is recommended that the existing fill, where encountered at the design founding elevation, be sub-excavated to a suitable native, in-situ soil bearing medium.

The area may be raised to the proposed founding elevation using an imported engineered fill such as OPSS Granular B Type II placed in 300 mm thick loose lifts and compacted to 98% of the materials SPMDD. The placement of this engineered fill layer should be reviewed and approved at the time of construction by Paterson personnel.

The bearing resistance values will be reviewed against the grading plan and boreholes once available. Bearing resistance values for footing design should be confirmed on a per lot basis by the geotechnical consultant at the time of construction

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the in-situ bearing medium soils or engineered fill when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, permissible grade raise restrictions are recommended for all structures placed on a silty clay bearing medium. The recommended grade raise restrictions are shown on Drawing PG5828-3 – Permissible Grade Raise Plan included in Appendix 2. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements of the soils surrounding the buildings. However, it should be noted that lightweight fill is not permitted under the ROWs.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered at this site. The soils encountered at the subject site consist of silty clays, which are cohesive in nature. These soils were evaluated for liquefaction susceptibility in accordance with the criteria prepared by Bray at al. 2004 which determines that all soils with a plasticity index exceeding 20% are not liquifiable (Figure 1). In general, the plasticity index results completed on samples taken from the silty clay layer were found to be above 20. Therefore, soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

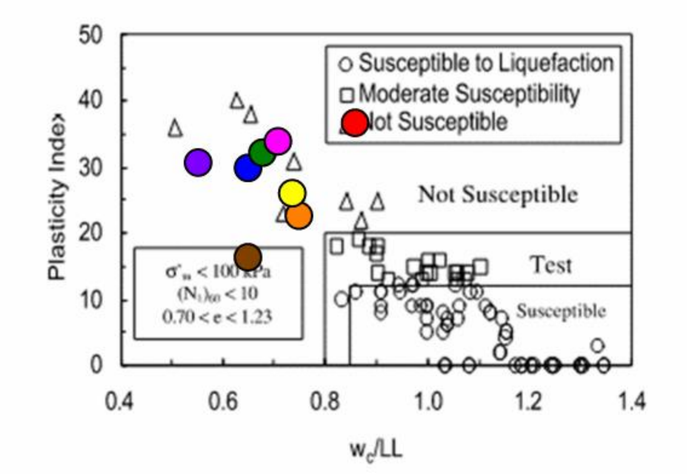

Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

Reference should be made to the Atterberg Limits Results sheet in Appendix 1 which provides the test results referenced in the above-noted chart.

5.5 Basement Slab Construction

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the native soils or approved engineered fill pad will be considered an acceptable subgrade upon which to commence backfilling for floor slab construction. It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone crushed stone.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Types I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings). All backfill material within the footprint of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of driveways and local residential streets and roadways. The proposed pavement structures are presented in Tables 5 and 6 on the following page.

Table 5 – Recommended Pavement Structure – Driveways

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

Table 6 – Recommended Pavement Structure – Local Residential Roadways

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course –** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II
SUBGRADE – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity. Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed residential development. The system should consist of a 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings, such as those for isolated exterior piers and retaining walls, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through a hard to very stiff silty clay. Where excavations are above the groundwater level to a depth of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavations below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring systems should be used.

The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Deep excavation is not anticipated for the proposed residential units. However, if deep services are anticipated at the subject site, then deep service trenches in excess of 3 m should be completed using a temporary shoring system, such as stacked trench boxes in conjunction with steel plates, designed by a structural engineer. The trench boxes should be installed to ensure that the excavation sidewalls are tight to the outside of the trench boxes and that the steel plates are extended below the base of the excavation to prevent basal heave, if required.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

The backfill material within the frost zone (about 1.5 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations should be carried in a manner to avoid the introduction of frozen materials, snow, or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a low to slightly aggressive corrosive environment.

6.8 Slope Stability Assessment

The west and north boundaries of the site are adjacent to a valley of Wilson Cowan Drain to Mud Creek and the main channel of Mud Creek, respectively. The existing slope conditions were reviewed by Paterson field personnel as part of the geotechnical investigation on May 19, 2021. Four (4) slope cross-sections were studied as the worst-case scenarios. The cross sections were analyzed considering existing and post-development conditions, considering an average grade raise of approximately 2m. The cross-section locations are presented on Drawing PG5828-1 - Test Hole Location Plan in Appendix 2.

Field Observations

The existing slope conditions along the north and west boundaries of the site are detailed below. Reference may also be given to photographs taken as part of our site review in Appendix 2.

Slope Conditions Along the Western Boundary

The existing slope along the western portion of the subject site was generally observed to be covered with well rooted vegetation across its surface. The slope was observed to be approximately 4 m high and appeared to have a profile ranging between 2.5H:1V and 4H:1V. An approximately 4 to 15 m wide valley floor was observed across the creek length which appeared to decrease up to 2 m along some bends.

The width of the Wilson Cowan Drain was noted to be between 1.5 m and 2.0 m wide long its length and typically decreased to between 1.2 and 1.5 m at its bends. At the time of our visit, the water level appeared to be up to 1.0 m in depth in deeper areas and bends, and no more than 150 mm in depth in shallower areas.

The majority of the Wilson Cowan Drain bed appeared to be covered by an in-situ stiff grey silty clay. The bank channels were generally observed to be well vegetated such that bank material did not appear to be exposed directly to stream flow. Signs of erosion were documented by the project geo-fluvial consultant and should be referred to in the associated report

The creek was generally observed to consist of Wilson Cowan Drain to the Mud Creek channel and discharged into the main channel along the north-west portion of the subject site.

Slope Conditions Along the Northern Boundary

The existing slope bordering Mud Creek to the north of the subject site is generally heavily vegetated with brush and some trees. Mud Creek generally consists of an active watercourse which flows from west to east and discharges into the Rideau River located to the east of Rideau Valley Drive. The majority of the channel was observed to be fronted onto by a valley floor with the exception of the area of Cross Section C-C which was observed to be fronted onto by a slope at the creeks bend. The majority of the channel banks were observed to be affected by active erosion and were exposed directly to stream flow. Additional signs of erosion consisted of exposed tree roots, fallen trees, over-steepening and under-cutting of the bank at bends in the creek alignment.

The width of the creek was noted to be between 4.0 m and 6.0 m wide and decreased to widths of approximately 4.0 m at its bends. At the time of our visit, the water level appeared to be approximately 600 mm in depth across the majority of the channel’s footprint.

The slopes’ gradient was observed to slope downward towards Mud Creek gradually at an approximately 2H:1V to 15H:1V grade.

Slope Conditions Along the North-East Boundary

The existing slope bordering the area along the north-east of the subject site is generally heavily vegetated with brush and trees. The area appeared to consist of a tributary between the Mud Creek and the Rideau River. An approximately 50 m wide valley floor was observed across separating the main channel and the tributary. The slope fronting onto the channel or the valley floor was observed to be approximately 2.5 to 4 m high and appeared to have a profile ranging between 2.5H:1V and 4H:1V.

The width of watercourse was noted to be between 5 m and 20 m wide along its length and typically decreased to approximately 10 m at its bends. At the time of our visit, the water level appeared to be up to 300 mm in depth in deeper areas and bends, and no more than 150 mm in depth in shallower areas. The majority of the watercourse’s bed appeared to be covered by an in-situ stiff grey silty clay.

The bank channels were generally observed to be well vegetated with well-rooted vegetation and mature trees. However, some erosion consisting of exposed banks had been noted along the toe of the slope throughout bend areas.

Slope Stability Analysis

The analysis of the stability of the upper slope was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

Subsoil conditions at the cross-section locations were determined based on test holes coverage conducted within the subject site. The subsurface profile across the proposed subdivision was observed to be generally consistent. Therefore, the soil profile used in the slope stability analysis for all cross sections was based on boreholes BH 1-21, BH 4-21, BH 5-21, and BH 6-21, which were in proximity to the watercourse and drain area. The soil profile considered in the slope stability analysis consists of 3m of very stiff brown silty clay crust underlain by firm grey silty clay. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis and exiting at the toe of the slope and across the creek section. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis and exiting at the toe of the slope and across the creek section.

Table 7 – Effective Stress Soil Parameters (Static – Drained Analysis)

Soil Layer	Depth (m)	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay/Site Excavated Silty Clay	-	17	33	5
Grey Silty Clay	4-5	16	33	10
Glacial Till	11	20	33	0

Table 8– Total Stress Soil Parameters (Seismic - Undrained Analysis)

Soil Layer	Elevation of Top of Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay/ Site Excavated Silty Clay	-	17	-	150
Grey Silty Clay	4-5	16	-	65
Glacial Till	11	20	33	0

Static Loading Analysis

The results are shown in Figures 2, 4, 6, 8, 10, 12, 14, & 16 in Appendix 2. The results indicate a slope with a factor of safety of 2.1 and 2.4 at Section A and Section B, respectively. The results also indicate slopes with factors of safety less than 1.5 beyond the top of slope at Section C and D. Based on these results, a stable slope setback varying between 1.3 and 5.3 m from the top of the slope are required to achieve a factor of safety of 1.5 for the limit of the hazard lands in the area of Sections C and D.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed. A horizontal acceleration of 0.16g was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the analyses including seismic loading are shown in Figures 3, 5, 7, 9, 11, 13, 15, and 17 in Appendix 2. The results indicate a slope with a factor of safety greater than 1.1 at all sections. However, it should be noted that the stable slope setback associated with our static loading analysis governs the required stable slope setback required for static conditions.

Toe Erosion and Access Allowances

Based on the soil profiles encountered at the borehole locations and the soil encountered throughout the watercourse, a stiff grey silty clay is anticipated to be subject to erosion activity by the watercourse within the main valley corridor.

Based on the anticipated soils, and the nature of the existing watercourse and drain, a toe erosion allowance of 5 m, and as advised in geo-fluvial study, may be applied from the watercourse edge for Mud Creek Watercourse and Wilson Cowan Drain.

Further, an access allowance of 6 m is required from the top of slope or geotechnical setback (where applicable). In areas where the watercourse edge has meandered to within 5 m of the toe of the existing slope, the toe erosion and access allowances should be applied in addition to geotechnical setback limit from the top of slope.

Limit of Hazard Lands

Based on the above, a setback taken from the top of the current slope has been provided as based on the above-noted observations and analysis. Reference should be made to Drawing PG5828-1 – Test Hole Location Plan for the proposed Limit of Hazard Lands setback for development considerations at the subject site. The existing vegetation on the slope faces should not be removed as it contributes to the stability of the slope and reduces erosion.

6.9 Landscaping Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed review of the soils in the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. The results of our Atterberg limit and sieve testing are presented in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay.

The following tree planting setbacks are recommended for low to medium sensitivity silty clay deposits throughout the subject site.

Large trees (mature tree height over 14 m) can be planted at the subject site provided a tree to foundation setback equal to full mature height of the tree can be provided (e.g., in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature height up to 7.5 m) and medium size trees (mature height 7.5 to 14 m), provided that the conditions noted below are met:

- ❒ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the center of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

- ❒ A small tree must be provided with a minimum 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ❒ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- ❒ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- ❒ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

Swimming Pools

The in-situ soils are considered to be acceptable for swimming pools. Above ground swimming pools must be placed at least 4 m away from the residence foundation and neighboring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Aboveground Hot Tubs

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

Installation of Decks or Additions

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

In addition to the above recommendations, it should be noted that the following is should be considered for the proposed development:

- ❒ It is important to avoid directing uncontrolled water towards the slope (drainage, gutter, septic field, pool & hot tub drainage, etc.)

- ❒ It is important to avoid overloading the top of the slope (backfill, fill, miscellaneous waste, grass cuttings, branches, leaves, snow, etc.)

- ❒ It is important to avoid excavating at the base of the slope.

- ❒ It is important to maintain a healthy native vegetation cover.

- ❒ Any future additions, such as aboveground swimming pools or accessory buildings, should entail reassessment of slope stability unless this has been pre-confirmed via supplementary slope stability analyses during the design stage.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined.

- Review detailed grading and site servicing plan(s) from a geotechnical perspective.
- Review detailed landscaping plan (s) from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to placing backfilling material.
- Observation of clay seal placement at specified locations.
- Field density tests to determine the level of compaction has been achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Uniform Development or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 19, 2024
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Mrunmayi Anvekar, M.Eng.

Drew Petahtegoose, P.Eng.

Report Distribution:

❑ Uniform Developments (email copy)
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN-SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

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9 Auriga Drive, Ottawa, Ontario, K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5828

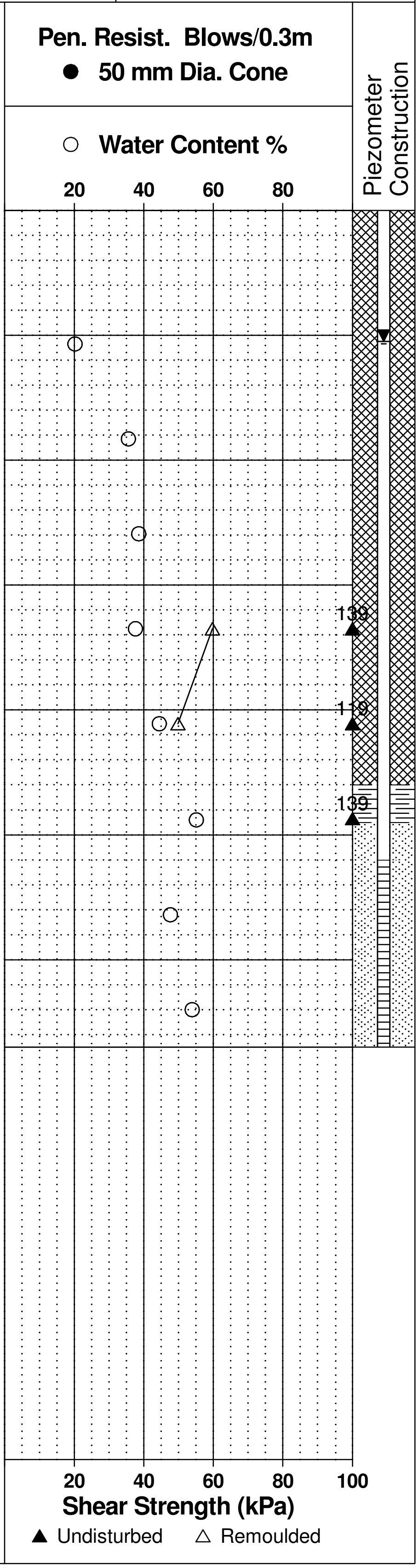
REMARKS

HOLE NO. BH 1-21

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.26
TOPSOIL 0.30		AU	1				
		SS	2	100	13	1	87.26
		SS	3	100	6		
						2	86.26
		SS	4	100	8		
						3	85.26
Hard to very stiff, brown **SILTY CLAY,** trace sand		SS	5	100	7		
						4	84.26
		SS	6	100	5		
		SS	7	83	P		
						5	83.26
- grey by 5.2m depth							
		SS	8	83	P		
						6	82.26
6.70		SS	9	83	P		
End of Borehole							
(GWL @ 1.05m - May 26, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
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DATUM Geodetic

FILE NO. PG5828

REMARKS

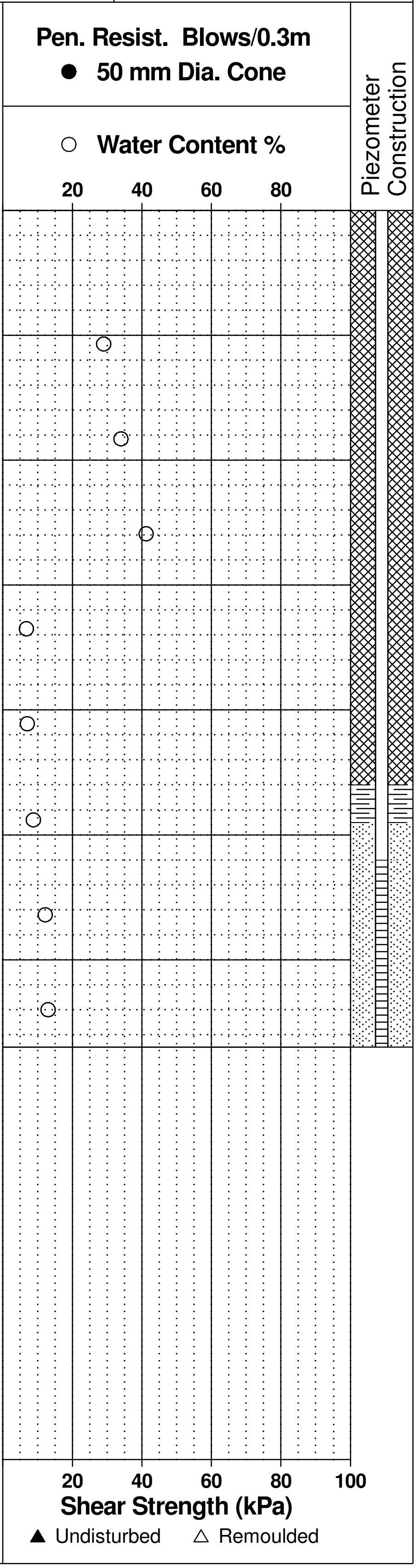
HOLE NO. BH 2-21

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.30	AU	1		
Hard to very stiff, brown **SILTY CLAY,** some to trace sand	SS	2	100	11
	SS	3	58	8
2.80	SS	4	60	8
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders, trace clay	SS	5	67	46
	SS	6	62	27
	SS	7	60	21
	SS	8	58	15
6.70	SS	9	50	10
End of Borehole (BH dry - May 26, 2021)				

DEPTH (m)	ELEV. (m)
0	89.55
1	88.55
2	87.55
3	86.55
4	85.55
5	84.55
6	83.55



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
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REMARKS

BORINGS BY Track-Mount Power Auger

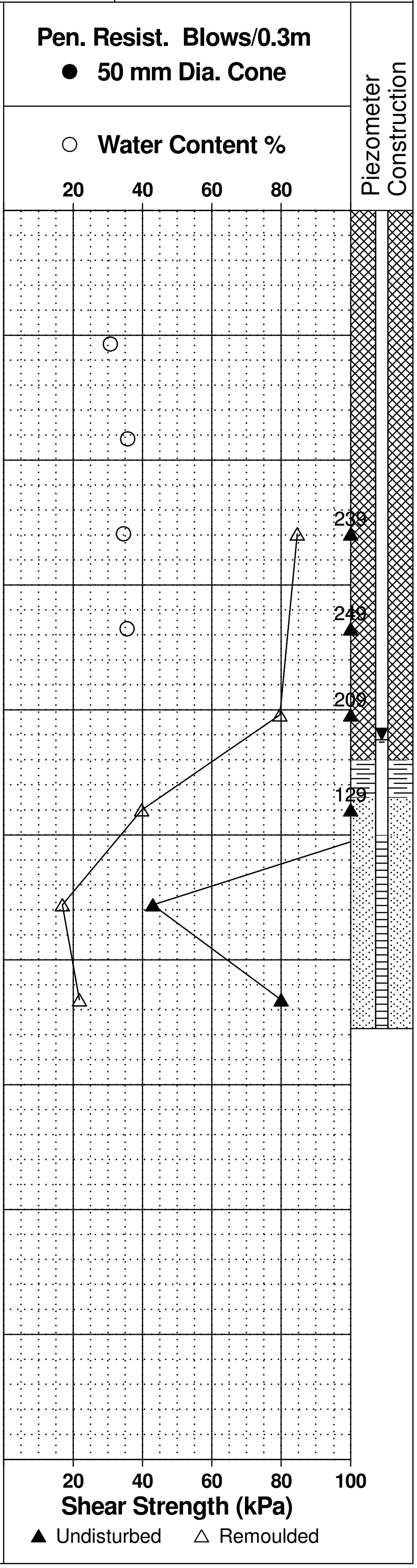
DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.25				
Hard to very stiff, brown **SILTY CLAY,** trace sand - sand content decreasing with depth - stiff and grey by 5.2m depth 6.55	AU	1		
	SS	2	83	9
	SS	3	83	5
	SS	4	83	P
	SS	5	83	P
Dynamic Cone Penetration Test commenced at 6.55m depth. Cone pushed to 11.0m depth				

DEPTH (m)	ELEV. (m)
0	87.89
1	86.89
2	85.89
3	84.89
4	83.89
5	82.89
6	81.89
7	80.89
8	79.89
9	78.89
10	77.89



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
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DATUM Geodetic

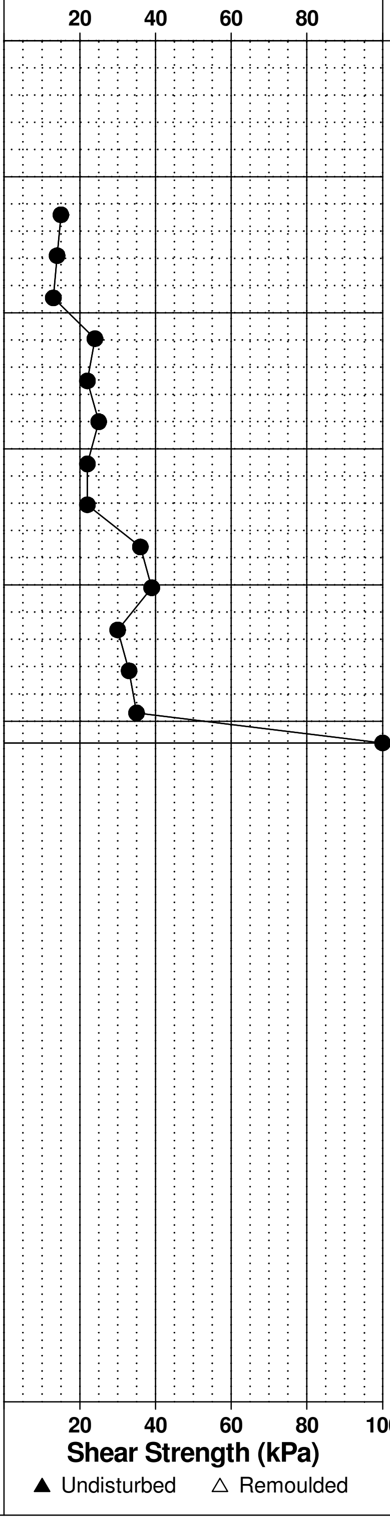
FILE NO. PG5828

REMARKS

HOLE NO. BH 3-21

BORINGS BY Track-Mount Power Auger **DATE** May 20, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						10	77.89		
						11	76.89		
						12	75.89		
						13	74.89		
						14	73.89		
						15	72.89		
15.16									
End of Borehole Practical refusal to DCPT at 15.16m depth (GWL @ 4.24m - May 26, 2021)									



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SOIL PROFILE AND TEST DATA

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Proposed Residential Development
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DATUM Geodetic

REMARKS

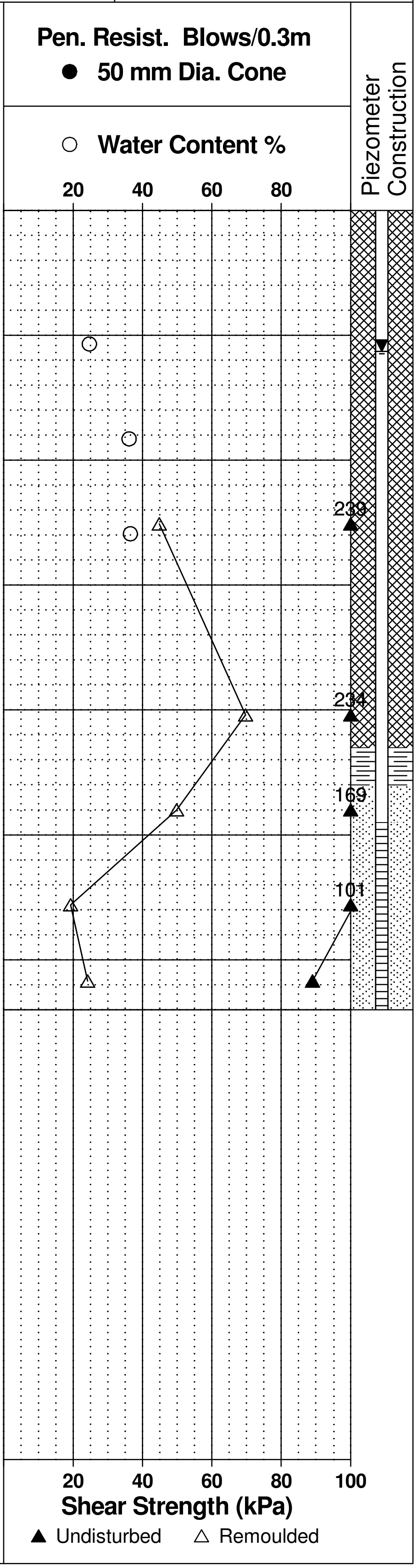
BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

FILE NO. PG5828

HOLE NO. BH 4-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	88.11
TOPSOIL 0.36		AU	1				
Hard to very stiff, brown **SILTY CLAY,** some silty sand - sand content decreasing with depth		SS	2	83	8	1	87.11
		SS	3	83	5	2	86.11
		SS	4	100	6	3	85.11
5.18						4	84.11
						5	83.11
Stiff, grey **SILTY CLAY** 6.40						6	82.11
End of Borehole (GWL @ 1.13m - May 26, 2021)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

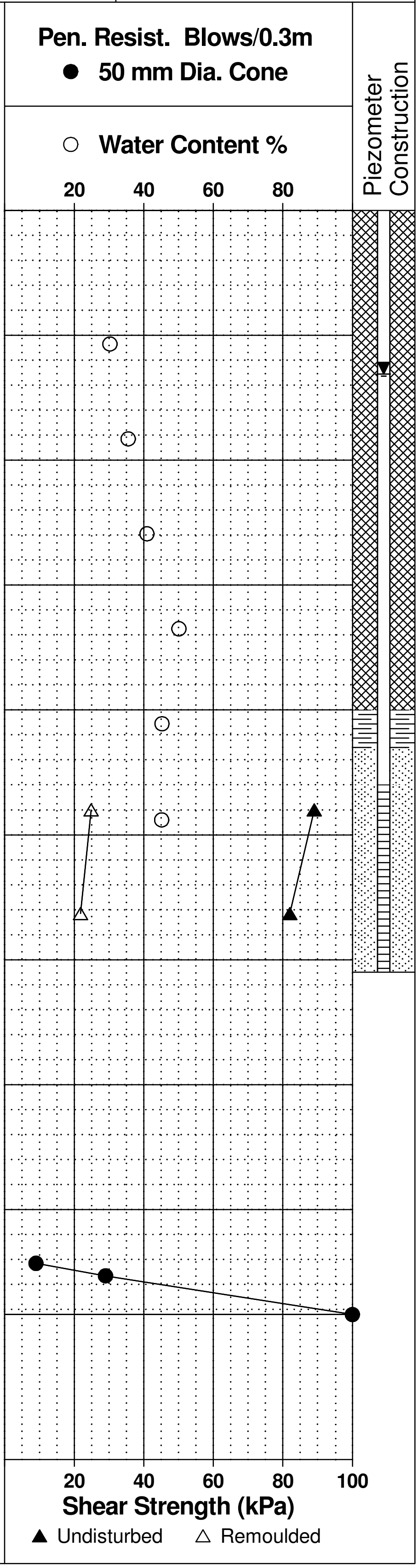
FILE NO. PG5828

HOLE NO. BH 5-21

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.30
Compact, brown **SILTY SAND**	0.60
Hard to very stiff, brown **SILTY CLAY** - stiff and grey by 4.3m depth	6.10
Dynamic Cone Penetration Test commenced at 6.10m depth. Cone pushed to 8.43m depth	8.84
End of Borehole Practical DCPT refusal at 8.84m depth. (GWL @ 1.31m - May 26, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	10
SS	3	58	11
SS	4	83	9
SS	5	100	7
SS	6	100	5
SS	7	100	P

DEPTH (m)	ELEV. (m)
0	85.36
1	84.36
2	83.36
3	82.36
4	81.36
5	80.36
6	79.36
7	78.36
8	77.36



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SOIL PROFILE AND TEST DATA

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FILE NO. **PG5828**

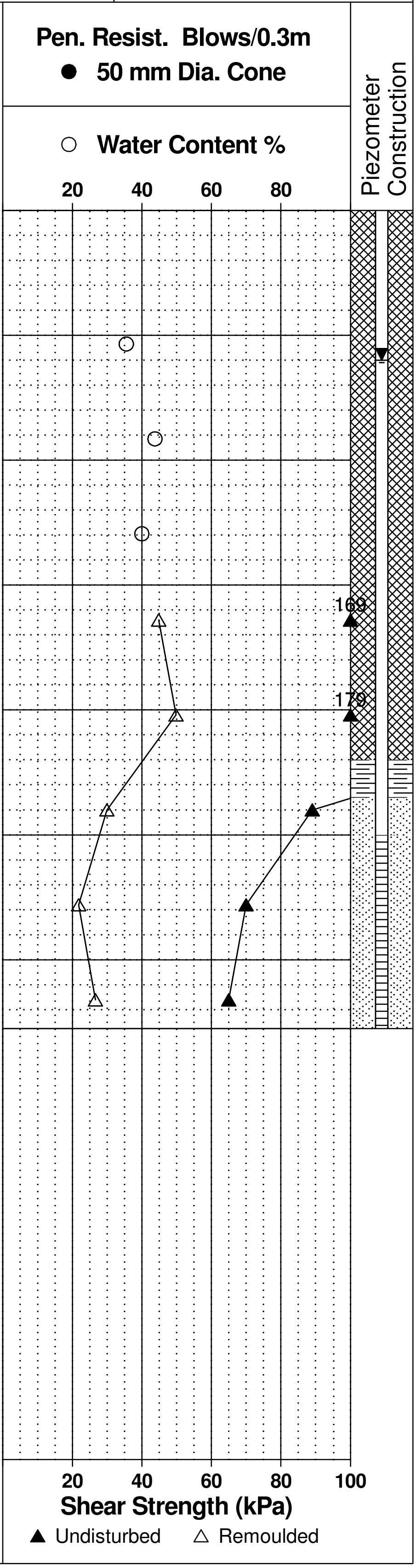
REMARKS

HOLE NO. **BH 6-21**

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	85.35
TOPSOIL 0.30							
Brown **SILTY SAND,** trace clay 0.60		AU	1				
						1	84.35
		SS	2	83	6		
		SS	3	83	6		
						2	83.35
		SS	4	83	5		
Very stiff to stiff, brown **SILTY CLAY,** trace sand							
						3	82.35
- sand content decreasing with depth							
						4	81.35
- grey by 4.6m depth							
						5	80.35
						6	79.35
6.55							
End of Borehole							
(GWL @ 1.20m - May 26, 2021)							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

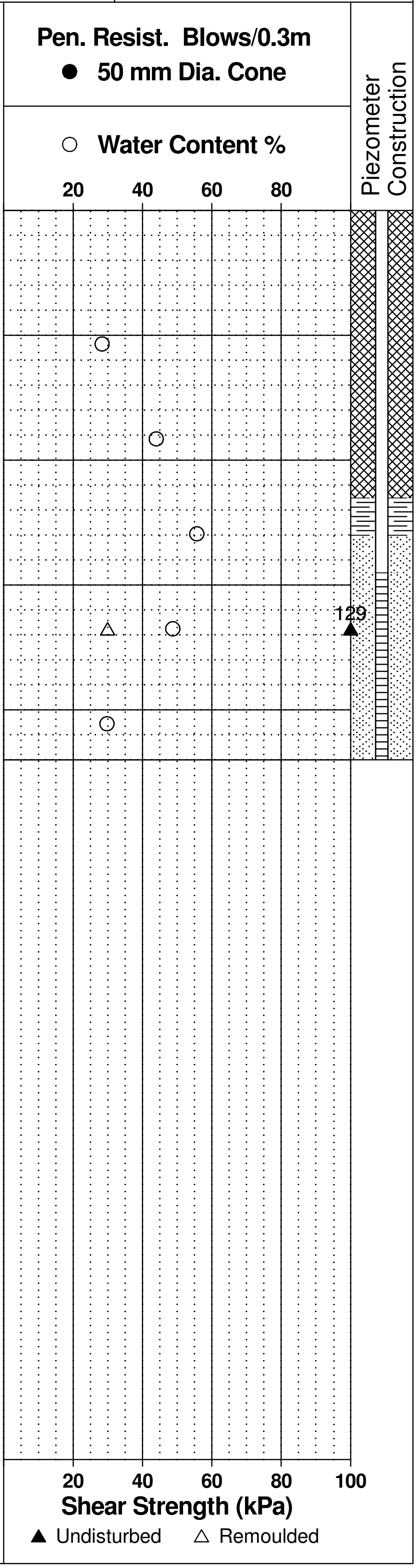
HOLE NO. BH 7-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	87.56
TOPSOIL 0.30							
Brown **SILTY SAND,** trace clay 0.76		AU	1				
Very stiff, brown **SILTY CLAY,** trace sand - sand content decreasing with depth - some sand, trace gravel by 4.1m depth 4.40		SS	2	75	7	1	86.56
		SS	3	83	7	2	85.56
		SS	4	83	3		
		SS	5	100	P	3	84.56
		SS	6	87	8	4	83.56

End of Borehole

Practical refusal to augering at 4.4m depth

(BH dry - May 26, 2021)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

REMARKS

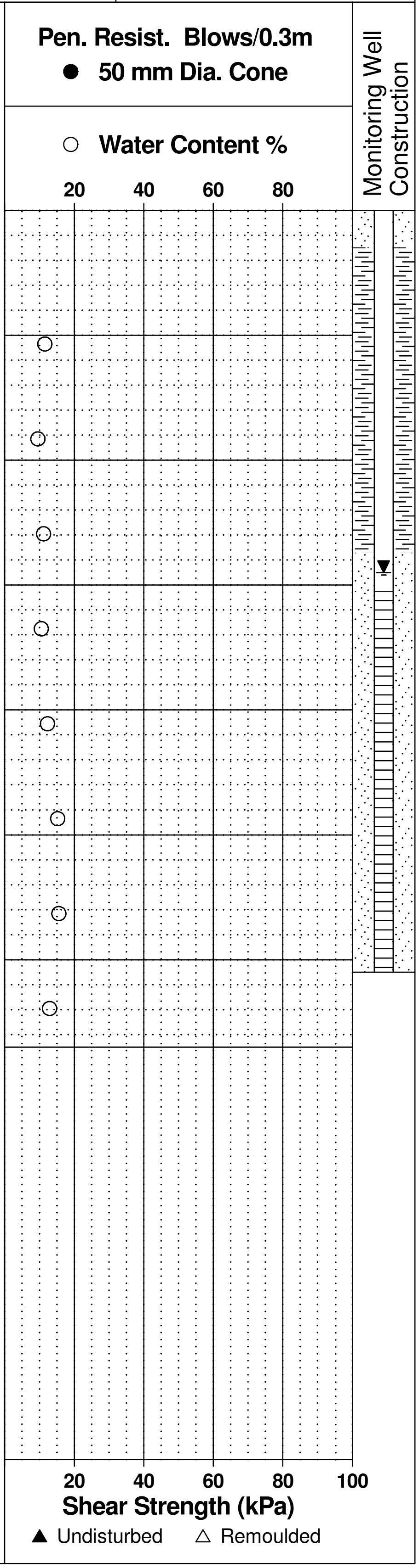
BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. PG5828

HOLE NO. BH 8-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	91.32
TOPSOIL 0.20						
FILL: Brown silty sand, some gravel, trace topsoil	AU	1				
1.07	SS	2	62	30	1	90.32
	SS	3	75	34	2	89.32
	SS	4	62	27	3	88.32
	SS	5	75	32		
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders	SS	6	62	39	4	87.32
	SS	7	50	27	5	86.32
	SS	8	42	26	6	85.32
	SS	9	42	21		
6.70						
End of Borehole						
(GWL @ 2.90m - May 26, 2021)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

DATUM Geodetic

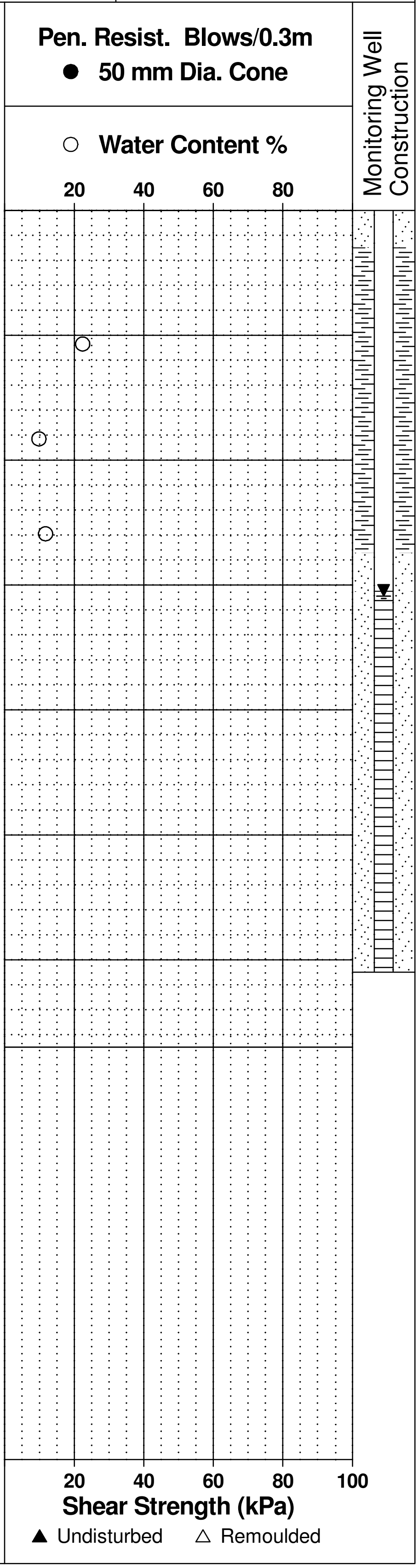
REMARKS

BORINGS BY Track-Mount Power Auger **DATE** May 20, 2021

FILE NO. PG5828

HOLE NO. BH 9-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.52
TOPSOIL 0.30						
Stiff, brown **SILTY CLAY,** some to trace sand	AU	1				
	SS	2	83	12	1	89.52
1.52						
					2	88.52
					3	87.52
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders	SS	3	75	23		
					4	86.52
					5	85.52
					6	84.52
	SS	4	75	32		
6.70 End of Borehole (GWL @ 3.09m - May 26, 2021)						



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.
p	-	Push spoon sampling

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

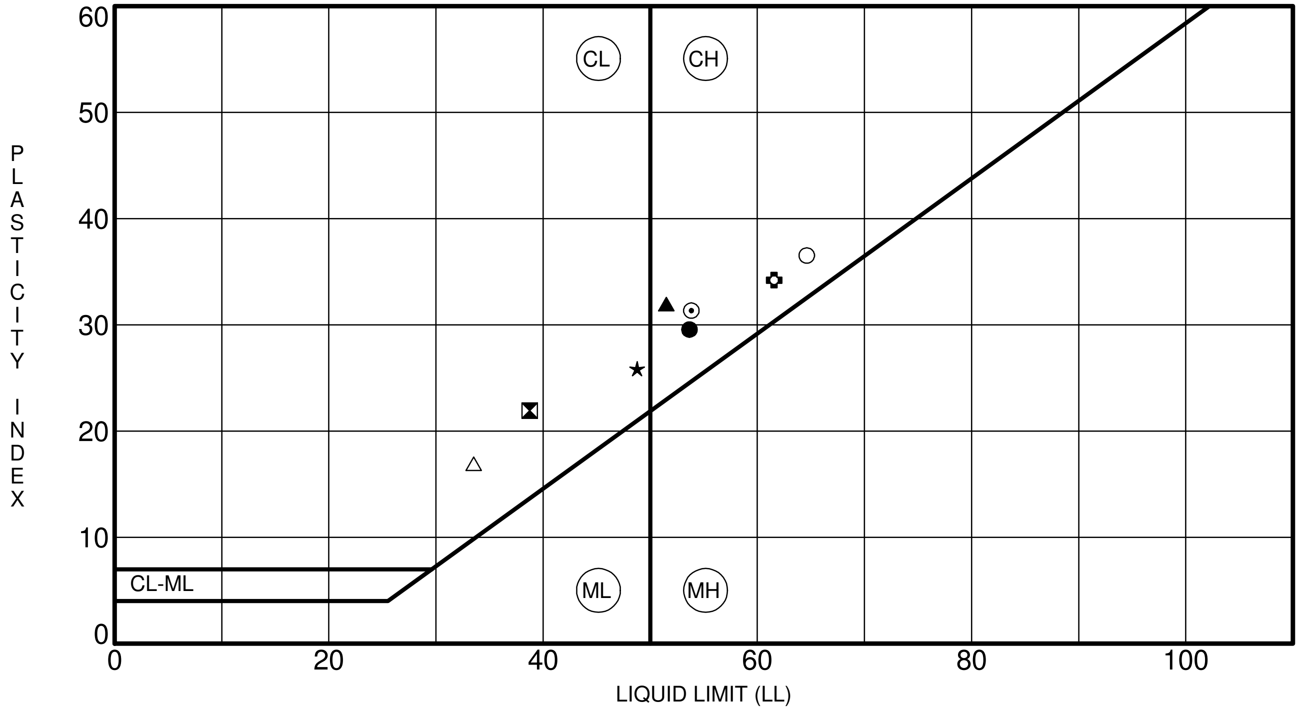
CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.



Specimen Identification				LL	PL	PI	Fines	Classification
●	**BH 1-21**	🔵	**SS 3**	**54**	**24**	**30**		**CH - Inorganic clays of high plasticity**
⧗	**BH 2-21**	🟠	**SS 2**	**39**	**17**	**22**		**CL - Inorganic clays of low plasticity**
▲	**BH 3-21**	🟢	**SS 4**	**51**	**20**	**32**		**CH - Inorganic clays of high plasticity**
★	**BH 4-21**	🟡	**SS 3**	**49**	**23**	**26**		**CL - Inorganic clays of low plasticity**
⊙	**BH 5-21**	🟣	**SS 2**	**54**	**22**	**31**		**CH - Inorganic clays of high plasticity**
✿	**BH 6-21**	🟣	**SS 3**	**62**	**27**	**34**		**CH - Inorganic clays of high plasticity**
○	**BH 7-21**	🔴	**SS 4**	**65**	**28**	**37**		**CH - Inorganic clays of high plasticity**
△	**BH 9-21**	🟤	**SS 2**	**34**	**17**	**17**		**CL - Inorganic clays of low plasticity**

CLIENT **Uniform Urban Developments**

FILE NO. **PG5828**

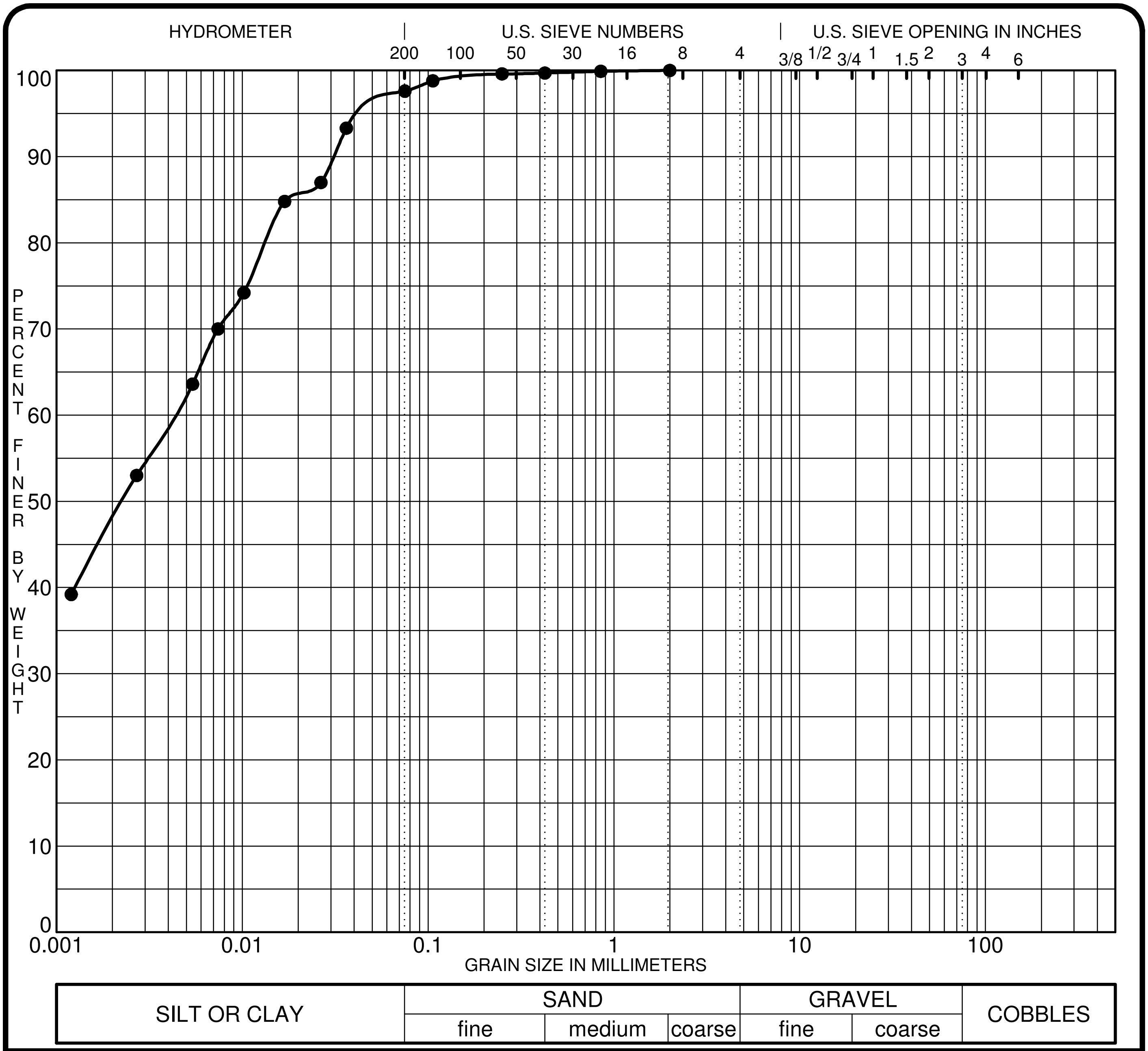
PROJECT **Geotechnical Investigation - Proposed Residential Development**

DATE **20 May 21**

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario, K2E 7T9

ATTERBERG LIMITS' RESULTS



Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 1-21 SS 4**	**CH - Inorganic clays of high plasticity**						
☒								
▲								
★								

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 1-21 SS 4**	**2.00**	**0.00**			**0.0**	**2.4**	**97.6**	
☒									
▲									
★									

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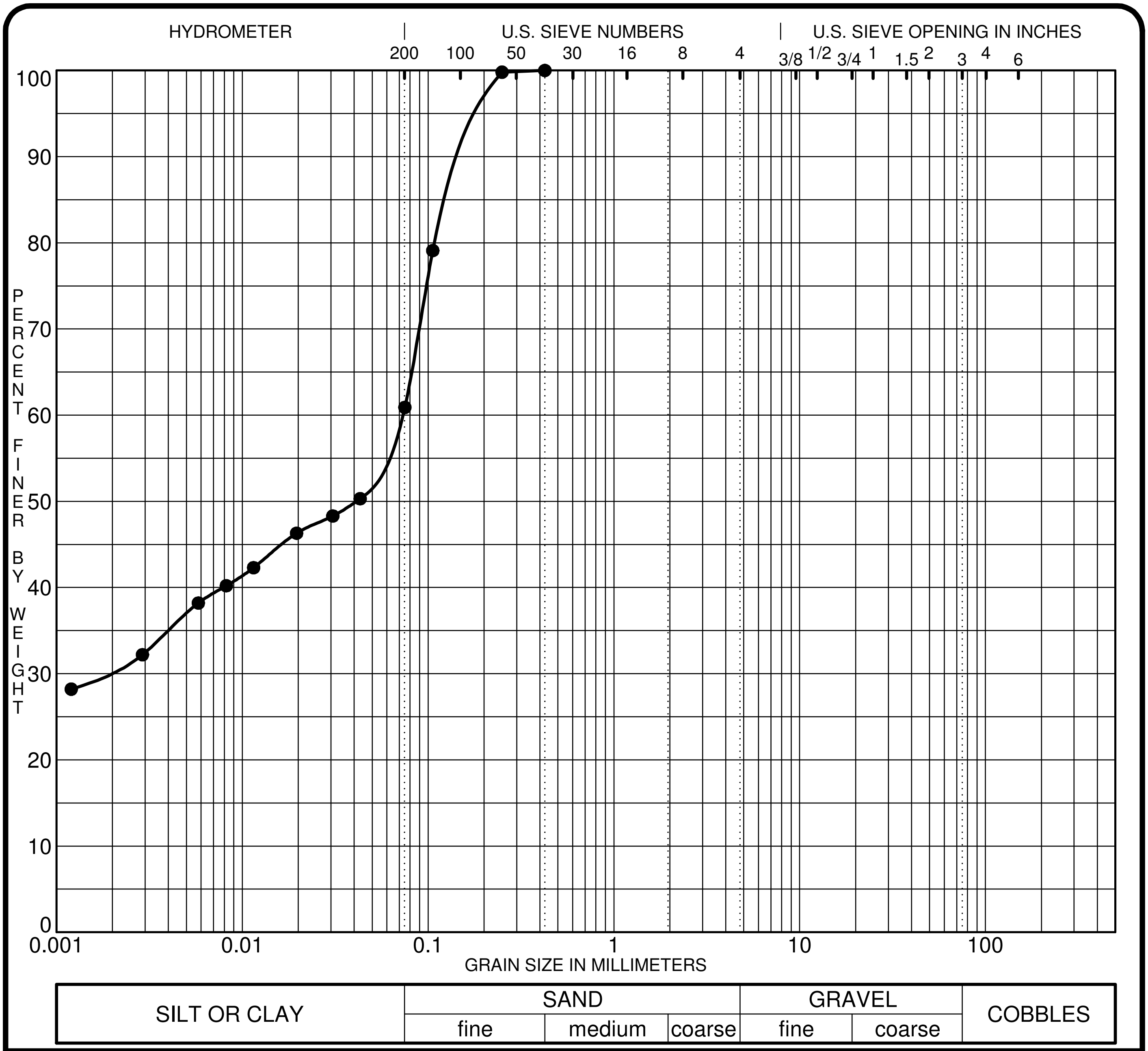
GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 4-21**	**SS 2**	**CL - Inorganic clays of low plasticity**						
⌧									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 4-21**	**SS 2**	**0.43**	**0.07**	**0.002**		**0.0**	**39.1**	**60.9**	
⌧										
▲										
★										

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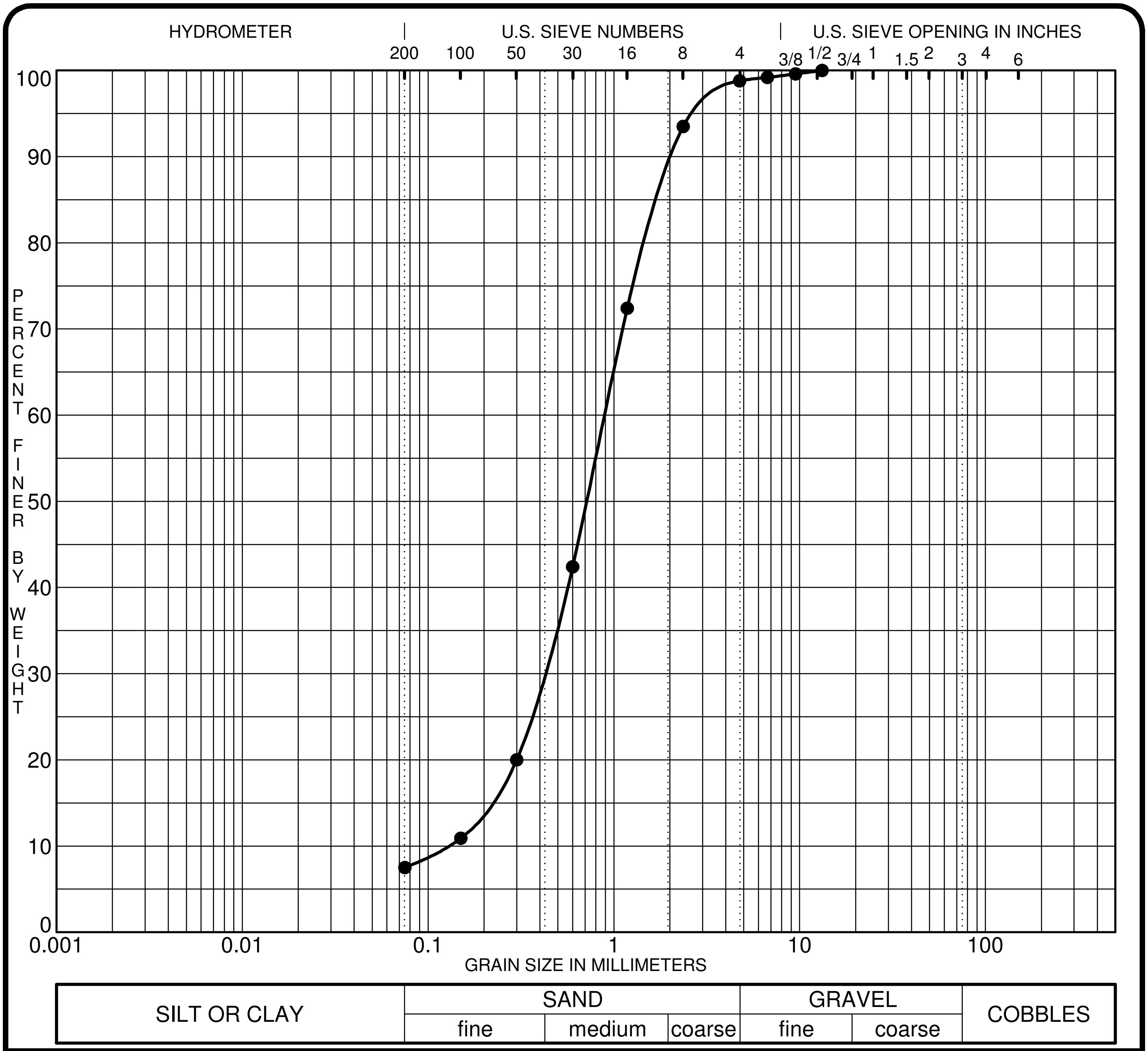
GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 6-21**	**SS 4**	**CH - Inorganic clays of high plasticity**					**1.50**	**7.1**
⌧									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 6-21**	**SS 4**	**13.20**	**0.89**	**0.409**	**0.1249**	**1.2**	**91.3**	**7.5**	
⌧										
▲										
★										

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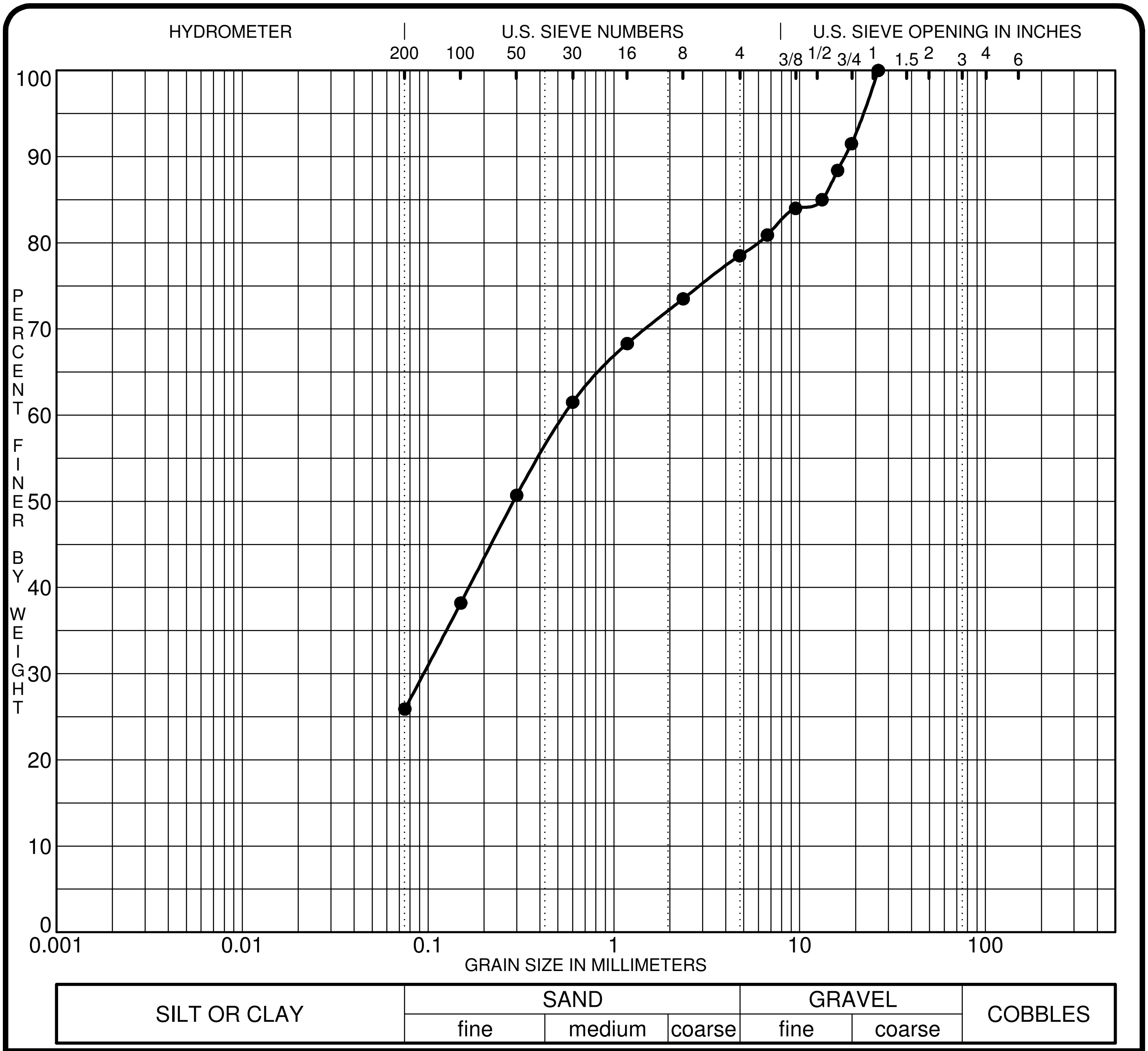
GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 9-21**	**SS 3**							
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 9-21**	**SS 3**	**26.50**	**0.54**	**0.094**		**21.5**	**52.6**	**25.9**	
⊠										
▲										
★										

CLIENT **Uniform Urban Developments**

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PROJECT **Geotechnical Investigation - Proposed Residential Development**

DATE **20 May 21**

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GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29757

Report Date: 28-May-2021

Order Date: 21-May-2021

Project Description: PE5828

	Client ID:	BH3-21, SS3	-	-	-
	Sample Date:	20-May-21 09:00	-	-	-
	Sample ID:	2121708-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.54	-	-	-
Resistivity	0.10 Ohm.m	59.3	-	-	-
Anions					
Chloride	5 ug/g dry	9	-	-	-
Sulphate	5 ug/g dry	23	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 TO FIGURE 17 – SLOPE STABILITY ANALYSIS CROSS SECTIONS

PHOTOGRAPHS FROM SITE VISIT – MAY 19, 2021

DRAWING PG5828-1 – TEST HOLE LOCATION PLAN

DRAWING PG5828-3 – PERMISSIBLE GRADE RAISE PLAN

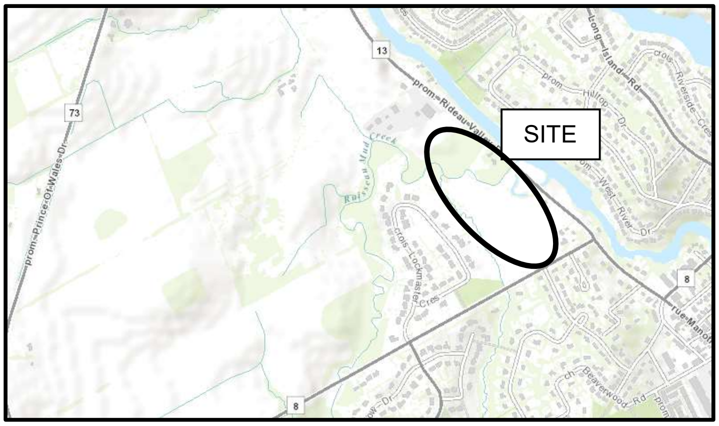
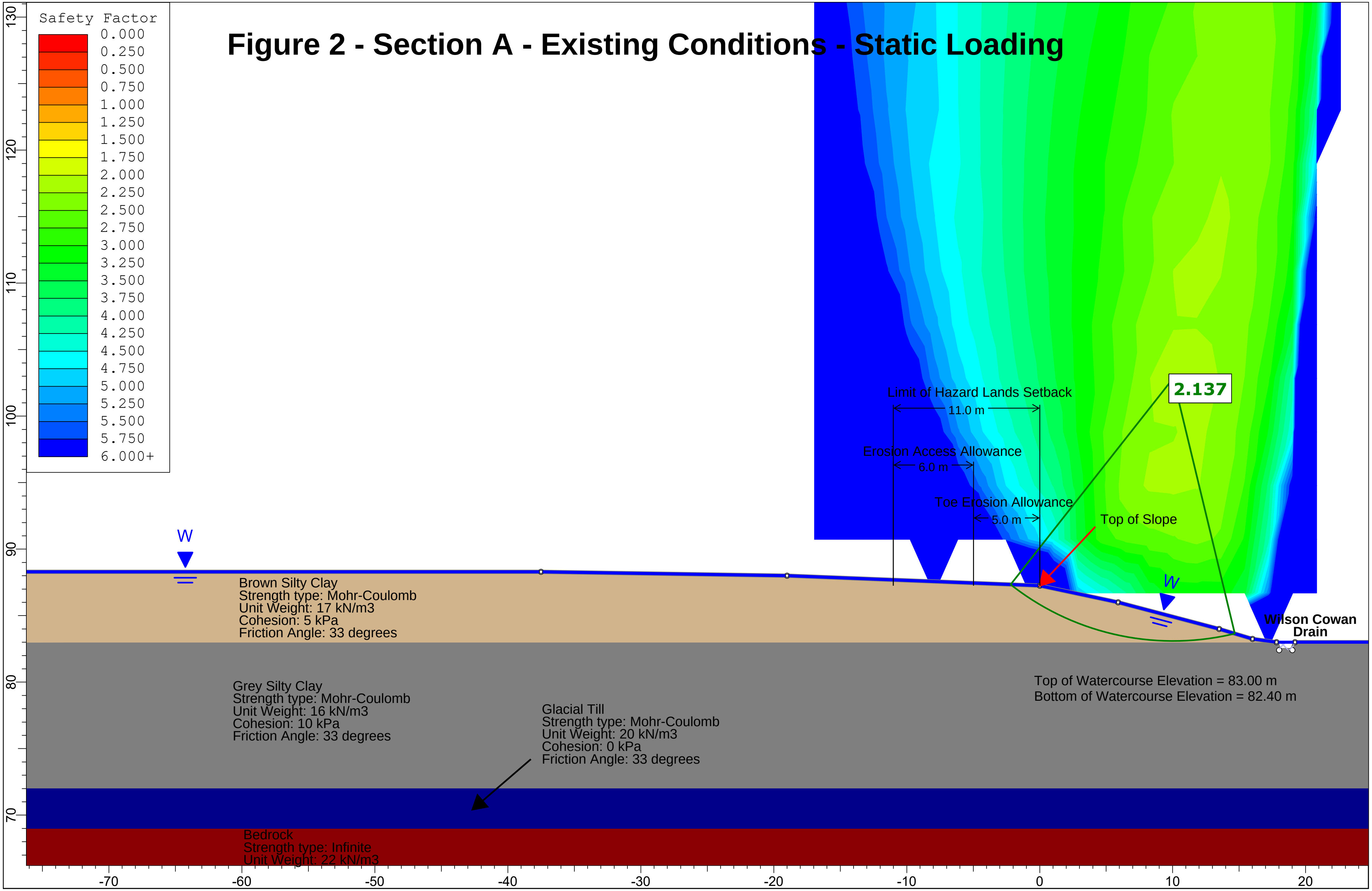


FIGURE 1

KEY PLAN

Figure 2 - Section A - Existing Conditions - Static Loading



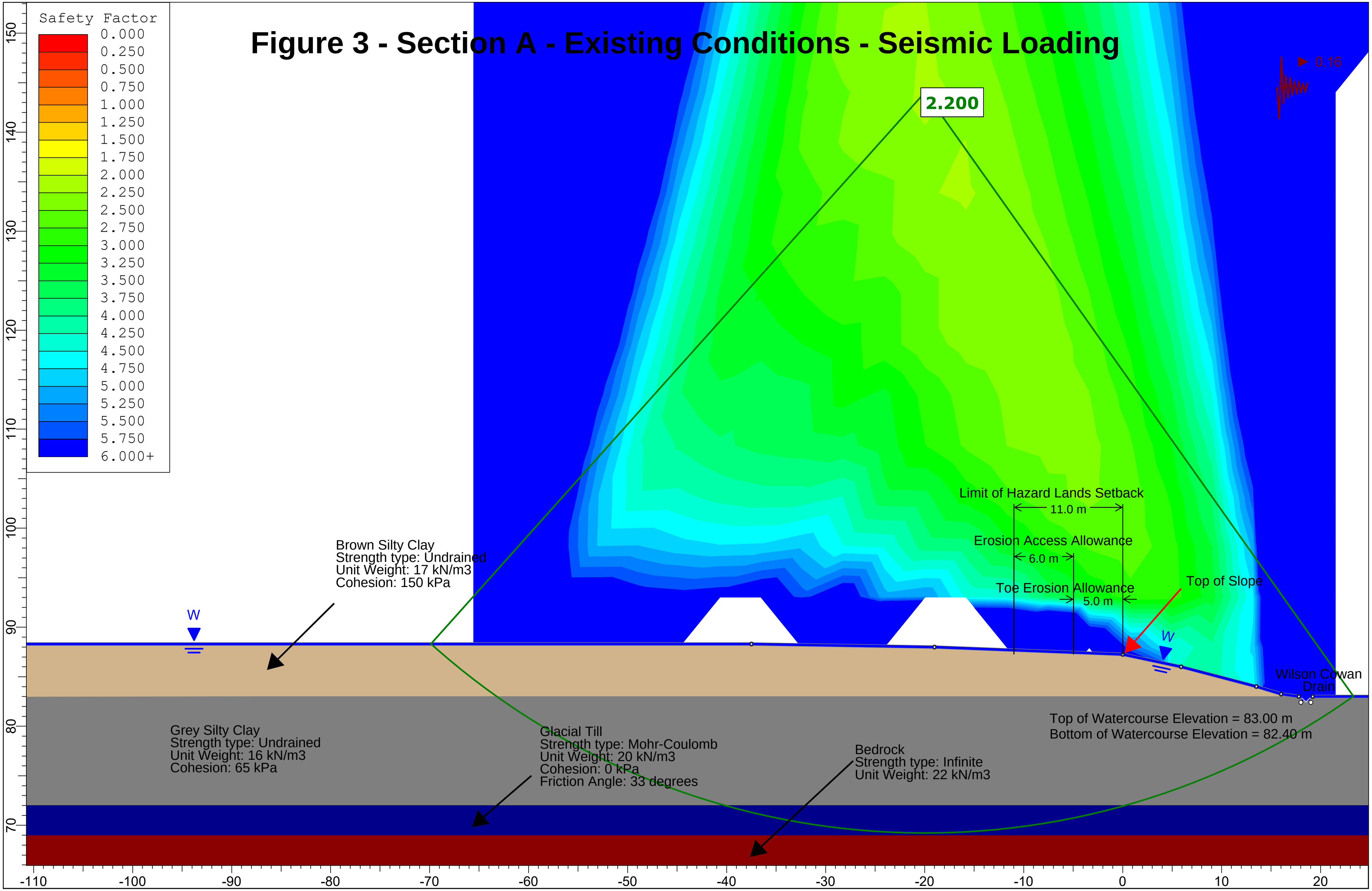


Figure 3 - Section A - Existing Conditions - Seismic Loading

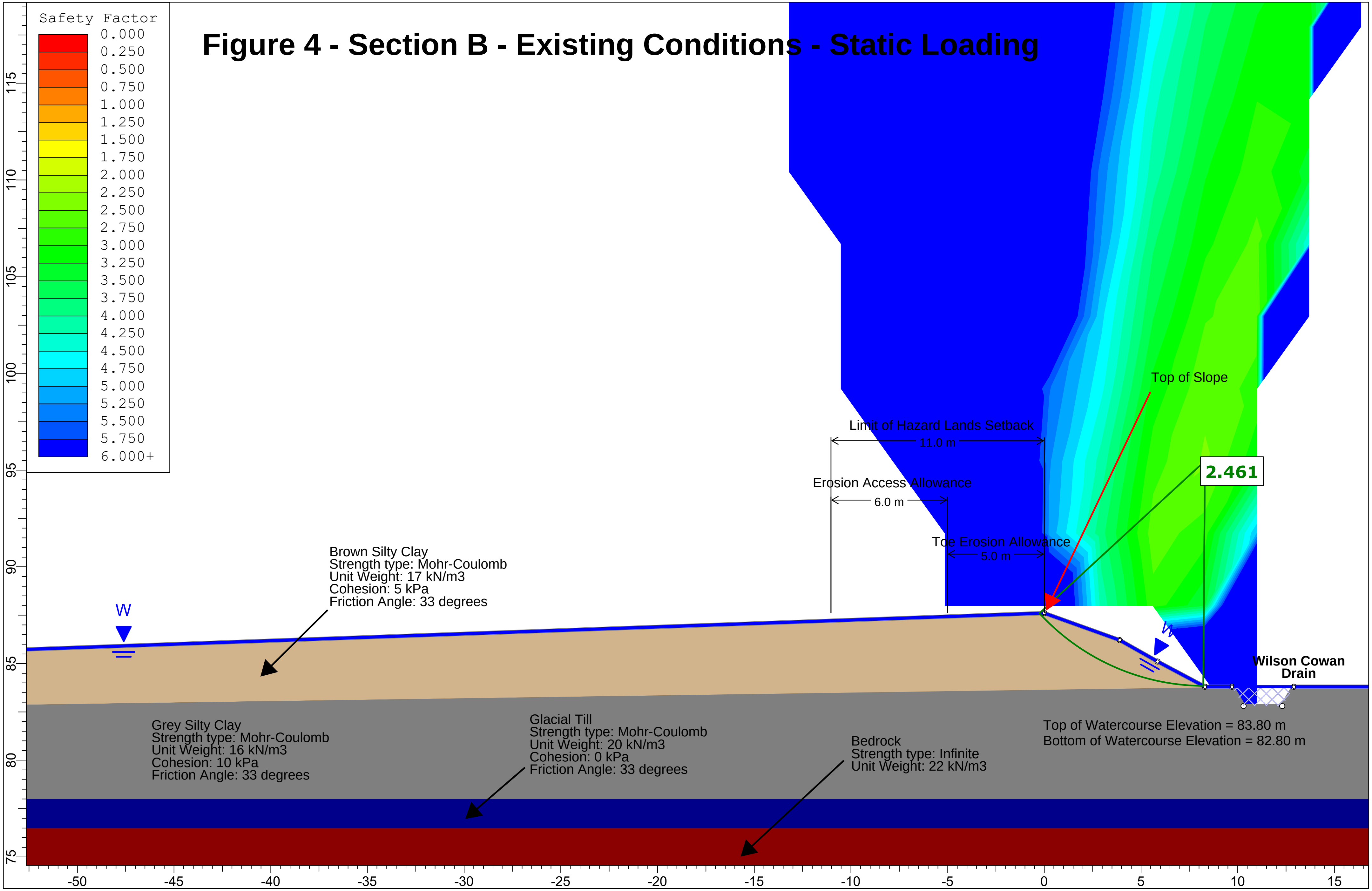
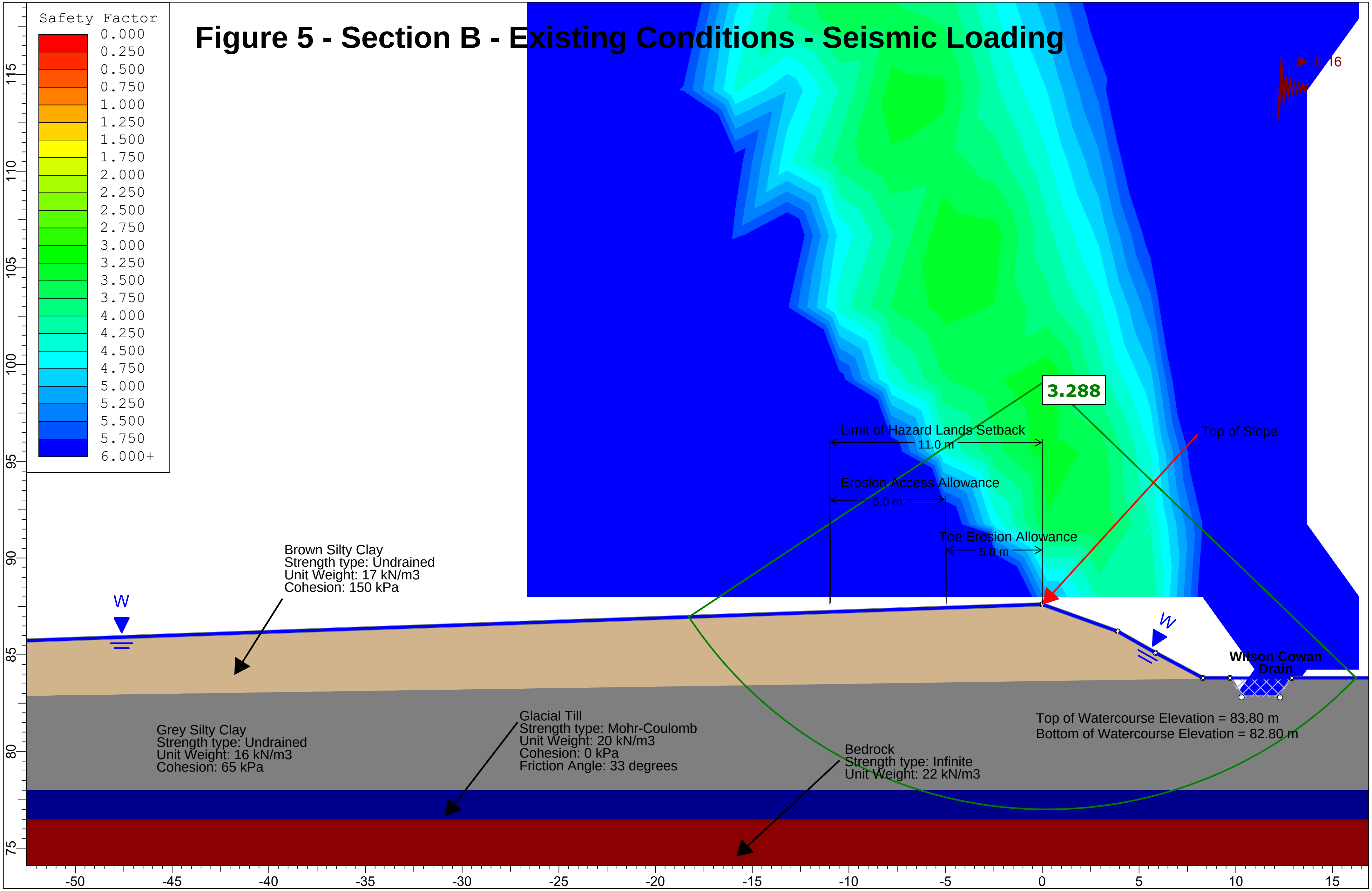


Figure 4 - Section B - Existing Conditions - Static Loading

Figure 5 - Section B - Existing Conditions - Seismic Loading



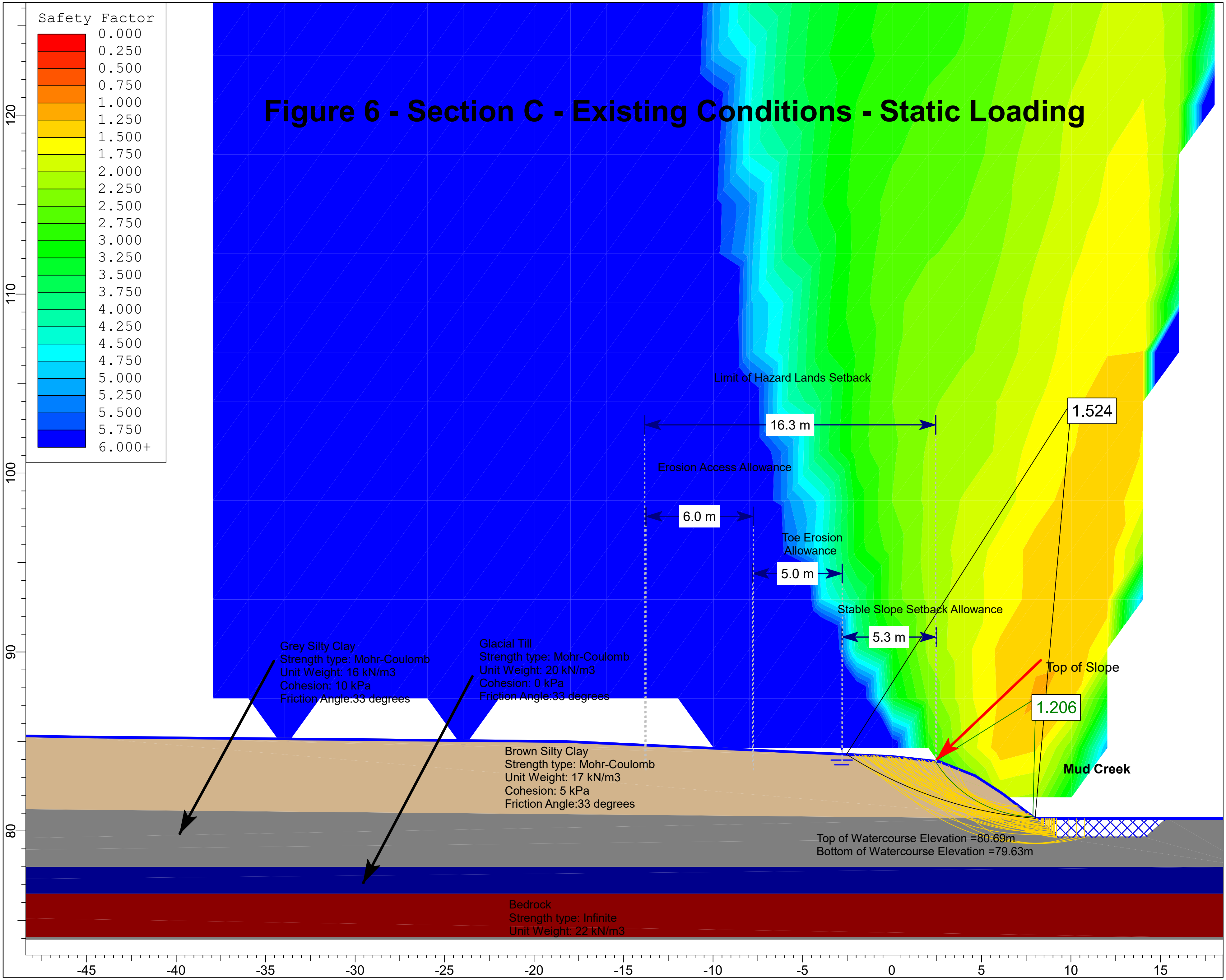


Figure 6 - Section C - Existing Conditions - Static Loading

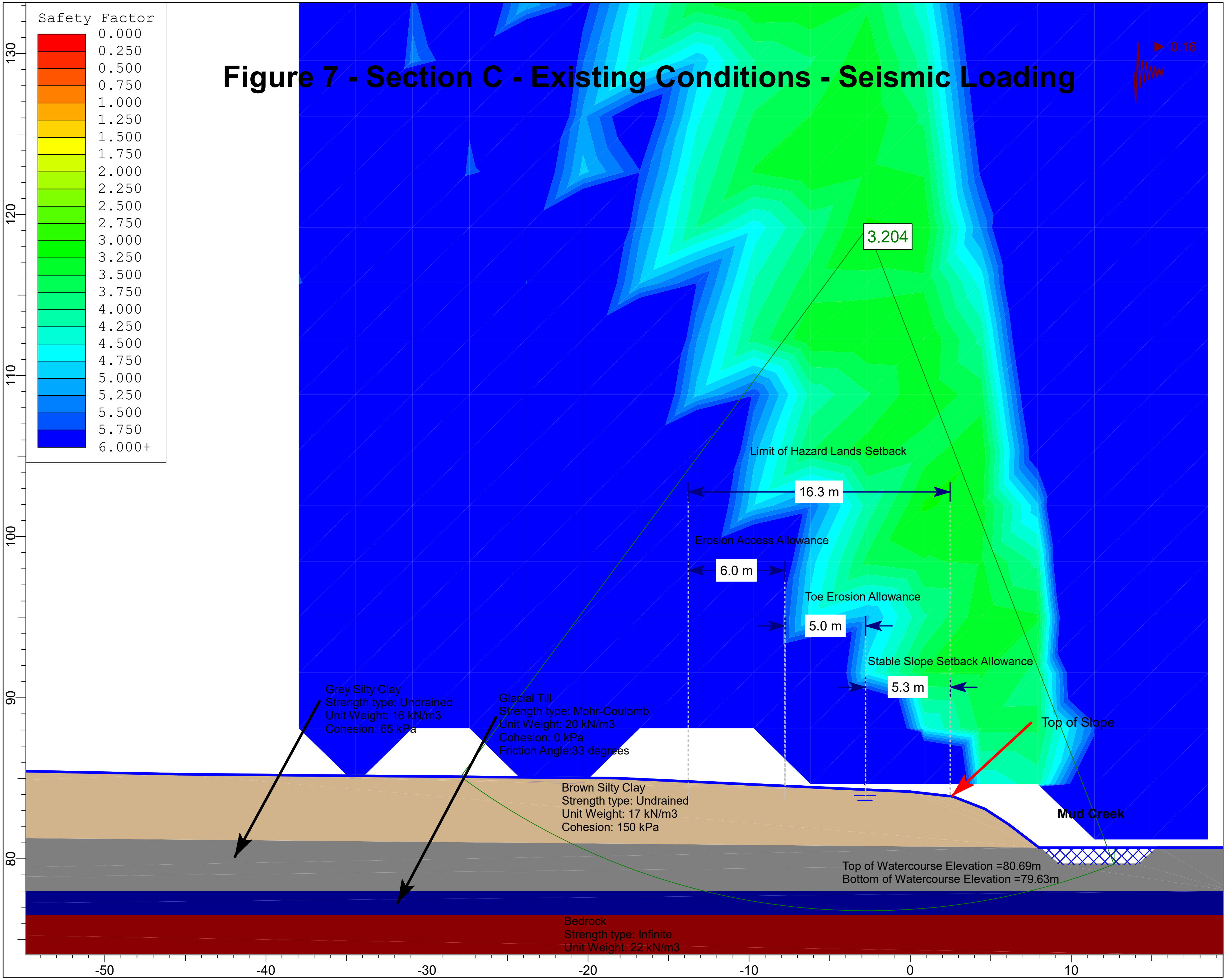
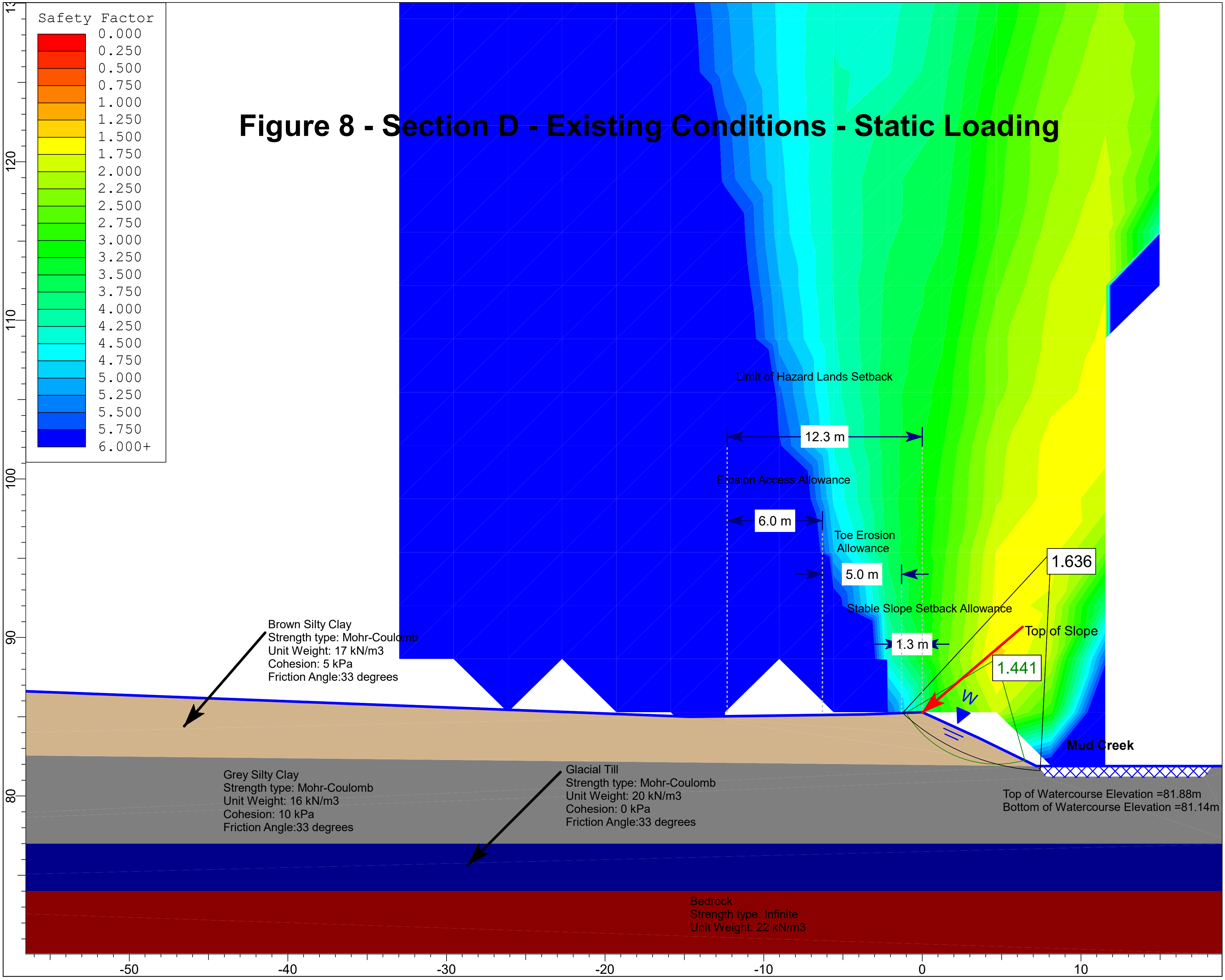


Figure 7 - Section C - Existing Conditions - Seismic Loading

Figure 8 - Section D - Existing Conditions - Static Loading



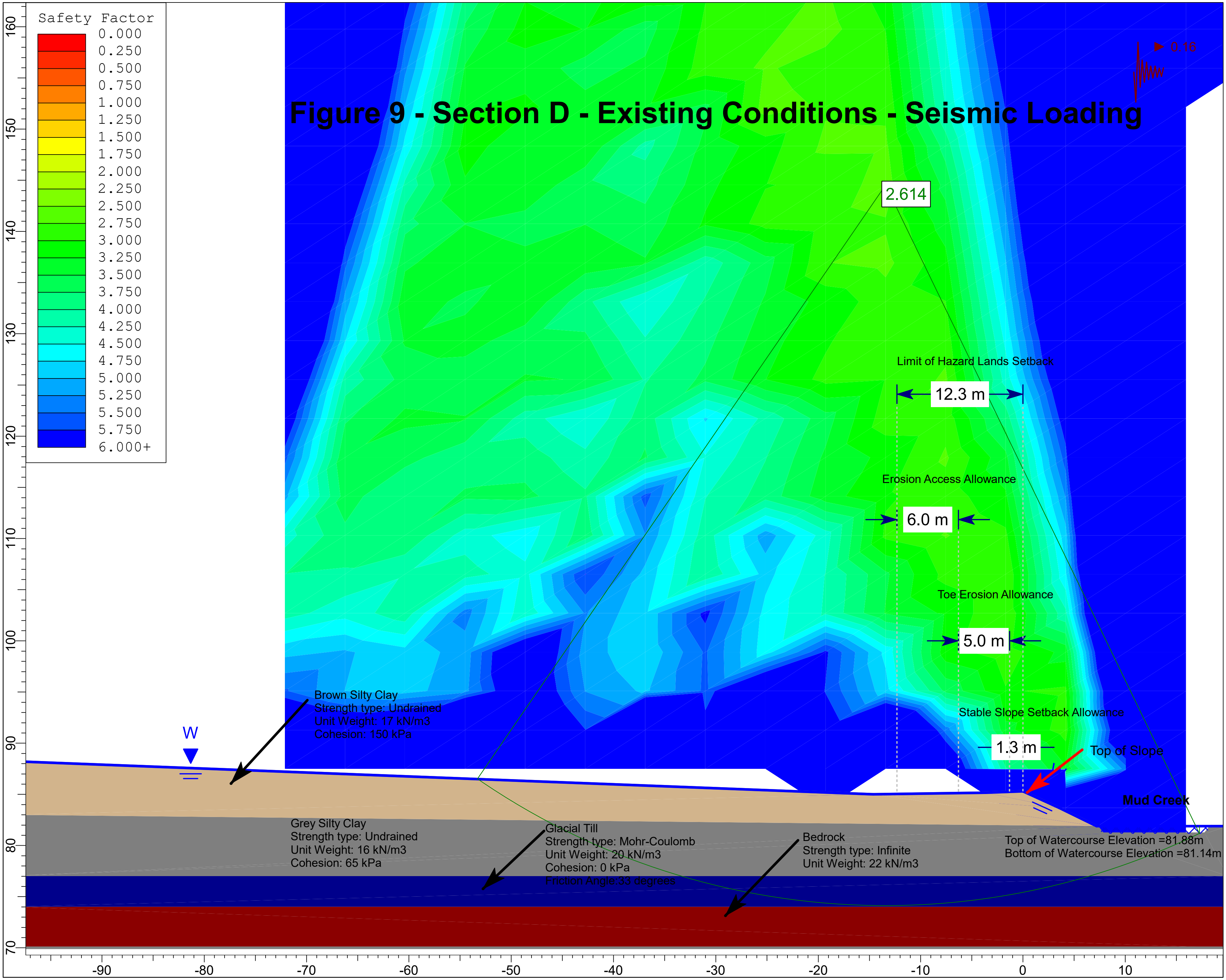


Figure 9 - Section D - Existing Conditions - Seismic Loading

Figure 10 - Section A - Proposed Conditions - Static Loading

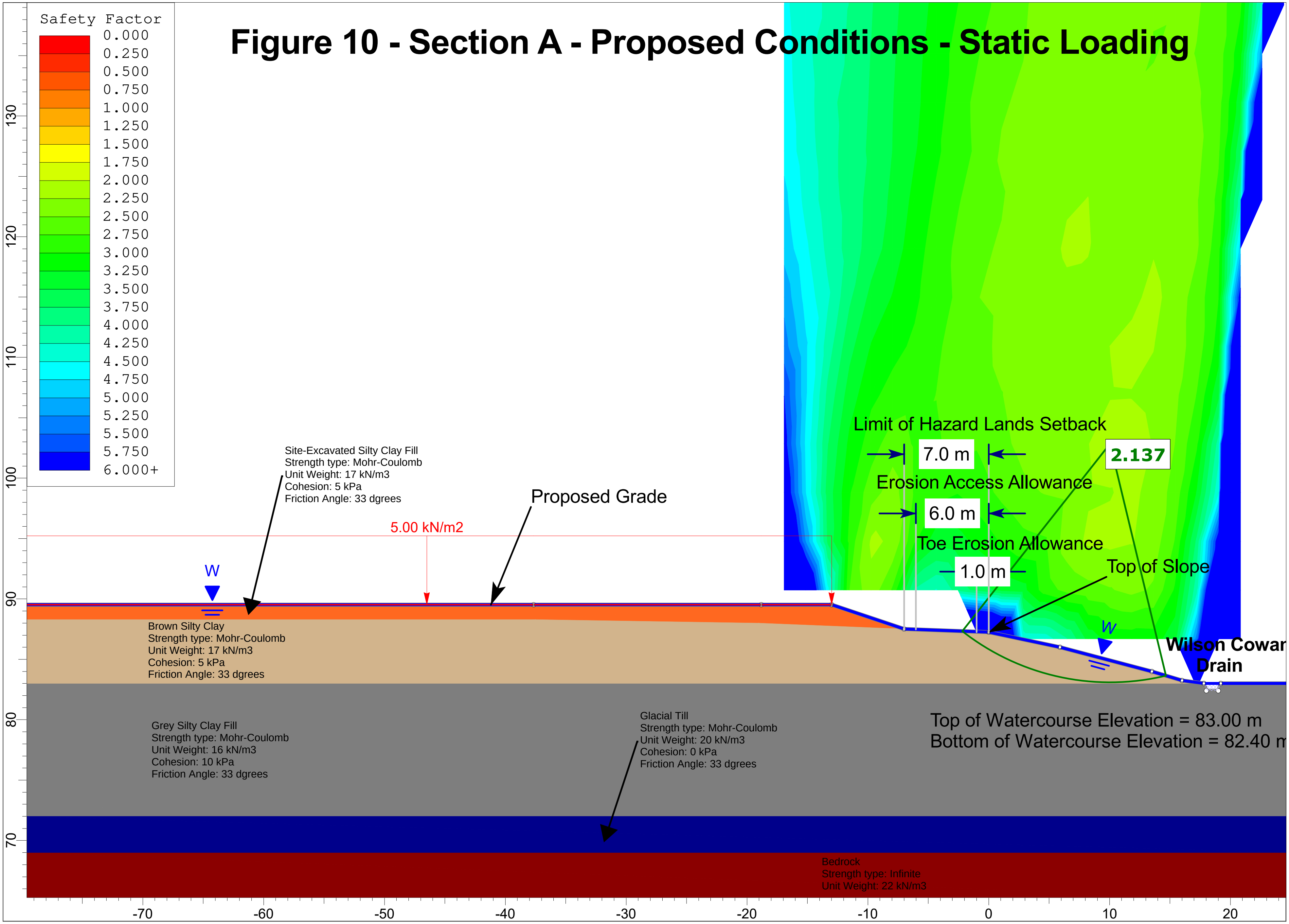
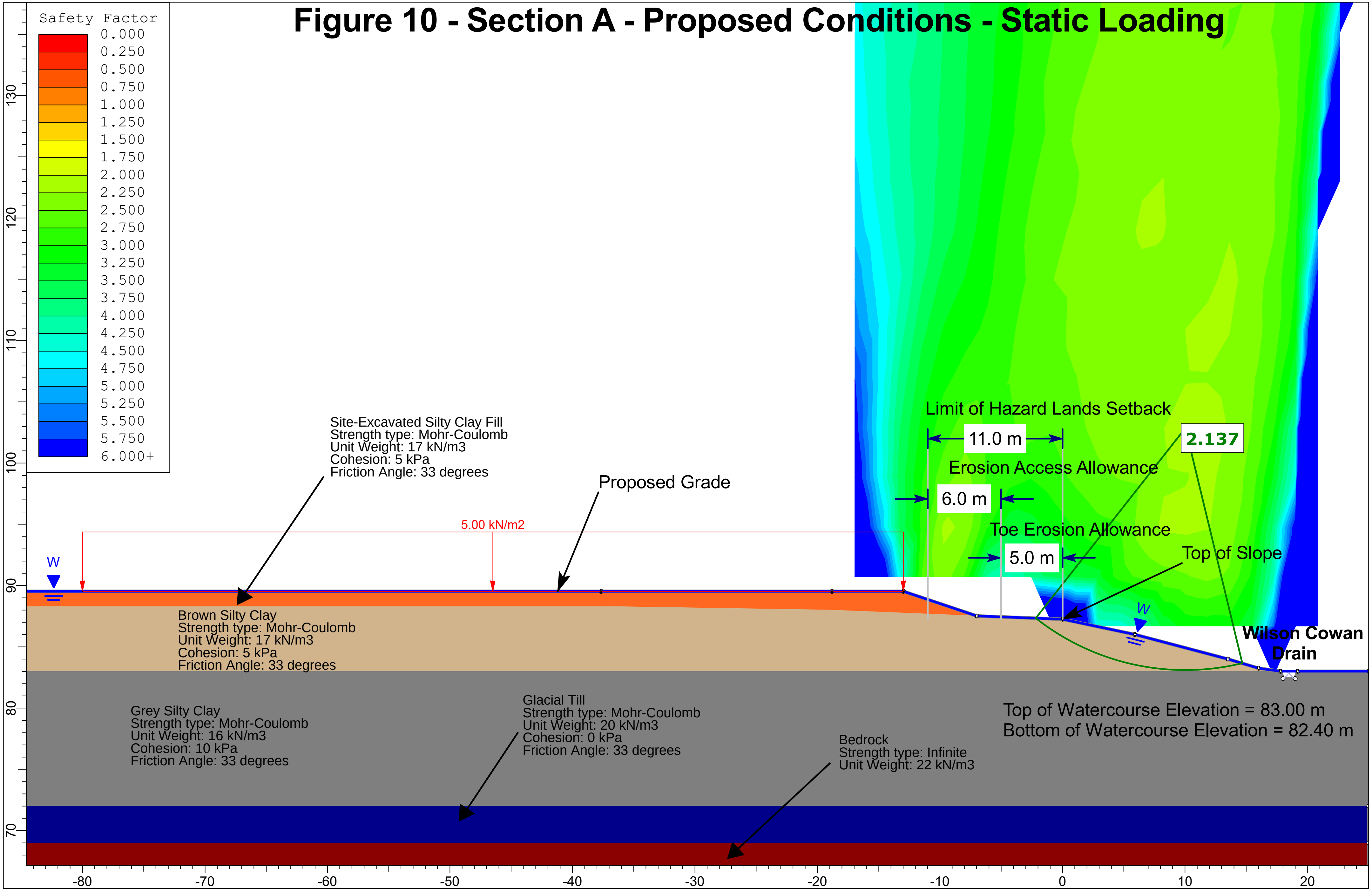
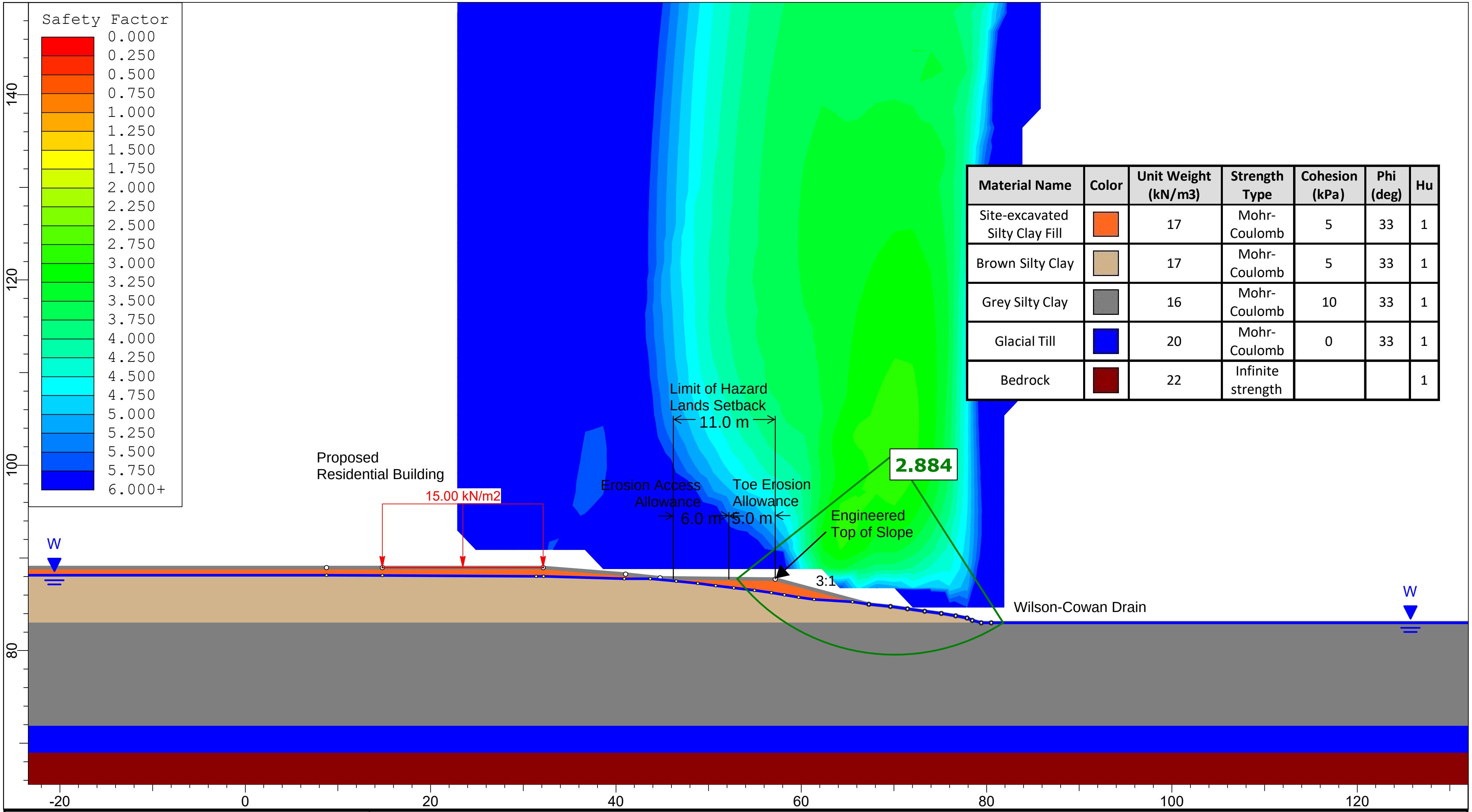


Figure 10 - Section A - Proposed Conditions - Static Loading



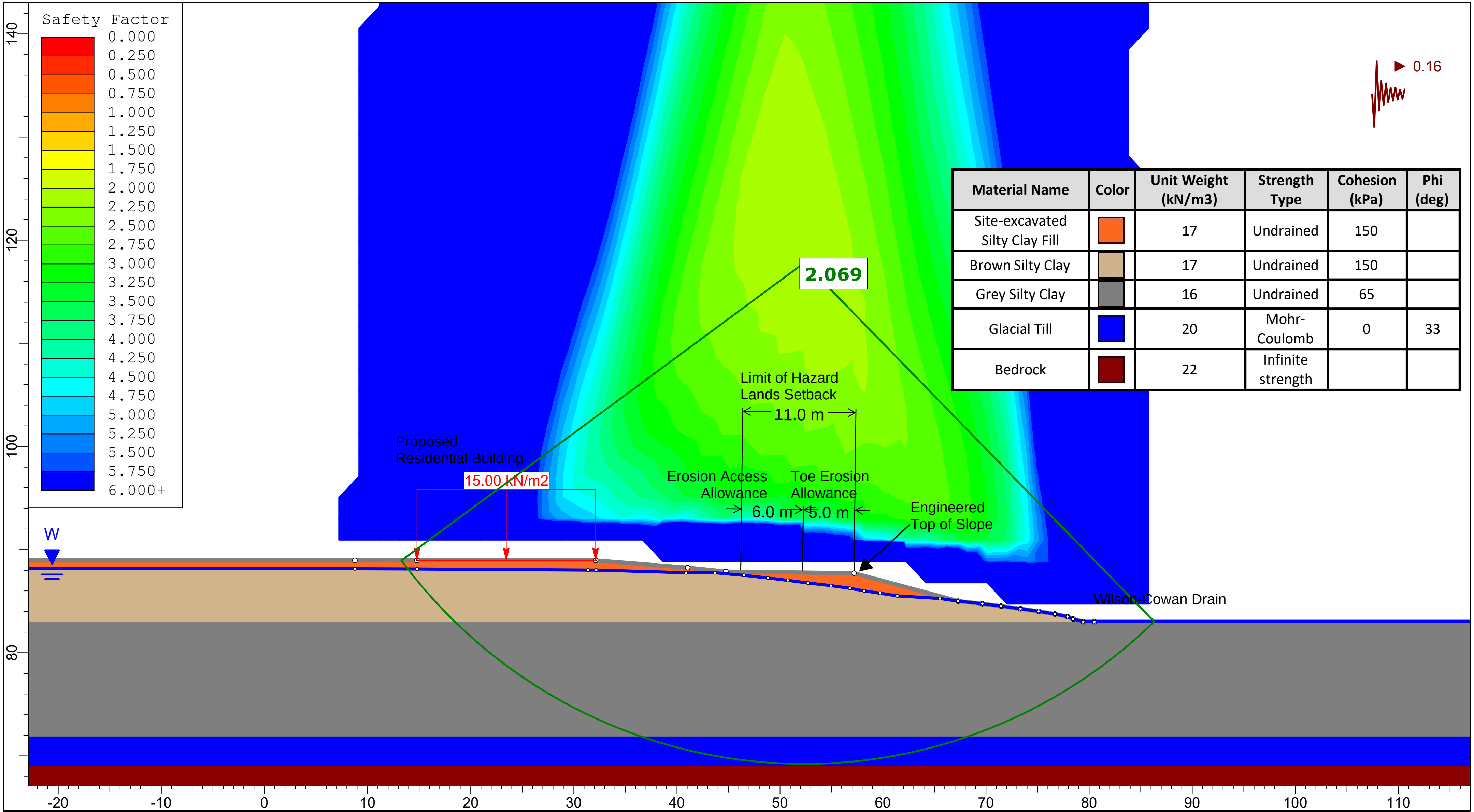


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Hu
Site-excavated Silty Clay Fill	🟧	17	Mohr-Coulomb	5	33	1
Brown Silty Clay	🟫	17	Mohr-Coulomb	5	33	1
Grey Silty Clay	■	16	Mohr-Coulomb	10	33	1
Glacial Till	🟦	20	Mohr-Coulomb	0	33	1
Bedrock	🟥	22	Infinite strength			1

PATERSON GROUP

SLIDEINTERPRET 9.025

Project	PG5828 - 4386 Rideau Valley Drive, Ottawa, ON		
Group	Figure 2A - Section A-A - Static Analysis		
Drawn By	MA	*Company*	Uniform Developments
Date	2024-07-12	*File Name*	Slope Stability Assessment



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Site-excavated Silty Clay Fill	🟧	17	Undrained	150	
Brown Silty Clay	🟫	17	Undrained	150	
Grey Silty Clay	⬛	16	Undrained	65	
Glacial Till	🟦	20	Mohr-Coulomb	0	33
Bedrock	🟥	22	Infinite strength		

PATERSON GROUP

SLIDEINTERPRET 9.025

Project
PG5828 - 4386 Rideau Valley Drive, Ottawa, ON

Group
Figure 2B - Section A-A - Seismic Analysis

Drawn By
MA

Company
Uniform Developments

Date
2024-07-12

File Name
Slope Stability Assessment

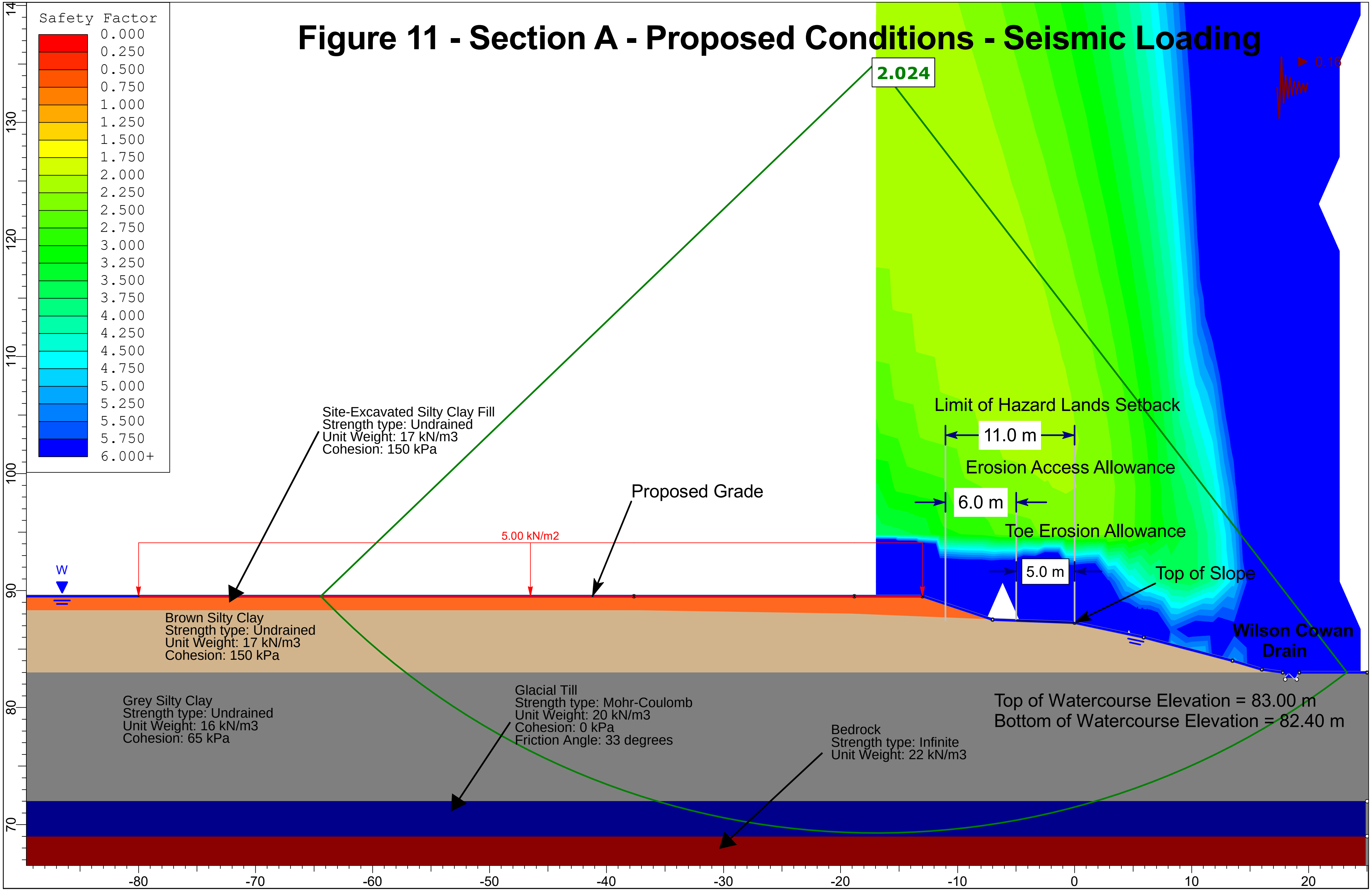


Figure 11 - Section A - Proposed Conditions - Seismic Loading

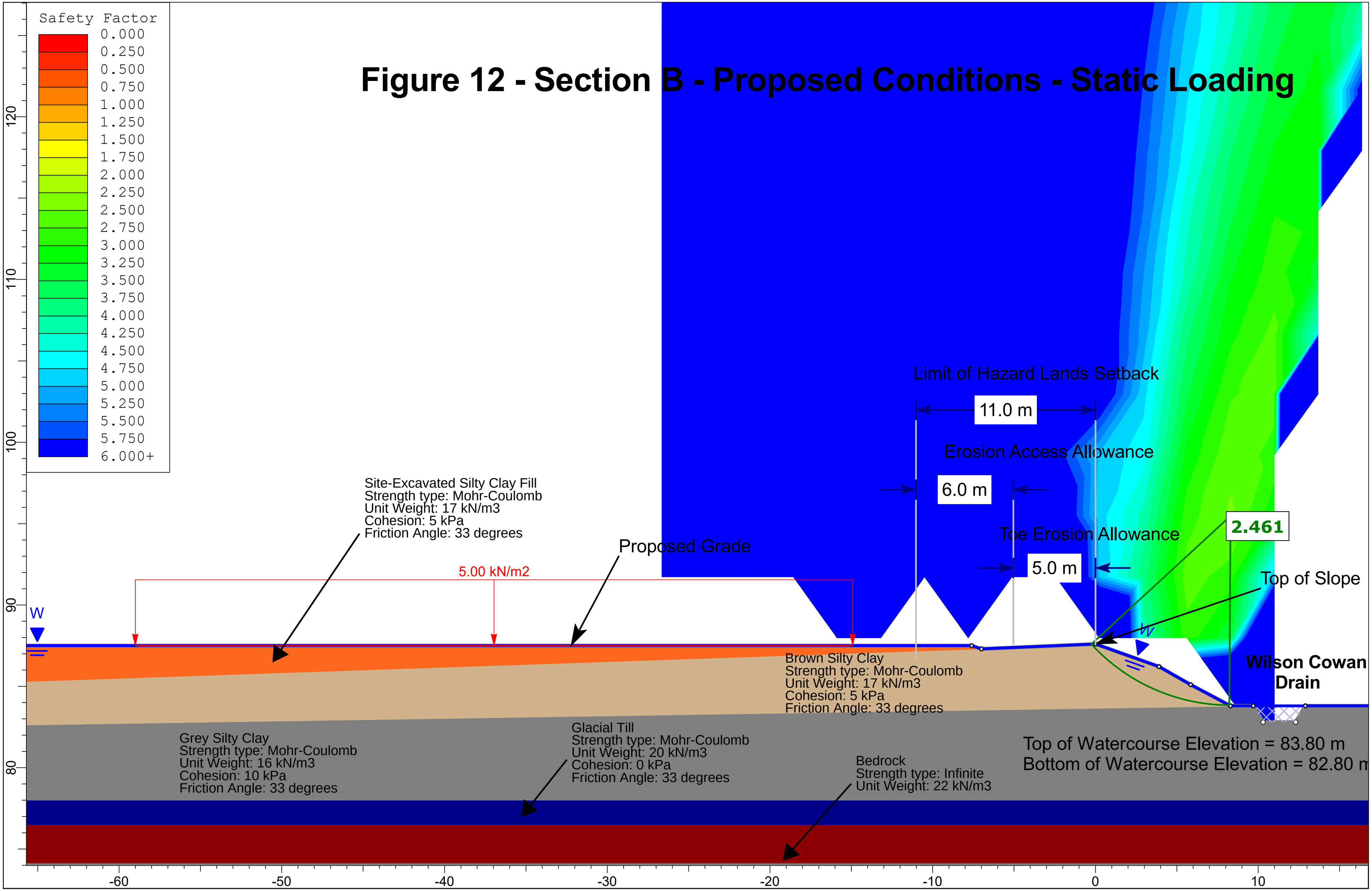


Figure 12 - Section B - Proposed Conditions - Static Loading

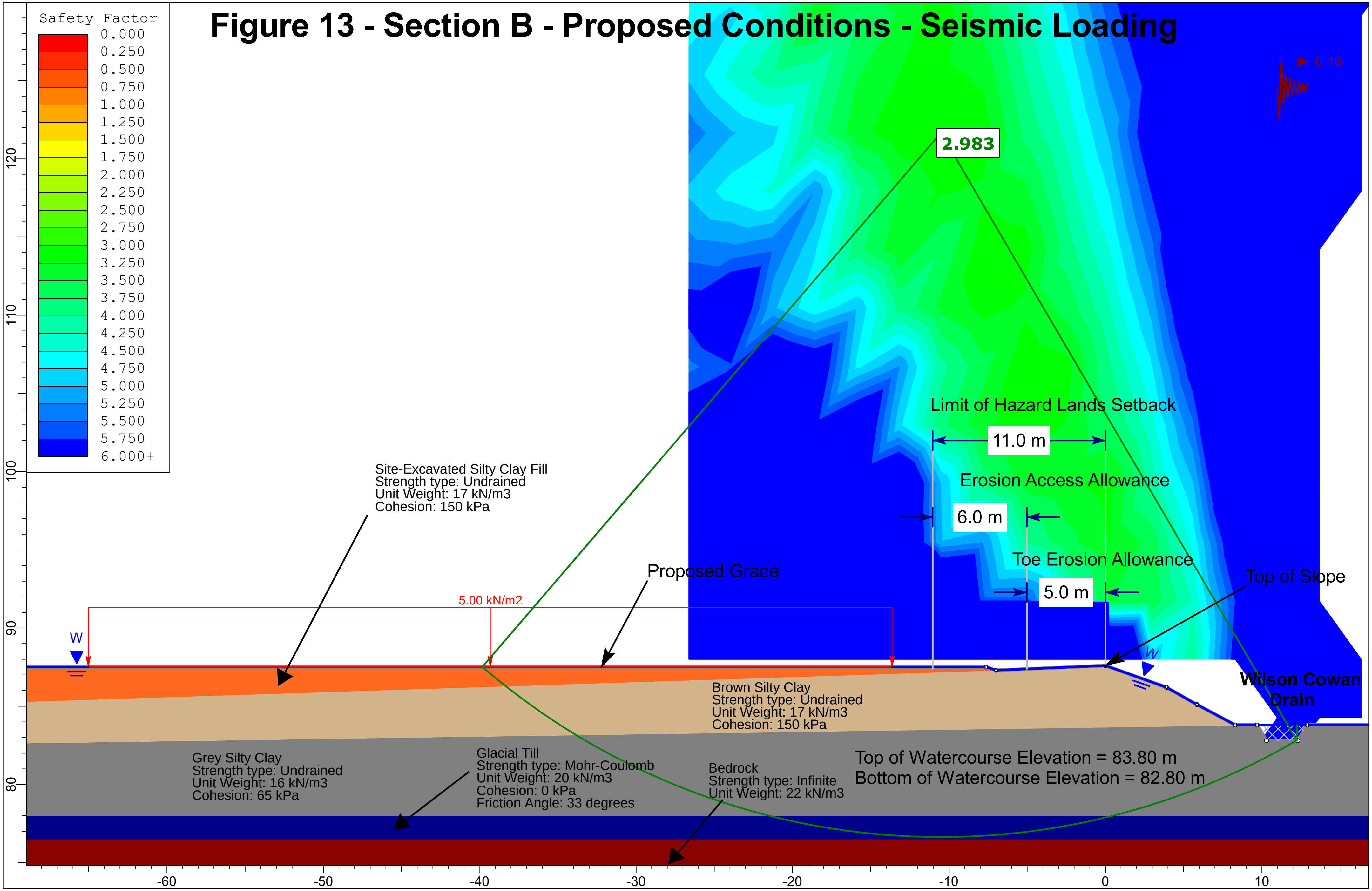


Figure 13 - Section B - Proposed Conditions - Seismic Loading

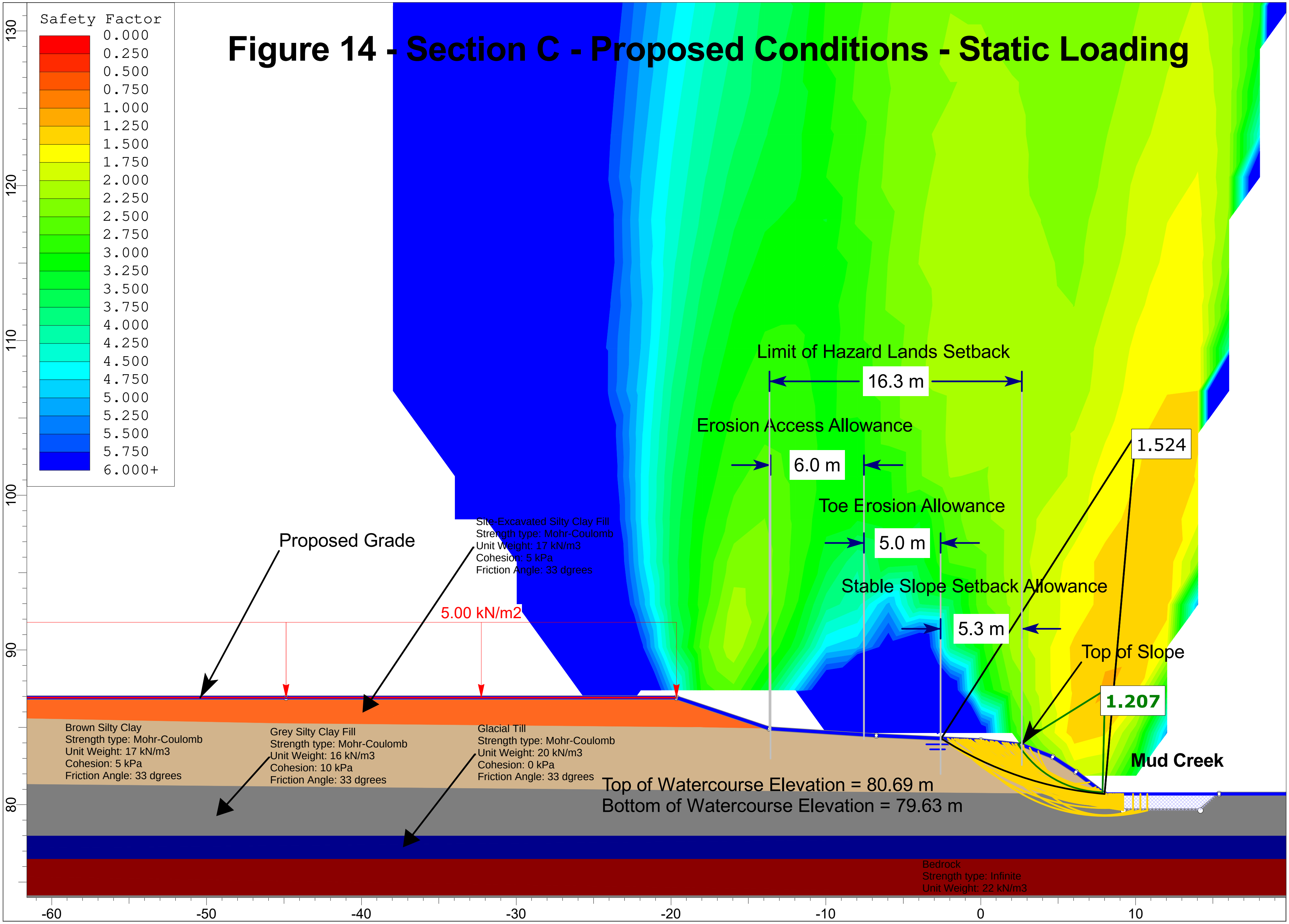
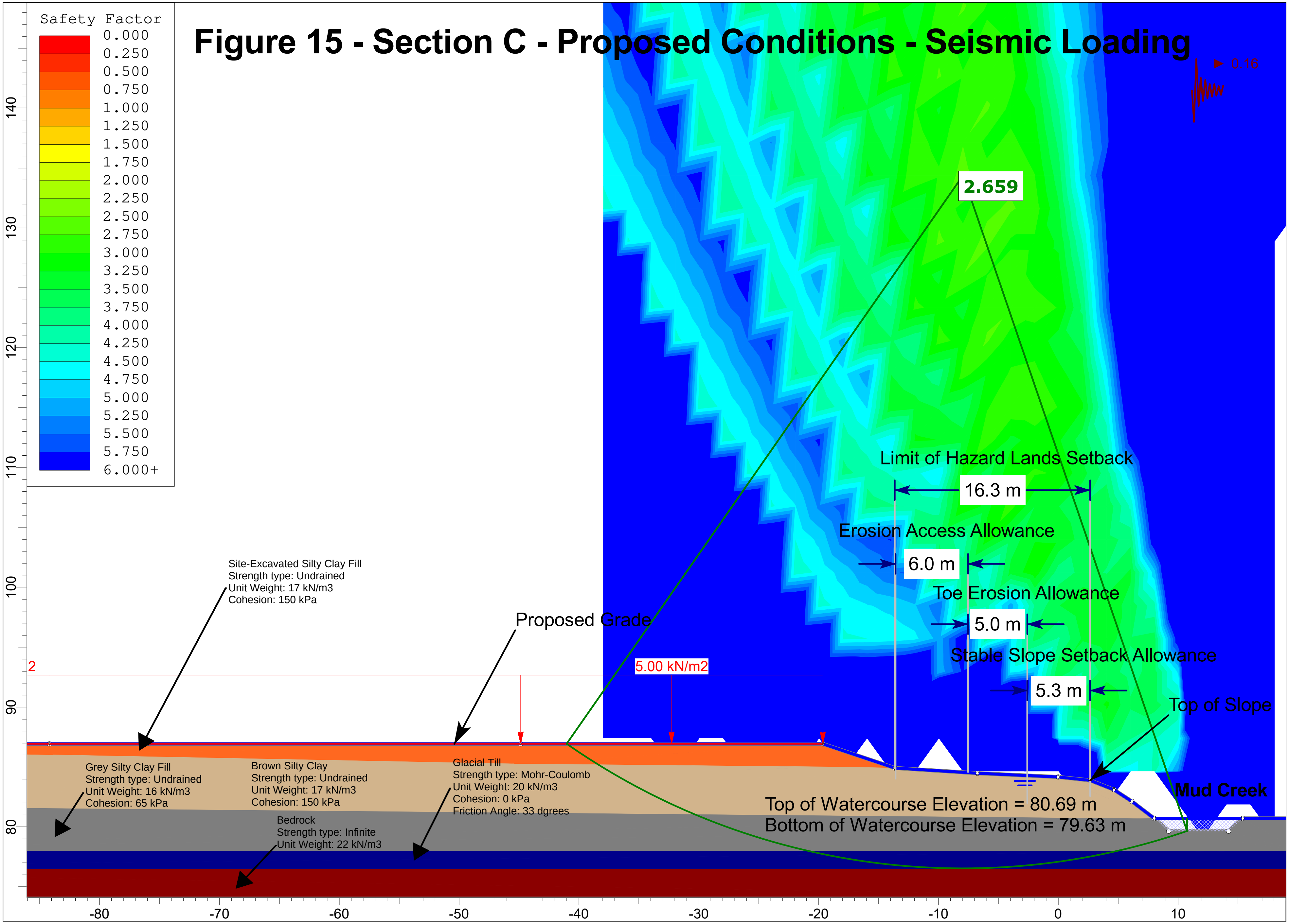


Figure 14 - Section C - Proposed Conditions - Static Loading

Figure 15 - Section C - Proposed Conditions - Seismic Loading



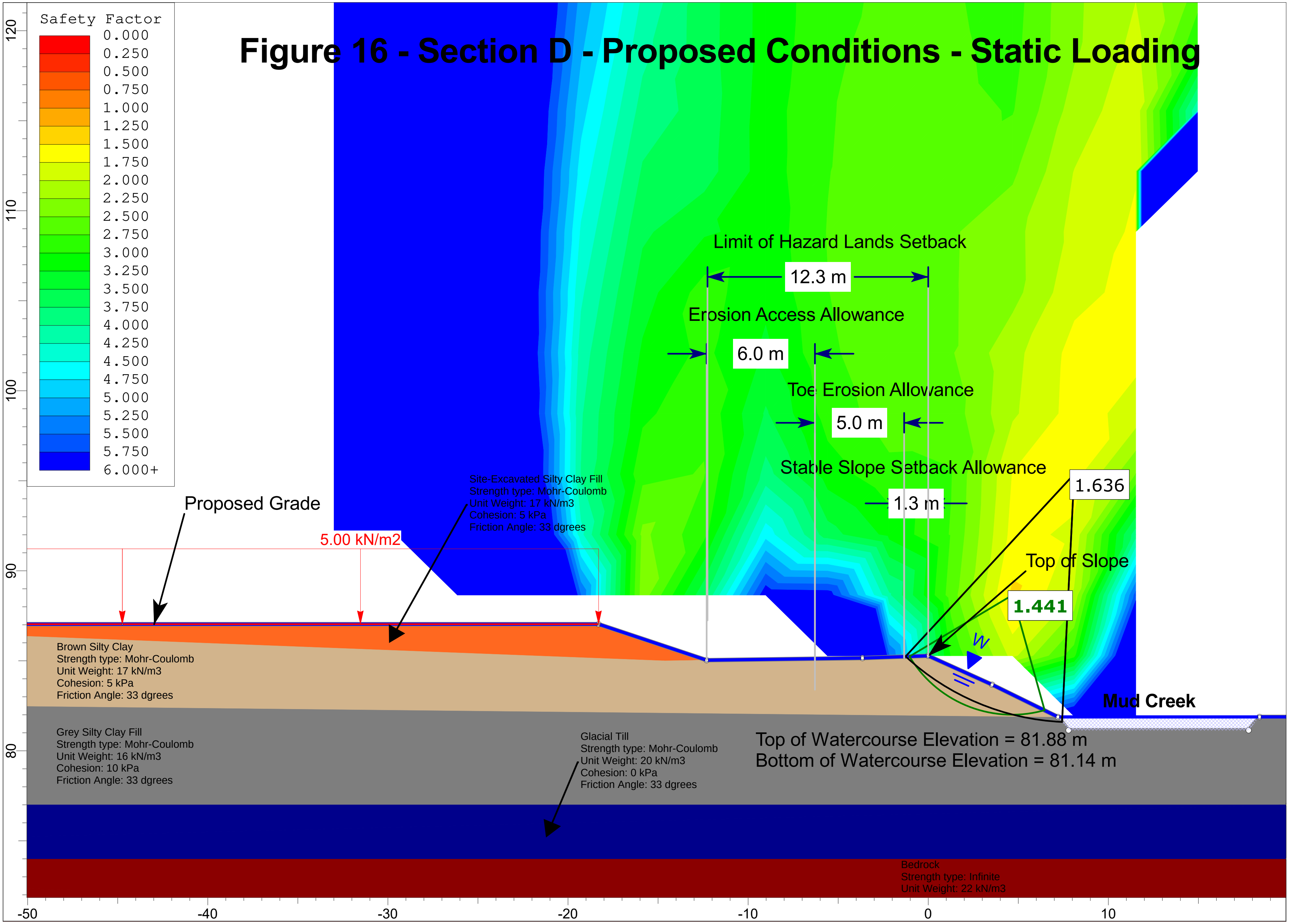


Figure 16 - Section D - Proposed Conditions - Static Loading

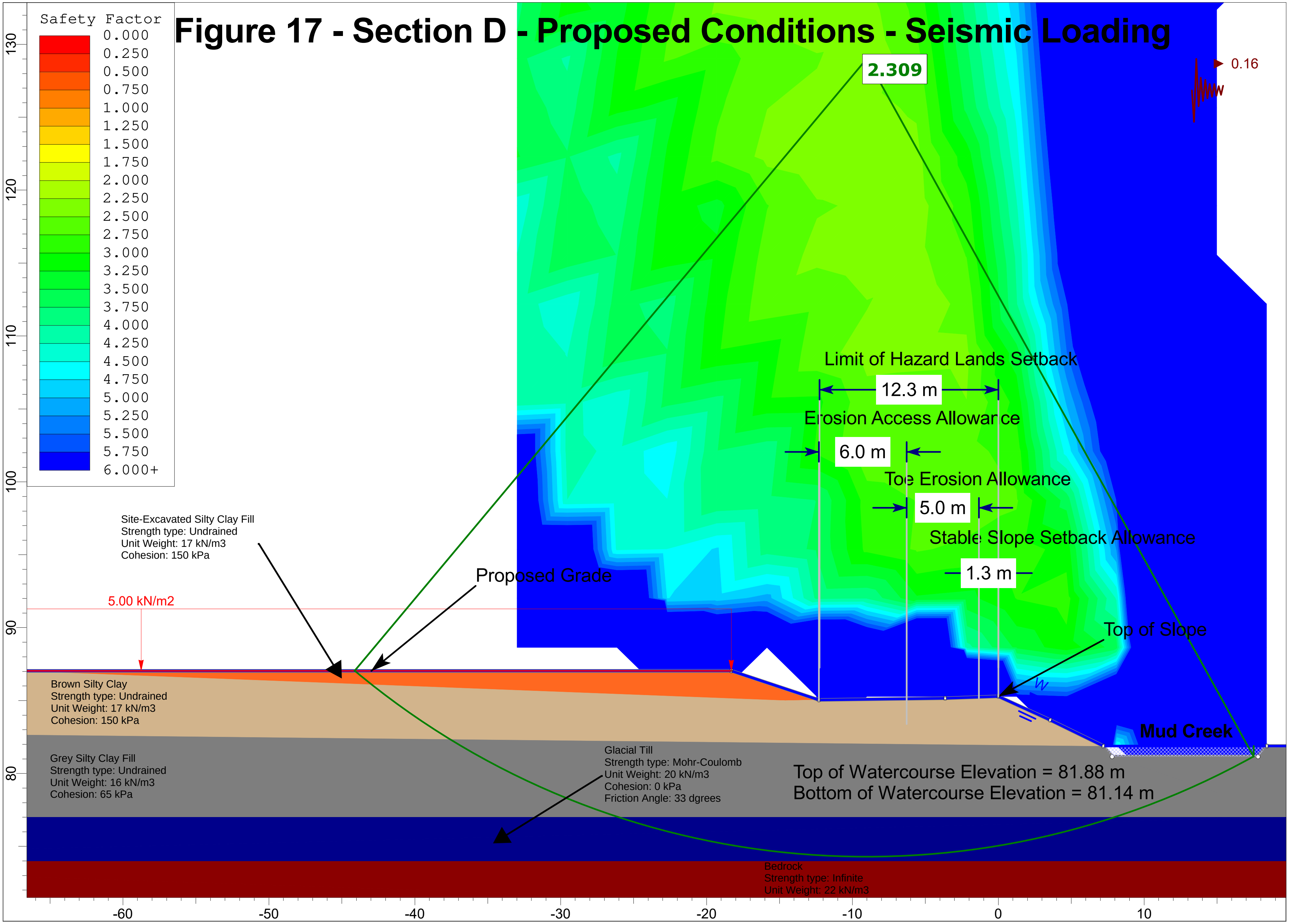


Figure 17 - Section D - Proposed Conditions - Seismic Loading

Photo 1: Area located at the bottom of the slope along the south-west portion of the subject site. Area is well vegetated and sloped gradually towards the valley floor.

Photo 2: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Water throughout Wilson Cowan Drain appeared to be flowing very slowly and/or ponding.

Photo 3: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 4: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Water throughout Wilson Cowan Drain appeared to be flowing very slowly and/or ponding.

Photo 5: Area along Wilson Cowan Drain and south-west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Water throughout Wilson Cowan Drain appeared to be flowing very slowly and/or ponding.

Photo 6: Area along Wilson Cowan Drain and west portion of the subject site. Area appeared to be well vegetated and did not appear to be eroded at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 7: Area along Wilson Cowan Drain and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along Wilson Cowan Drain at the time of site visit. Gradual slope observed from subject site to the valley floor. Active erosion was not observed.

Photo 8: Area along Wilson Cowan Drain and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along Wilson Cowan Drain at the time of site visit. Gradual slope observed from subject site to the valley floor. Active erosion was not observed.

Photo 9: Area along Wilson Cowan Drain and north-west portion of the subject site. Area appeared to be well vegetated with a gentle flow throughout Wilson Cowan Drain at the time of site visit. Gradual slope observed from subject site to the valley floor.

Photo 10: Area of intersection of Wilson Cowan Drain along west portion of subject site and Mud Creek. Area of Mud Creek appeared to have banks exposed to streams flow. Mature trees noted to have previously fallen across creek alignment. Some over-steepening of banks also observed at the time of site visit.

Photo 11: Area of Mud Creek along north-west portion of subject site. Area appeared to have banks exposed to streams flow and lack of well rooted vegetation along bank. Some over-steepening of banks also observed. Creek appeared to be flowing very slowly at the time of site visit.

Photo 12: Area of Mud Creek along north-west portion of subject site. Area appeared to have banks exposed to streams flow and along with slumping and oversteepening of banks at the time of our site visit.

Photo 13: Area of Mud Creek along north-west portion of subject site. Area of valley floor appeared to have well rooted vegetation with relatively steep banks along creek. No active erosion observed along photographed portion of creek at the time of site visit.

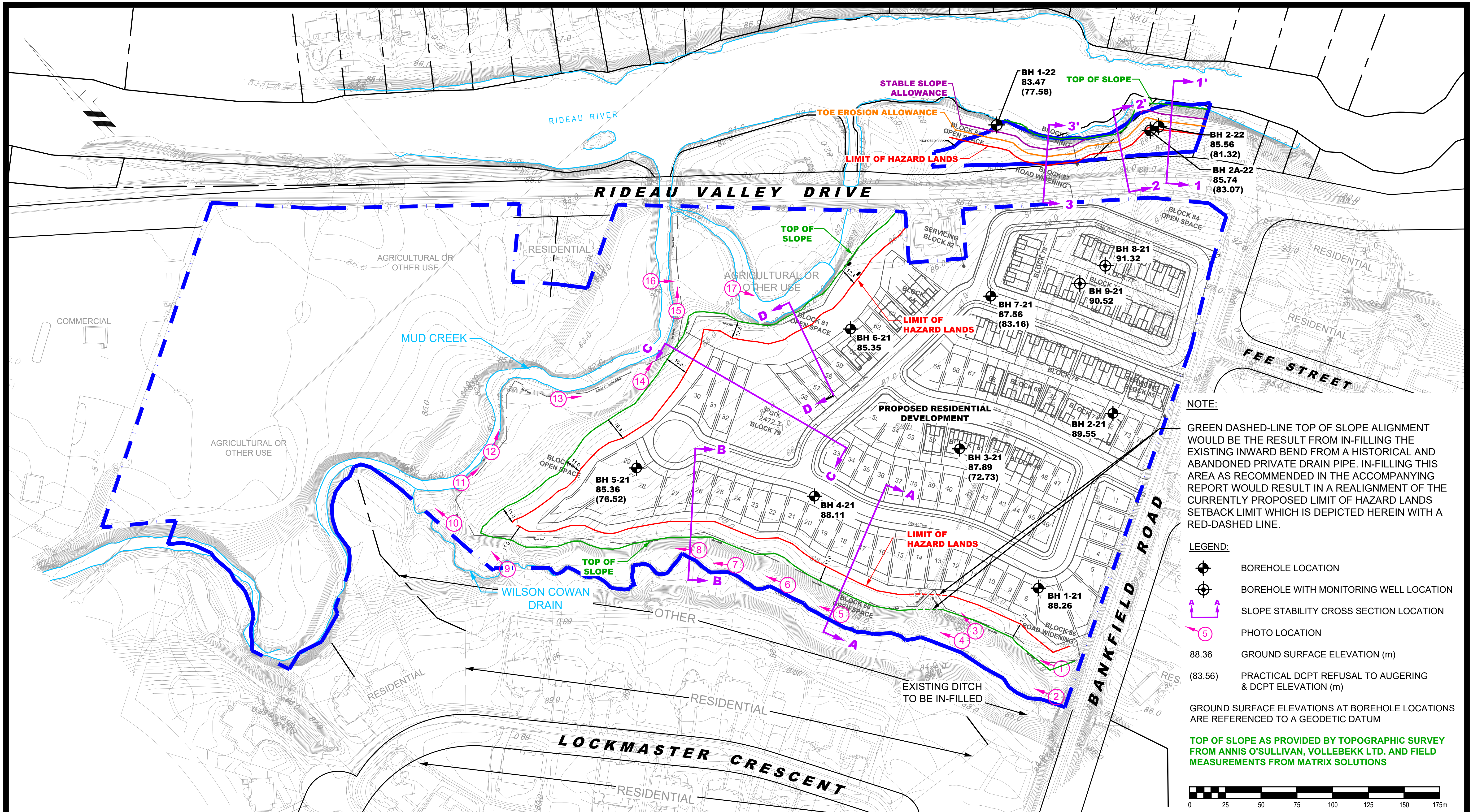
Photo 14: Area of Mud Creek along north-west portion of subject site. Area of valley floor appeared to have well rooted vegetation with relatively steep banks along creek. Some active erosion and fallen trees observed along photographed portion of creek.

Photo 15: Area of Mud Creek along northern portion of subject site. Photographed area appeared to have banks exposed to streams flow along with slumping and undercutting of banks at the time of our site visit.

Photo 16: Area of Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees.

Photo 17: Area of Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees. No erosion observed along toe of slope at time of site visit.

Photo 17b: Close-up of Photo 17 - Area of Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees. No erosion observed along toe of slope at time of site visit.



NOTE:

GREEN DASHED-LINE TOP OF SLOPE ALIGNMENT WOULD BE THE RESULT FROM IN-FILLING THE EXISTING INWARD BEND FROM A HISTORICAL AND ABANDONED PRIVATE DRAIN PIPE. IN-FILLING THIS AREA AS RECOMMENDED IN THE ACCOMPANYING REPORT WOULD RESULT IN A REALIGNMENT OF THE CURRENTLY PROPOSED LIMIT OF HAZARD LANDS SETBACK LIMIT WHICH IS DEPICTED HEREIN WITH A RED-DASHED LINE.

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

0 25 50 75 100 125 150 175m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
4	SITE PLAN UPDATED	04/04/2024	SK
3	WATER COURSES NAME UPDATED	20/10/2023	MS
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

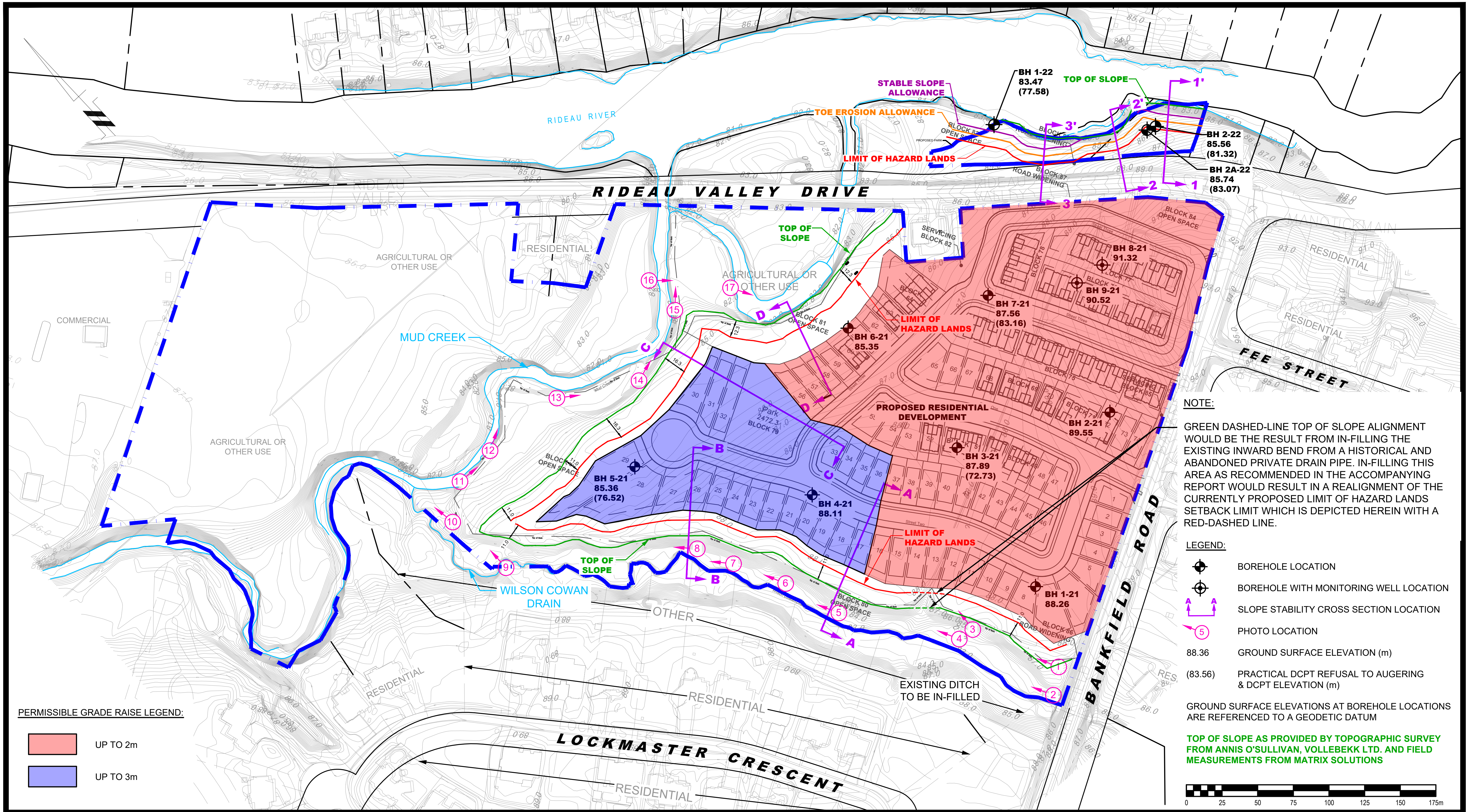
UNIFORM DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500
Date: 06/2021
Drawn by: JM
Report No.: PG5828-1
Checked by: KP
Dwg No.: PG5828-1
Approved by: DJG
Revision No.: 4

p:\autocad drawings\geotechnical\pg58xx\pg5828\pg5828-1 thlp (rev.04) (july 2024).dwg



NOTE:

GREEN DASHED-LINE TOP OF SLOPE ALIGNMENT WOULD BE THE RESULT FROM IN-FILLING THE EXISTING INWARD BEND FROM A HISTORICAL AND ABANDONED PRIVATE DRAIN PIPE. IN-FILLING THIS AREA AS RECOMMENDED IN THE ACCOMPANYING REPORT WOULD RESULT IN A REALIGNMENT OF THE CURRENTLY PROPOSED LIMIT OF HAZARD LANDS SETBACK LIMIT WHICH IS DEPICTED HEREIN WITH A RED-DASHED LINE.

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

0 25 50 75 100 125 150 175m

PERMISSIBLE GRADE RAISE LEGEND:

🟥	UP TO 2m
🟦	UP TO 3m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7J9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:2500
Date: 06/2021
Drawn by: NFRV
Report No.: PG5828-1
Checked by: KP
Dwg No.: PG5828-3
Approved by: DJG
Revision No.:

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APPENDIX 3

RELEVANT REPORTS

PATERSON GROUP

memorandum

re: **Geotechnical Response to City Comments**
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

to: Uniform Urban Developments Ltd. – **Mr. Ryan MacDougall –** rmacgougall@uniformdevelopments.com

date: October 17, 2023

file: PG5828-MEMO.01

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the City of Ottawa listed in the letter dated May 1, 2023 (File Nos. D02-02-220118, D07-16-22-0026) regarding the proposed residential development at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5828-1 Revision 3 dated October 17, 2023.

Geotechnical Investigation Comments

Comment 2.11

Please refer to the watercourses as Mud Creek and the Wilson Cowan Drain, rather than Mud Ruisseau Creek and tributary, to remain consistent with other reports and plans submitted.

Response:

Noted. Reference to the watercourses has been modified in our revised geotechnical report mentioned above, as requested.

Comment 2.12

Please expressly state whether any of the clay soils on site may be 'sensitive marine clays', or not. [page 8 of 65].

Response:

As noted under subsection 6.9-Landscaping Considerations in our original geotechnical report, and based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples.

In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay.

Having said that, it should be noted that page 8 has been revised to indicate the presence of low to medium sensitivity marine silty clay deposit in the subject site under subsection 5.1 in the above-mentioned revised geotechnical report, as requested.

Comment 2.15

Do the results of your study of the Slope Stability study align with the results from the Geo-fluvial Study? [page 18 of 65].

Response:

Paterson reviewed the geo-fluvial study completed by Matrix Solutions, dated November 2022, for the proposed residential development. Based on our review of the above-noted study, it appears that the results of our slope stability study are in general agreement with the results of the geofluvial study for the majority of the proposed limit of hazard lands with the exception of the recommended toe erosion allowance along Wilson Cowan Drain. Paterson is recommending 1m for toe erosion along that drain based on the nature and size of the drain (i.e. not a permanent watercourse) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season, as opposed to 5m for toe erosion as suggested by the geofluvial study. Furthermore, the geofluvial study did not provide photographs depicting active erosion along the Wilson Cowan drain. Further justification for the toe erosion allowance has been included in our geotechnical report under subsection 6.8. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan and the erosion limit proposed by Matrix solutions as well as the limit of hazard lands proposed by Paterson are both outside the limits of the proposed development.

Comment 2.16

Please provide further detail regarding the area proposed to be filled in the rear of Lots 5 & 6.

Response:

Backfilling of the slope face in the vicinity of the rear yards of lots 5 and 6 can be completed in a stepped fashion to provide a finish grade with a slope face of minimum 3H:1V. Site preparation and backfilling should be completed under dry weather conditions (specifically for the clay placement portion of the program) and above freezing temperatures, and in accordance with our geotechnical recommendations provided under section 5.2 of the revised geotechnical report noted above.

Comment 2.17

Please explain what the shrinkage limit and other Atterberg limits results infer.

Response:

Due to the presence of a silty clay deposit at the subject site, Paterson completed a review of the soils on the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Based on our review of the results of the shrinkage limit and Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples indicating that the silty clay across the subject site is considered ***low to medium sensitivity marine clay***, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Reference should be made to subsection 6.9- Landscaping Considerations in our above-mentioned revised geotechnical report.

Comment 2.18

Please state why the June 16, 2022, results were not included.

Response:

The geotechnical investigation conducted on June 16, 2022 pertained to the proposed park, located across Rideau Valley Drive which was done after submitting the geotechnical report for the residential development. Having said that, the results of the geotechnical investigation conducted for the proposed park have been added to the above-mentioned revised geotechnical report. Furthermore, the geotechnical letter mentioned above has been added as an addendum to Appendix 3 of the above-mentioned geotechnical report.

Comment 2.19

Consolidation results not found in the report.

Response:

No consolidation tests were completed on the encountered silty clay deposit at the subject site. Consolidation testing is not possible within the silty clay deposit, where encountered within the subject site, due to the stiffness of the overall deposit. Consolidation testing in the Ottawa area is typically carried out on soft to firm silty clay samples which are recovered from Shelby tubes taken during the field investigation. To accurately complete consolidation testing, the soft to firm (undrained shear strength of 12 to 50 kPa) silty clay samples are required to be undisturbed. The consistency of the silty clay encountered at the subject site was determined to be generally hard to stiff (undrained shear strength ranging between 50 to >200 kPa), based on in-situ vane testing completed as part of our geotechnical

investigation. Due to the consistency, advancement of Shelby tubes and subsequent recovery of an undisturbed silty clay sample is not possible.
Damage to either the piston sampler or the thin-walled Shelby tube is expected based on our experience with silty clay of similar consistency. Therefore, in our professional opinion, the available information collected from the boreholes drilled at the subject site is sufficient for us to provide a permissible grade raise for the proposed subdivision, without the need for a consolidation test. Reference should be made to subsection 5.3-Foundation Design, in our revised geotechnical report.

Comment 2.20

Sensitivity results are required.

Response:

The sensitivity index of the encountered silty clay deposit was calculated based on the ratio between the undisturbed and remolded shear vane test measured in the field, for all the boreholes, and it was found to be generally below 4, indicating a normal sensitivity clay. Please refer to subsection 4.2 in the revised above-mentioned geotechnical report fur further discussion regarding the sensitivity index calculation for the encountered silty clay deposit.

Comment 2.21

Atterberg limits results are required from a number of elevations in each borehole.

Response:

Atterberg limits tests were conducted at the encountered silt clay deposit in each borehole at the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade, and are considered to be sufficient from a geotechnical perspective to provide valuable information and satisfy the requirements for the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) in assessing the sensitivity of the silty clay deposit for tree planting.

Comment 2.22

A longer-term, or year-long groundwater level analysis is required.

Response:

Based on our understanding, LID measures are not considered for the subject site. Therefore, year-long groundwater level is not required from a geotechnical perspective at the subject site.

Comment 2.23

Groundwater cannot be stated to be expected to lower based on the LID directive documents without analysis showing that it will be so (with similitude, if necessary/appropriate).

Response:

Reference should be made to our response to comment 2.22 above. Furthermore, it is unclear what the reviewer is referring to LID directives. Further clarification is required. In any case, post-development groundwater level lowering is conservatively anticipated following construction of site servicing at residential developments, as observed by Paterson from previous similar jobs.

Comment 2.24

For section 5.1, please note that lightweight fill is not permitted in ROWs.

Response:

Noted. Lightweight fill is not permitted in ROWs. Please refer to subsections 5.1 and 5.3 in the revised above-mentioned geotechnical report.

Comment 2.25

It is suggested that the plastic, sensitive soils be restricted in section 5.2 under the heading Fill Placement.

Response:

Our recommendation for fill placement under subsection 5.2 clearly state that fill placed beneath the building areas should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. It is further stated in our report under section 5.2 that placement of a non-specified existing fill along with site-excavated soil (including the plastic sensitive soils) is permitted only under landscape areas where settlement of the ground surface is of minor concern.

Comment 2.26

Section 5.3, under the heading Bearing Resistance Values (Conventional Shallow Foundation), should be reviewed against the grading plan and the boreholes.

Response:

Noted. A statement was added to the report to indicate that the bearing capacity will be reviewed against the grading plans for the proposed residential subdivision, once available. Reference should be made subsection 5.3 in the above-mentioned revised geotechnical report.

Comment 2.27

The comments that the subject site are not susceptible to liquefaction requires an exhaustive discussion: whichever approach the consultant takes will require proof of similitude and full copies of papers provided to the City showing unequivocal support.

Response:

The soils encountered at the subject site consist of silty clays, which are cohesive in nature. These soils were evaluated for liquefaction susceptibility in accordance with the criteria prepared by Bray at al. 2004 which determines that all soils with a plasticity index exceeding 20% are not liquifiable (Figure 1). In general, the plasticity index results completed on samples taken from the silty clay layer were found to be above 20. Therefore, the encountered soils are not susceptible to liquefaction. Reference should be made to subsection 5.4- Design for Earthquakes in the abovementioned revised geotechnical report, for further details on liquefaction susceptibility at the subject site.

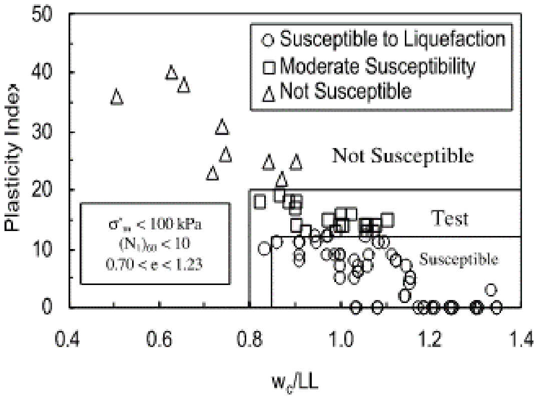

Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

Comment 2.28

The comments under the heading of Foundation Drainage, within section 6.1, Foundation Drainage and Backfill, appear to be from another report; please review the report and confirm that all other comments are for the address intended.

Response:

Recommendations for foundation drainage for the proposed residential development are provided under section 6.1-Foundation Drainage and Backfill, of the above revised geotechnical report. These recommendations are applicable to the proposed residential development at the subject site.

Comment 2.29

For the end of section 6.3 please state if deep excavations will be occurring.

Response:

Based on the available conceptual plans, it is understood that the proposed subdivision will consist of single and townhouse style residential houses. Therefore, deep excavation for buildings is generally not anticipated at the subject site. Furthermore, the detailed design servicing plans were not provided at the time of writing the report. However, recommendations for deep excavations for construction of services, if deemed needed, are included in subsection 6.3- Excavation Side Slopes in the revised geotechnical report for the subdivision, referenced above.

Comment 2.30

Please state why the horizontal acceleration of 0.16g was included under the heading of Seismic Loading Analysis (as opposed to another value).

Response:

Per the City of Ottawa Slope Stability Guidelines for Development Applications, the seismic coefficient to be used in the analyses is typically half the peak ground acceleration (PGA) specified in the National Building Code of Canada (NBCC). The PGA at the location of the subject site, based on the 2015 NBCC is approximately 0.266. Therefore the seismic coefficient at the location of the subject site is 0.133. However, based on previous versions of the NBCC, the PGA for the Ottawa area is 0.32, thus using a seismic coefficient of 0.16 is generally a more conservative approach, and is considered acceptable from a geotechnical perspective.

Comment 2.31

A toe erosion allowance of 1 m is not acceptable. The comments on "active erosion was not observed" are contested in a number of the photographs in Appendix 2. The toe erosion allowance, under the heading of Toe Erosion and Access Allowances shall be revised as per Table 3 of the Ministry of Natural Resources, and Forestry (MNRF) Technical Guide- River and Stream Systems: Erosion Hazard Limit due to the active erosion and the soils of the boreholes. It is noted that the Fluvial Geomorphic and Erosion Hazard Assessment completed by Matrix Solutions Inc. recommended a 5 m toe erosion allowance for the Wilson Cowan Drain. Based on the penetration resistance blows of the Soil Profile and Test Data Sheets the soils on site may be Soft/Firm Cohesive Soils, loose granular, (sand, silt) fill, in the MNRF Guide.

Response:

Based on our field review and engineering analysis, active erosion was not encountered along the western watercourse at Wilson Cowan drain. It is to be clarified that the photographs depicting active erosion in Appendix 2 of the geotechnical report are for the Mud Creek watercourse, as indicated in the description, not for Wilson Cowand Drain, where no active erosion was recorded. In addition, Paterson recommended a 1m toe erosion allowance along the Wilson Cowan Drain based on the nature and size of the drain (i.e. not a permanent watercourse, anthropogenic not natural) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season. Therefore, based on our review, the recommended toe erosion allowance from the watercourse edge of 5 m for Mud Creek (main channel) and 1 m for Wilson Cowan Drain (western tributary), respectively is considered acceptable from a geotechnical perspective. Further justification for the toe erosion allowance has been included in our geotechnical report under section 6.8. In addition, Paterson revised the limit of hazard lands to show both the geotechnical limit of hazard lands setback based on our slope stability analysis, as well as the erosion hazard limit based on the Matrix Solutions geofluvial study, which considered a 5m toe erosion for Wilson Cowan Drain. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan.

Comment 2.32

The sensitivity results in section 6.9 should be derived from vane shear results.

Response:

For tree planting setbacks, the sensitivity of the clay was based on the Atterberg limit test results, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Sensitivity index which is calculated from the vane shear results is not used to determine tree planting setbacks, as per the City of Ottawa Guidelines for Tree Planting in Sensitive Marine Clays.

Comment 2.33
Please state if above ground swimming pools were contemplated in the section headed Swimming Pools in section 6.9.

Response:
Above ground swimming pools are contemplated *under section 6.9* in our geotechnical investigation report.

Comment 2.35
Section 7 should also include review of trees in proximity to foundations.

Response:
Noted. A statement has been added under section 7 indicating the requirement for completing a landscaping plan review by the geotechnical consultant. Please refer to the revised above-mentioned geotechnical report.

Comment 2.36
In Appendix 1 please add a determination, in the Symbols and Terms, of an n value of P.

Response:
The Symbols and Terms of ‘p’ reference in Appendix 1 is used to describe the “push spoon”, which we conducted to collect soil samples for testing. The definition of p has been added to the symbols list in Appendix 1.

Comment 2.37
It is suggested that a number of borehole logs should be modified due to the presence of a blow count record of P, yet the description is listed as “hard to very stiff”, for example, BH 1-21.

Response:
As explained in our response for comment 2.36, P (or push spoon) is not an SPT test. A push spoon sample is completed to collect a soil sample for visual observation and further testing. Therefore, it does not measure the consistency of the soil and it should not be correlated with N values.

Comment 2.38
Please discuss how the shear strength of BH 1-21 is 119 kPa at 4 m depth (with an N count of 5, while, at 5 m depth the shear strength is 139 with a blow count of P).

Response:
Please refer to our response to comment 2.37 and 2.38 above. It is erroneous to correlate P with the N value obtained from the SPT for clayey soils.

Comment 2.39
Please include DCPT results from 6.55 to 11 for borehole BH 3-21

Response:
The DCPT was pushed from 6.55 to 11 at the location of BH 3-21 with no recorded penetration resistance, which is typical for the grey silty clay deposit in Ottawa.

Comment 2.40
Please provide documentation confirming bedrock elevation.

Response:
Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth. Bedrock was not encountered within the maximum investigated depth of 6.4m. The proposed residential development is anticipated to consist of single and townhouse style residential homes, of slab-on-grade construction, and founded on shallow footings. Therefore, there is no requirement to determine the elevation of bedrock for the proposed residential development at the subject site, from a geotechnical perspective.

Comment 2.41
Please add DCPT results from 6.1 to 8.4 m to BH 5-21.

Response:
Refer to our response to comment 2.39 above.

Comment 2.42
Please include laboratory results for the sections shown on Appendix 2.

Response:
It is to be noted that the subsoil conditions at the analyzed cross-sections were inferred based on nearby boreholes, completed within the subject site, as well as on the results of the insitu vane shear tests, as discussed under section 6.8 of the above-mentioned revised geotechnical report.

Comment 2.43
The soil annotations on Figure 3 appear to be floating.

Response:
Noted. The annotations for soil layers in Figure 3 have been modified in the above-mentioned revised report.

Comment 2.44
Please include bathymetric survey data used for Figure 4 (amongst others).

Response:
The bottom elevations of the watercourses at the studied cross sections has been determined using a high precision GPS, during our site visit to review the slope conditions. These elevations have been added to the slope cross sections included in the revised geotechnical report referenced above.

Comment 2.45
The annotation in the red area is not legible.
Response:
Noted. The annotation in the red area has been enhanced to be legible. Please refer to the revised geotechnical report mentioned above.

Comment 2.46

Some non-circular slip circles should be analyzed (considering the soil types).

Response:

The analysis of the stability of the slopes was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. According to standard practice for slope stability analysis, a simple circular failure surface method is applicable for a slope in a homogenous soil layer. On the other hand, a non-circular failure surface would be investigated in case of a heterogeneous multi-soil layered slope. Based on the encountered subsurface conditions along the north and west slopes at the subject site, it is not required to complete a non-circular slip circle analysis for the subject slopes, from a geotechnical perspective.

Comment 2.47

It is suggested that additional cross-sections are required along north and west sides of the subdivision lands.

Response:

Based on our review of the existing slope conditions, five (5) slope cross-sections were studied as the worst-case scenarios and are considered sufficient, based on the observed side slopes and on the existing conditions. From a geotechnical perspective, additional cross-sections are not required along north and west sides of the subdivision lands. However, additional analysis considering proposed loading conditions, including the porposed grade raises, buildings & roads has been added to the revised geotechnical report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

Maha Saleh, M.A.Sc., P.Eng.

LICENSED PROFESSIONAL ENGINEER
October 17, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

PATERSON GROUP

memorandum

re: **Geotechnical Response to RVCA Comments**
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

to: Uniform Urban Developments Ltd. – **Mr. Ryan MacDougall –** rmacgougall@uniformdevelopments.com

date: October 17, 2023

file: PG5828-MEMO.02

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the RVCA listed in the letters dated April 27, 2023 and May 1, 2023 (File: 23-NEP-SUB-0041) regarding the proposed residential development at the aforementioned site as well as the porposed Park block to be located east of Rideau Valley Drive, along Rideau River. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5828-1 Revision 3 dated October 17, 2023 and PG5828-LET.01 Revision 2 dated October 17, 2023.

It should be noted that Paterson completed the previous and current slope stability analyses for the slopes along Mud Creek, Wilson Cowan Drain, and Rideau River at the subject sites based on current practice for slope stability analysis in Ottawa, and in accordance with the City of Ottawa Slope Stability Guidelines for Development Applications. The adopted methodology as well as the selection of soil parameters for the encountered soil properties have been done taking into account our vast experience in the area and in similar applications.

Discussion Topic 1: Geotechnical Investigation Report for the Proposed Residential Development, 4386 Rideau Valley Drive, Ottawa, Ontario; prepared by: Paterson Group; report no: PG5828-1; Rev no: 2; dated 14-Oct-2022.

Comment 1

In section 6.9 – General landscaping comments should include additional best practices recommendations, such as but not limited to:

- *i.) It is important to avoid directing uncontrolled water towards the slope (drainage, gutter, septic field, pool & hot tub drainage, etc.)*
- *ii.) It is important to avoid overloading the top of the slope (backfill, fill, miscellaneous waste, grass cuttings, branches, leaves, snow, etc.)*
- *iii.) It is important to avoid excavating at the base of the slope.*
- *iv.) It is important to maintain a healthy native vegetation cover.*
- *v.) Any future additions, such as aboveground swimming pools or accessory buildings, should entail reassessment of slope stability unless this has been pre-confirmed via supplementary slope stability analyses during the design stage.*

Response:

Noted. Additional considerations regarding the above items have been added to Subsection 6.9- Landscaping Considerations in the above mentioned revised geotechnical report.

Comment 2

Section 6.8 – Slope Conditions Along the Western Boundary: It is recommended to provide Paterson Group with the Matrix Solution report, since the field inspection was conducted before the fluvial geomorphological study. This will ensure that Paterson has all the relevant information and can make informed decisions and recommendations in their report.

Response:

The slope stability analysis completed by Paterson for the porposed development takes into account our field observations of the existing slope conditions along Mud Creek and Wilson Cowan Drain, made during our site visit on May 19, 2021. Having said that, Paterson reviewed the geo-fluvial study completed by Matrix Solutions, dated November 2022, for the proposed development. Based on our review of the above-noted study, it appears that the results of our slope stability study are in general agreement with the results of the geofluvial study for the majority of the proposed limit of hazard lands. The main deviation from the above-noted geofluvial study is the recommended toe erosion allowance along Wilson Cowan Drain. Paterson recommended a 1m toe erosion allowance along that drain based on the nature and size of the drain (i.e. not a permanent watercourse, anthropogenic not natural) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season, as opposed to the 5m toe erosion allowance suggested by the geofluvial study. It is to be noted that the geofluvial study did not provide photographs depicting active erosion along the Wilson Cowan Drain nor did Paterson note any active erosions during our previous site visit. Further justification for the toe erosion allowance has been included in our geotechnical report under section 6.8. In addition, Paterson revised the limit of hazard lands to show both the geotechnical limit of hazard lands setback based on our slope stability analysis, as well as the erosion hazard limit based on the Matrix Solutions geofluvial study. Having aid that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan.

Comment 3

Section 6.8 – Slope Stability analysis: Soil strength parameters (c and Φ) for drained (effective stress conditions) and undrained (total stress conditions), as well as information for the rational on how they were established should be provided within the body of the report (how are they inferred from in situ and laboratory testing, any correlations used?). There is currently not sufficient information to accept that soil strength parameters used by the consultant reflect accurately the site conditions.

Response:

The soil strength parameters for drained and undrained conditions used in the slope stability analysis were chosen based on the subsurface conditions observed in the test holes located within the proximity of the slopes, and our general knowledge of the geology in the area. Furthermore, the adopted soil strength parameters are within the range of recommended values for different soil layers based on the City of Ottawa's slope stability guidelines and academic literature such as M.A. Klugman and P. Chung, 1976. Further discussion on the selection of the soil strength parameters has been added to Subsection 6.8- Slope Stability Assessment, in the above mentioned geotechnical report.

Comment 4

Section 6.8 – Slope Stability analysis: We noted that soil strength parameters for grey softer clays under the drained static analyses were higher than for the upper brown clays (desiccated crust), please explain rational, as in standard practice the contrary is observed.

Response:

Based on the City of Ottawa's slope stability guidelines and academic literature such as M.A. Klugman and P. Chung, 1976, brown clay has lower cohesion values compared to grey clay. Due to the loss of water in Brown silty clay and weathering of the silty clay particle, the cohesion values are decreased in comparison with the grey clay. However, it should be noted that our calculations and assumptions in the slope stability models are in the range of recommended values for different soil layers based on the above noted guidelines.

Comment 5

We noted that only drained analyses were undertaken for the static conditions. It is generally geotechnical best practice to undertake both drained and undrained analyses when in presence of clayey soils, even if the drained conditions governed.

Response:

Paterson completed the slope stability assessment for the slopes along Mud Creek and Wilson Cowan Drain, within the subject site, in accordance with best practice for slope stability analysis in Ottawa as well as the City of Ottawa's slope stability guidelines. Based on the City guidelines for slope stability analysis, the potential for a drained failure should be checked for the case of slow loading (i.e. realistic condition of natural slope) whereas that of undrained failure should be checked for the case of sudden or short term loading (i.e. seismic loading). Completing an undrained analysis under static loading would always provide a higher safety factor compared to the same undrained analysis completed under seismic loading, because it would be the same analysis minus the seismic load.

The critical scenario in this case is the undrained analysis under seismic loading. Reference should be made to Subsection 6.8 -Slope Stability Assessment in the abovementioned geotechnical report for further details on the analysis methodology.

Comment 6

Please provide information within the body of the report to support that the clay is not sensitive.

Response:

The sensitivity index of the encountered silty clay deposit was calculated based on the ratio between the undisturbed and remolded shear vane test measured in the field, for all the boreholes, and it was found to be generally below 4, indicating a normal sensitivity clay. Please refer to Subsection 4.2-Subsurface Profile, in the abovementioned geotechnical report.

Comment 7

Additionally, the sections should display the water level used in the stability. Generally, it should consider the design low water level (present flow) as well as the 100-year flood level.

Response:

The water level used in the analysis is displayed on the cross sections in the previous and current geotechnical reports. The slope stability analysis was completed for the worst-case scenario at several cross sections, considering a conservative review of the groundwater conditions, where the silty clay deposit was considered to be fully saturated and the groundwater level was taken at ground surface, which is common practice for completing slope stability analysis for natural slopes in Ottawa. The 100- year flood level is typically completed for storm ponds in confined excavations and would generally yield a higher safety factor for slope stability as compared to the current water level in the watercourse due to the balancing of the hydrostatic pressure.

Comment 8

Section 5.3 – Permissible Grade Raise Restriction allow for up to 2 m of fill to be added. This scenario should be analysed where fill is proposed to ensure that this would not negatively affect the Factor of Safety (FoS). It would be important to consider potential water seepage/perched water table at the interface of the fill and impermeable existing clay layer that could result after the placement of the fill material (expected to be more permeable).

Response:

Paterson completed additional slope stability analyses for the proposed conditions considering an approximate average grade raise of 2m at the location of the studied cross sections areas. The new slope stability cross sections account for the proposed grade raise as well as the proposed buildings/roads within the development. Based on our slope stability analysis, a stable slope setback varying between 1.3 and 5.3 m from the top of the slope are required to achieve a factor of safety of 1.5 for the limit of the hazard lands along Mud Creek. The results of the new slope stability analysis have been added to the abovementioned geotechnical report. Reference should be made to Drawing PG5828-1 – Test Hole Location Plan for the proposed Limit of Hazard Lands setback for development considerations at the subject site.

Comment 9

Where applicable, on lots along the slopes, surcharge from proposed structures/roads should be incorporated within the analyses.

Response:

Refer to our response for Comment 8 above.

Comment 10

Section 6.8 – Limit of Hazard Lands: The consultant established a toe erosion allowance of 5 m along Mud Creek and 1m along Wilson-Cowan drain based on their review of erosion on site with a future 6 m erosion access allowance. This is supplemented with a stable slope allowance where needed. Please update with a toe allowance of 5 m along all watercourses as recommended in the Fluvial Geomorphic and Erosion Hazard Assessment prepared by Matrix Solution Inc.

Response:

Refer to our response for Comment 2 above.

Slope Stability Assessment; Proposed River Park, 4386 Rideau Valley Drive - Ottawa, Ontario; prepared by: Paterson Group Report PG5828-LET.01 Rev. 1 dated: July 5th, 2022.

Comment 11

The study may have to be revised such as to address the following: a. Section 2.0 – Slope Stability analysis: Please confirm if the Rideau Valley Road is present within the analysis sections. We would generally recommend that it be labelled, modelled as fill with proper traffic transient loading conditions.

Response 11

The slope stability analysis does not include the Rideau Valley Road since it is located far enough from the top of slope and will have negligible influence on the slope stability of the subject slope.

Comment 12

Soil strength parameters (c and Φ) for drained (effective stress conditions) and undrained (total stress conditions), as well as information for the rational on how they were established should be provided within the body of the report (how are they inferred from in situ and laboratory testing, any correlations used?). There is currently not sufficient information to accept that soil strength parameters used by the consultant reflect accurately the site conditions.

Response

Refer to our response for Comment 3 above.

Comment 13

We noted that soil strength parameters for grey softer clays under the drained static analyses were higher than for the upper brown clays (desiccated crust), please explain rational, as in standard practice the contrary is observed.

Response:

Refer to our response for Comment 4 above.

Comment 14

We noted that only drained analyses were undertaken for the static conditions. It is generally geotechnical best practice to undertake both drained and undrained analyses when in presence of clayey soils, even if the drained conditions governed.

Response:

Refer to our response for Comment 5 above.

Comment 15

Please provide information within the body of the report to support that the clay is not sensitive.

Response:

Refer to our response for Comment 6 above.

Comment 16

Additionally, the sections should display the water level used in the stability. Generally, it should consider the design low water level (present flow) as well as the 100-year flood level.

Response:

Reference should be made to our response for Comment 7 above.

Erosion Hazard General Comments

Comment *17*

As mentioned in the Geotechnical Investigation comments above, it is important to avoid directing water and discharging it in an uncontrolled manner towards the slopes.

Response:

Noted. Reference should be made to the revised letter report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

Maha Saleh, M.A.Sc., P.Eng.

LICENSED PROFESSIONAL ENGINEER
October 17, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

re: **Geotechnical Response to City Comments**
Proposed Residential Development
4386 Rideau Valley Drive, Ottawa, Ontario

to: Uniform Urban Developments Ltd. – **Mr. Ryan MacDougall –** rmacgougall@uniformdevelopments.com

date: July 4, 2024

file: PG5828-MEMO.03

Further to your request and authorization, Paterson Group (Paterson) prepared the following memorandum to provide responses to the geotechnical-related comments from the City of Ottawa listed in the letter dated June 14, 2024 (File Nos. D02-02-22-0118, D07-16-22-0026) regarding the proposed residential development at the aforementioned site. This memorandum should be read in conjunction with Paterson Geotechnical Report PG5828-1 Revision 5 dated July 18, 2024.

Geotechnical Investigation Comments

(City01): Comment 2.11 *Please refer to the watercourses as Mud Creek and the Wilson Cowan Drain, rather than Mud Ruisseau Creek and tributary, to remain consistent with other reports and plans submitted.*

Paterson's Previous Response: Noted. Reference to the watercourses has been modified in our revised geotechnical report mentioned above, as requested.

(City02): Outstanding: *There are still some references to 'Mud Ruisseau' in your report. (Pages 71 thru 75 of 114, "Photographs From Site Visit – May 19, 2021").*

Response:
Noted. Reference to the watercourses has been modified in our revised geotechnical report mentioned above.

(City01): Comment 2.15 *Do the results of your study of the Slope Stability study align with the results from the Geo-fluvial Study? [page 18 of 65].*

Paterson's Previous Response: Paterson reviewed the geo-fluvial study completed by Matrix Solutions, dated November 2022, for the proposed residential development. Based on our review of the above-noted study, it appears that the results of our slope stability study are in general agreement with the results of the geofluvial study for the majority of the proposed limit of hazard lands with the exception of the recommended toe erosion allowance along Wilson Cowan Drain. Paterson is recommending 1m for toe erosion along that drain based on the nature and size of the drain (i.e. not a permanent watercourse) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season, as opposed to

5m for toe erosion as suggested by the geofluvial study. Furthermore, the geofluvial study did not provide photographs depicting active erosion along the Wilson Cowan drain. Further justification for the toe erosion allowance has been included in our geotechnical report under subsection 6.8. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan and the erosion limit proposed by Matrix solutions as well as the limit of hazard lands proposed by Paterson are both outside the limits of the proposed development.

(City02): Outstanding: *The Slope and Hazard Land layouts do not agree with that provided in the City's 'Slope Stability Guidelines (Dec-2004)', Figures 12 and 13. See attached. In addition, as the Fluvial report recommends a 5-metre toe erosion, this is the value that the City feels is applicable. Further the fluvial geomorphology report should be taken as superior to the geotechnical report for fluvial issues.*

Response:
This comment has been acknowledged. The toe erosion along the Wilson Cowan Drain has been revised to 5.0m. Please refer to the above-mentioned revised report.

(City01): Comment 2.22 *A longer-term, or year-long groundwater level analysis is required.*

Paterson's Previous Response: Based on our understanding, LID measures are not considered for the subject site. Therefore, year-long groundwater level is not required from a geotechnical perspective at the subject site.

(City02): Outstanding: *An accurate seasonal high groundwater level is necessary for the general design of subdivisions. All as per the Sewer Design Guidelines (Section 8.3.13) and the City's Low Impact Development Technical Guidance Report (Section 2.3.3, sheet 14 of 68).*
Please note that 'Low Impact Development' within subdivisions is also required as per the MECP Bulletin: 'Interpretation Bulletin, Ontario Ministry of Environment and Climate Change Expectations Re: Stormwater Management, February 2015'.

> *"Low impact development stormwater management is relevant to all forms of development, including new development, redevelopment, infill, and retrofit development." (page 2 of 7)*
>
> *"Infiltration of stormwater is needed to maintain ground water sources of drinking water, and to maintain stream base flows. At the same time, ground water quality must be protected from contamination, requiring the appropriate selection of LID measures, which would be determined by the hydrogeology*

of an area." (page 3 of 7)
The City notes that the 'Conceptual Site Servicing & Stormwater Management Report' provided with this application already provides some general guidance on LID Design. See Section 4.4.3 (sheet 21 of 324). This information should be referenced here.

Response:
????

(City01): Comment 2.23 *Groundwater cannot be stated to be expected to lower based on the LID directive documents without analysis showing that it will be so (with similitude, if necessary/appropriate).*

Paterson's Previous Response**:** Reference should be made to our response to comment 2.22 above. Furthermore, it is unclear what the reviewer is referring to LID directives. Further clarification is required. In any case, post-development groundwater level lowering is conservatively anticipated following construction of site servicing at residential developments, as observed by Paterson from previous similar jobs.

(City02): Outstanding: *See City of Ottawa response to Comment 2.22 (above) and the LID Technical Guidance Report declines estimations of groundwater lowering with development.*

Response:
????

(City01): Comment 2.27 *The comments that the subject site are not susceptible to liquefaction requires an exhaustive discussion: whichever approach the consultant takes will require proof of similitude and full copies of papers provided to the City showing unequivocal support.*

Paterson's Previous Response: The soils encountered at the subject site consist of silty clays, which are cohesive in nature. These soils were evaluated for liquefaction susceptibility in accordance with the criteria prepared by Bray at al. 2004 which determines that all soils with a plasticity index exceeding 20% are not liquifiable (Figure 1). In general, the plasticity index results completed on samples taken from the silty clay layer were found to be above 20. Therefore, the encountered soils are not susceptible to liquefaction. Reference should be made to subsection 5.4- Design for Earthquakes in the abovementioned revised geotechnical report, for further details on liquefaction susceptibility at the subject site.

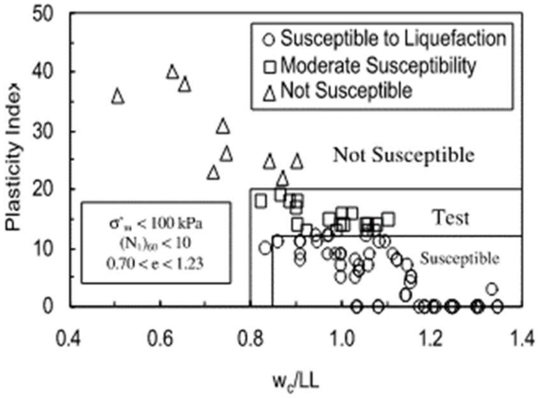

Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

(City02): Outstanding: *While the City understands the comparison implied here, we need to see testing or other data that confirms that this specific site meets these requirements.*

Response:
During our site investigation, Paterson conducted several field and laboratory tests to evaluate soil liquefaction potential. These included the Standard Penetration Test (SPT), which measures soil resistance to penetration using a hammer-driven sampler. Field vane testing was also completed within the silty clay deposits encountered in the test holes to assess soil strength under pore water pressure conditions. Shear strength values obtained from the field vane ranged between 50 and >200 kPa.
Additionally, Plasticity Index (PI) tests were conducted on selected soil samples to assess cohesive soil plasticity based on liquid and plastic limits. As previously indicated, the results showed a plasticity index above 20%. Based on these findings, the conducted field and laboratory testing provide sufficient evidence from a geotechnical perspective to confirm that the soils at the subject site are not susceptible to liquefaction.

(City01): Comment 2.31 *A toe erosion allowance of 1 m is not acceptable. The comments on "active erosion was not observed" are contested in a number of the photographs in Appendix 2. The toe erosion allowance, under the heading of Toe Erosion and Access Allowances shall be revised as per Table 3 of the Ministry of Natural Resources, and Forestry (MNRF) Technical Guide- River and Stream Systems: Erosion Hazard Limit due to the active erosion and the soils of the boreholes.*
It is noted that the Fluvial Geomorphic and Erosion Hazard Assessment completed by Matrix Solutions Inc. recommended a 5 m toe erosion allowance for the Wilson Cowan Drain. Based on the penetration resistance blows of the Soil Profile and Test Data Sheets the soils on site may be Soft/Firm Cohesive Soils, loose granular, (sand, silt) fill, in the MNRF Guide.

Paterson's Previous Response: Based on our field review and engineering analysis, active erosion was not encountered along the western watercourse at Wilson Cowan drain. It is to be clarified that the photographs depicting active erosion in Appendix 2 of the geotechnical report are for the Mud Creek watercourse, as indicated in the description, not for Wilson Cowand Drain, where no active erosion was recorded. In addition, Paterson recommended a 1m toe erosion allowance along the Wilson Cowan Drain based on the nature and size of the drain (i.e. not a permanent watercourse, anthropogenic not natural) and the fact that the drain is mostly dry for the majority of the year outside the snow melt season. Therefore, based on our review, the recommended toe erosion allowance from the watercourse edge of 5 m for Mud Creek (main channel) and 1 m for Wilson Cowan Drain (western tributary), respectively is considered acceptable from a geotechnical perspective. Further justification for the toe erosion allowance has been included in our geotechnical report under section 6.8. In addition, Paterson revised the limit of hazard lands to show both the geotechnical limit of hazard lands setback based on our slope stability analysis, as well as the erosion hazard limit based on the Matrix Solutions geofluvial study, which considered a 5m toe erosion for Wilson Cowan Drain. Having said that, it is understood that Novatech considered a conservative setback which takes into account 5m of toe erosion along Wilson Cowan Drain in their site plan.

(City02): Outstanding: *As discussed in comment 2.15 above, as the Fluvial report recommends a 5-metre toe erosion, this is the value that the City recognizes.*

Response:
This comment has been acknowledged. The toe erosion along the Wilson Cowan Drain has been revised to 5.0m. Please refer to the above-mentioned revised report.

(City01): Comment 2.37 *It is suggested that a number of borehole logs should be modified due to the presence of a blow count record of P, yet the description is listed as "hard to very stiff", for example, BH 1-21.*

Paterson's Previous Response: As explained in our response for comment 2.36, P (or push spoon) is not an SPT test. A push spoon sample is completed to collect a soil sample for visual observation and further testing. Therefore, it does not measure the consistency of the soil and it should not be correlated with N values.

(City02): Outstanding: *The N values provided on BH 1-21 at the 4m, 5m, and 6m depths states that the N value are 'P' (or push, or no resistance implying very soft soils). This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise.*

Response:
As we previously explained, P (or push spoon) is not an SPT test and is completed just to collect a soil sample for visual observation and further testing only. It does not measure the consistency of the soil, and therefore, it should not be correlated with N values. *The description of the soil as hard to very stiff soils is obtained from our field observations and the completed* field vane testing within the silty clay deposits. Shear strength values obtained from the field vane at this borehole location and at 4m and 5m depth ranged between 139 kPa and 119 kPa, respectively. Please reference the symbols and terms in Appendix 1 in the above-mentioned report for the consistency guide or range based on the undrained shear strength values.

(City01): Comment 2.38 *Please discuss how the shear strength of BH 1-21 is 119 kPa at 4 m depth (with an N count of 5, while, at 5 m depth the shear strength is 139 with a blow count of P).*

Paterson's Previous Response: Please refer to our response to comment 2.37 and 2.38 above. It is erroneous to correlate P with the N value obtained from the SPT for clayey soils.

(City02): Outstanding: *The N values provided on BH 1-21 at the 4m, 5m, and 6m depths states that the N value are 'P' (or push, or no resistance implying very soft soils). This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise.*

Response:
Please refer to our response to comments 2.37 above.

(City01): Comment 2.39 *Please include DCPT results from 6.55 to 11 for borehole BH 3-21*

Paterson's Previous Response: The DCPT was pushed from 6.55 to 11 at the location of BH 3-21 with no recorded penetration resistance, which is typical for the grey silty clay deposit in Ottawa.

(City02): Outstanding: *The DCPT results suggest soft soils. This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise.*

Response:
As explained, at BH 3-21, the DCPT showed no recorded penetration resistance from depths of 6.55 to 11 meters, indicating stiff consistency of the soil at this borehole location, typical for grey silty clay deposits in Ottawa. However, hard to very stiff soils were measured at BH 1-21, BH 4-21, BH 5-21, BH 6-21, and BH 1-22 at depths ranging from 3 to 5m, characteristic of brown silty clay deposits.
Overall, our investigation revealed that the silty clay deposits generally consist of a hard to very stiff brown weathered crust extending from 1.5 to 5.2m below the ground surface, followed by stiff grey silty clay at BH 1-21, BH 3-21, BH 4-21, BH 5-21, BH 6-21, and BH 1-22. Therefore, there are contradicting in our description of the encountered soils.

(City01): Comment 2.40 *Please provide documentation confirming bedrock elevation.*

Paterson's Previous Response: Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth. Bedrock was not encountered within the maximum investigated depth of 6.4m. The proposed residential development is anticipated to consist of single and townhouse style residential homes, of slab-on-grade construction, and founded on shallow footings. Therefore, there is no requirement to determine the elevation of bedrock for the proposed residential development at the subject site, from a geotechnical perspective.

(City02): Outstanding: *Please confirm that all the proposed homes will be constructed as slab on grade.*

Response:
This needs to be confirmed with the client

(City01): Comment 2.41 *Please add DCPT results from 6.1 to 8.4 m to BH 5-21*.

Paterson's Previous Response: Refer to our response to comment 2.39 above.

(City02): Outstanding: *The Dynamic Cone Penetration Tests (DCPT) results suggest soft soils. This seems to contradict the description of the soil as hard to very stiff soils. Please review and advise*

Response: Refer to our response to comment 2.39 above.

(City01): Comment 2.43 *The soil annotations on Figure 3 appear to be floating.*

Paterson's Previous Response: Noted. The annotations for soil layers in Figure 3 have been modified in the above-mentioned revised report.

(City02): Outstanding: As established in the 'Fluvial Geomorphic and Erosion Hazard Assessment' the toe erosion allowance should be 5 metres. Page 23 of 46, Section 4.3.2.

Response:
This comment has been acknowledged. The toe erosion along the Wilson Cowan Drain has been revised to 5.0m. Please refer to the above-mentioned revised report.

(City01): Comment 2.46 *Some non-circular slip circles should be analyzed (considering the soil types).*

Paterson's Previous Response: The analysis of the stability of the slopes was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. According to standard practice for slope stability analysis, a simple circular failure surface method is applicable for a slope in a homogenous soil layer. On the other hand, a non-circular failure surface would be investigated in case of a heterogeneous multi-soil layered slope. Based on the encountered subsurface conditions along the north and west slopes at the subject site, it is not required to complete a non-circular slip circle analysis for the subject slopes, from a geotechnical perspective.

(City02): Outstanding: *Referencing Figure 3, page 52 of 114, three soil types are indicated to be included in the slip circle. Also note that grey silty clay soils are a significantly weaker soil and not considered homogenous. The City will need to see a couple of non-circular failure surface calculations.*

Response:
This comment has been acknowledged. Multiple non-circular failure surfaces have been added to Figure 3. Please refer to the above-mentioned revised report.

We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

July 18, 2024

Zubaida Al-Moselly, P.Eng.

Faisal I. Abou-Seido, P.Eng.

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Retaining Wall Design
Noise and Vibration Studies

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October 17, 2023
PG5828-LET.01 Rev. 2

Uniform Developments
300-117 Centrepoint Drive
Ottawa, Ontario
K2G 5Y6

Attention: **Mr. Ryan MacDougall**

Subject: **Slope Stability Assessment**
Proposed River Park
4386 Rideau Valley Drive - Ottawa, Ontario

Dear Sir,

Paterson Group (Paterson) was commissioned by Uniform Developments to conduct a slope review for the proposed river park to be located across 4386 Rideau Valley Drive in the City of Ottawa, Ontario.

1.0 Field Observation

The field program for the proposed river park was completed on June 16, 2022. At that time, a total of two boreholes were advanced down to a maximum depth of 5.9 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5828-2 – Limit of Hazard Lands Plan attached to this letter.

Surface Conditions

The subject site is currently vacant and covered with grass and trees. It is bound to the east by Rideau River, to the west by Rideau Valley Drive followed by a future development, to the south by a single-family dwelling, and to the north by a similar vacant lot. The ground surface across the subject site is generally flat and gently sloping upwards towards the south and west from an approximate geodetic elevation of 80 m at the north to 88 m at the south. The site is approximately 1.5 to 2.0m lower than Rideau Valley Drive. The southern portion of the site is generally covered with mature trees.

The slope conditions were reviewed by Paterson on May 17, 2022. The existing slopes were generally observed to be covered with well rooted vegetation across the surface. The western slopes were observed to be approximately 2 to 3 m high and appeared to have a relatively steep profile of less than 1H:1V. On the other hand, the eastern slopes were observed to be 4 to 5m high and appeared to have a slope profile ranging between 2H:1V to 3H:1V.

The width of the Rideau River was noted to be between 26 m wide to the south and 80 m wide to the north along the site length. The majority of the riverbed appeared to be covered by an in-situ stiff to stiff brown silty clay. The majority of the riverbanks were observed to be affected by active erosion and were exposed directly to stream flow. Additional signs of erosion consisted of exposed tree roots.

Subsurface Conditions

Generally, the subsurface soil profile at the test hole locations consists of topsoil underlain by a deposit of very stiff to stiff brown silty clay underlain by glacial till. The brown silty clay was observed to be underlain by a stiff grey silty clay at BH 1-22. Glacial till was encountered below the clay deposit at all boreholes. The glacial till deposit was generally observed to consist of compact to dense brown silty sand with gravel, cobbles and boulders. Practical refusal to augering was encountered at an approximate depth of 5.9m and 2.7m at the locations of BH 1-22 and 2A-22, respectively. Practical refusal to DCPT was encountered at an approximate depth of 4.24m at BH 2-22. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Based on available geological mapping, the bedrock in the subject area consists of Dolomite of the Oxford formation, with an overburden drift thickness of 10 to 25 m depth.

2.0 Slope Stability Assessment

The existing slope conditions were reviewed by Paterson to define a conceptual limit of hazard lands setback, which is to be respected for any permanent structures, such as gazebos. It should be noted that stone dust paths with minor grading adjustments and park benches are acceptable to be placed within the limit of hazard lands line from a geotechnical perspective. The proposed limit of hazard lands designation line consists of the following:

- ☐ a stable slope with a minimum factor of safety of 1.5 under static conditions and 1.1 under seismic loading
- ☐ a toe erosion allowance
- ☐ a 6 m access allowance and top of slope

Three slope cross sections were studied as the worst-case scenario. The cross-section locations are presented on Drawing PG5828-2 – Limit of Hazard Lands Plan attached to this report.

Stable Slope Setback

The analyses of the stability of the slopes were carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. Minimum factors of safety of 1.5 and 1.1 are generally recommended for static and seismic conditions, respectively, where the failure of the slope would endanger permanent structures.

The cross-sections were analysed using the existing slope geometry from the topographical site survey provided by the client and information collected during our site visit. The slope stability analysis was completed at the slope cross-sections under worst-case-scenario by assigning cohesive soil layers as being fully saturated.

Subsoil conditions at the cross-section locations were determined based on test holes coverage conducted within the subject site. The soil profile used in the slope stability analysis for cross section 1 was based on borehole BH 1-22 and that for cross sections 2 and 3 was based on BH 2-22 and BH 3-22. The soil profile considered in the slope stability analysis generally consists of stiff to very stiff silty clay underlain by glacial till. Within the vicinity of cross sections 2 and 3, the clay consists of a brown silty clay crust underlain by a stiff grey silty clay. For a conservative review of the groundwater conditions, the silty clay deposit was noted to be fully saturated for our analysis.

Table 1 – Effective Stress Soil Parameters (Static – Drained Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	33	5
Grey Silty Clay	16	33	10
Glacial Till	20	36	5

Table 2– Total Stress Soil Parameters (Seismic - Undrained Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown Silty Clay	17	-	150
Grey Silty Clay	16	-	65
Glacial Till	20	36	5

Static Loading Analysis

The results are shown in Figures 1, 3, and 5. The results indicate a slope with a factor of safety of 1.16, 1.66, and 0.4 at Sections 1, 2, and 3, respectively. Based on these results, a stable slope setback varying between 7 and 9 m from the top of the slope are required for sections 1-1 and 3-3 to achieve a factor of safety of 1.5 for the limit of the hazard lands in the park area. Section 2-2 will not require a stable slope allowance.

Seismic Loading Analysis

An analysis considering seismic loading and the groundwater at ground surface was also completed. A horizontal acceleration of 0.16g was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading. The results of the analyses including seismic loading are shown in Figures 2, 4, and 6. The results indicate a slope with a factor of safety greater than 1.1 at all sections. However, it should be noted that the stable slope setback associated with our static loading analysis governs the required stable slope setback required for static conditions.

Toe Erosion and Access Allowances

Based on the soil profiles encountered at the borehole locations and the soil encountered throughout the river, a stiff grey silty clay is anticipated to be subject to erosion activity by the river flow. Based on the encountered soils and the observed active erosion, a toe erosion allowance of 5 m should be applied for the subject slope. Furthermore, a minimum 6 m access allowance should be considered.

Limit of Hazard Lands

Based on the above, a setback taken from the top of the current slope has been provided as based on the above-noted observations and analysis. Reference should be made to Drawing PG5828-2 – Limit of Hazard Lands Plan for the proposed River Park at the subject site.

Drainage Requirements

It should be noted that the following should be considered for the proposed park:

- ❐ It is important to avoid directing uncontrolled water towards the slope (drainage, gutter, pool drainage, etc.)
- ❐ It is important to avoid overloading the top of the slope (backfill, fill, miscellaneous waste, grass cuttings, branches, leaves, snow, etc.)
- ❐ It is important to avoid excavating at the base of the slope.
- ❐ It is important to maintain a healthy native vegetation cover.
- ❐ Any future additions, such as aboveground swimming pools or accessory buildings, should entail reassessment of slope stability unless this has been pre-confirmed via supplementary slope stability analyses during the design stage.

3.0 Conclusions

The recommendations provided in this letter report are in accordance with Paterson's present understanding of the project. Should any conditions at the site be encountered which differ from our site observations, Paterson requests immediate notification to permit reassessment of the recommendations.

The present letter report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Uniform Developments, or her agents, is not authorized without review by Paterson Group Inc. for the applicability of our recommendations to the altered use of the report.

We trust this report meets your present requirements.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
October 17, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Maha Saleh, M.A.Sc., P.Eng.

David J. Gilbert, P.Eng

Attachments

- ❏ Soil Profile and Test Data Sheets
- ❏ Symbols
- ❏ Figures 1 to 6 - Sections for Slope Stability Analysis
- ❏ Drawing PG5828-2 – Limit of Hazard Lands Plan

Report Distribution

- ❏ Uniform Developments (e-mail copy)
- ❏ Paterson Group (1 copy)

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed River Park - 4386 Rideau Valley Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

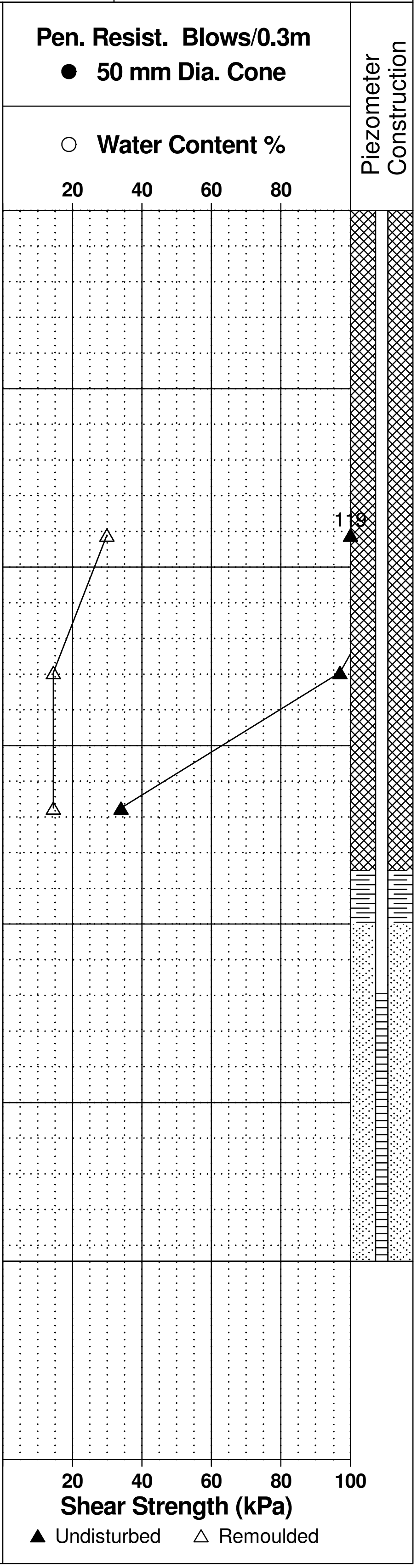
FILE NO. **PG5828**

HOLE NO. **BH 1-22**

SOIL DESCRIPTION	Depth (m)
Ground Surface	
TOPSOIL	0.25
Very stiff to stiff, brown **SILTY CLAY,** trace sand and gravel	0.69
Very stiff to firm, brown **SILTY CLAY** - grey by 3.0m depth	3.50
GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders, trace clay	5.89
End of Borehole Practical refusal to augering at 5.89m depth	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	7
SS	3	100	P
SS	4		P
SS	5	25	P
SS	6	17	15
SS	7	8	10
SS	8	38	50+

DEPTH (m)	ELEV. (m)
0	83.47
1	82.47
2	81.47
3	80.47
4	79.47
5	78.47



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed River Park - 4386 Rideau Valley Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

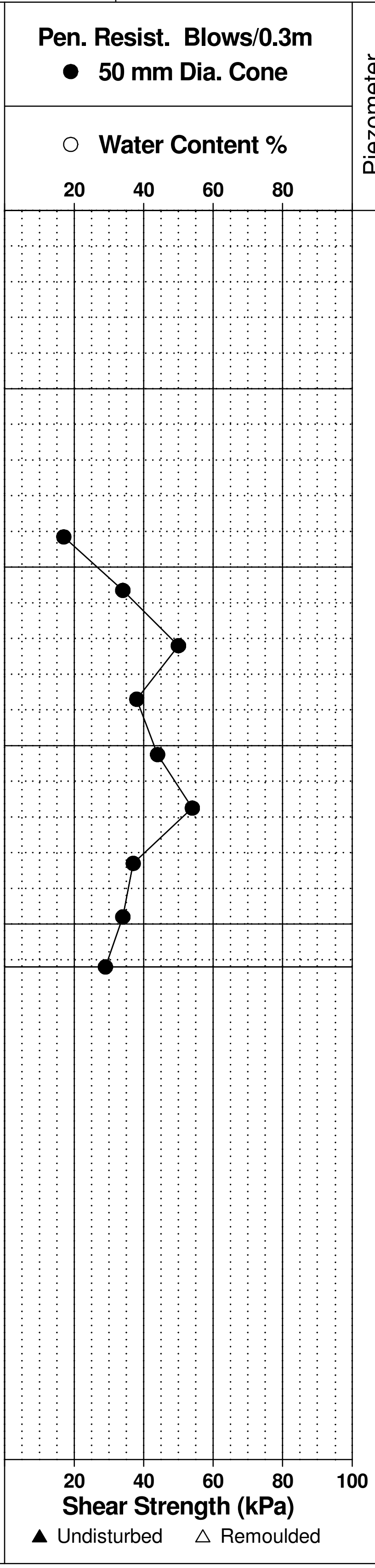
BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

FILE NO. **PG5828**

HOLE NO. **BH 2-22**

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	85.56
TOPSOIL 0.33						
Stiff to firm, brown **SILTY CLAY,** some sand	AU	1				
	SS	2	75	9	1	84.56
1.45						
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders 1.65	SS	3	100	50+		
Dynamic Cone Penetration Test commenced at 1.65m depth.					2	83.56
					3	82.56
					4	81.56
4.24						
End of Borehole Practical DCPT refusal at 4.24m depth (BH dry upon completion)						



Piezometer Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed River Park - 4386 Rideau Valley Drive
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

FILE NO. **PG5828**

HOLE NO. **BH 2A-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone; ○ Water Content %	Piezometer Construction
Ground Surface						0	85.74		
OVERBURDEN						1	84.74		
1.52									
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders	^^^	SS	1	67	42	2	83.74		
2.67									
End of Borehole Practical DCPT refusal at 2.67m depth (BH dry upon completion)									

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.
p	-	Push spoon sampling

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

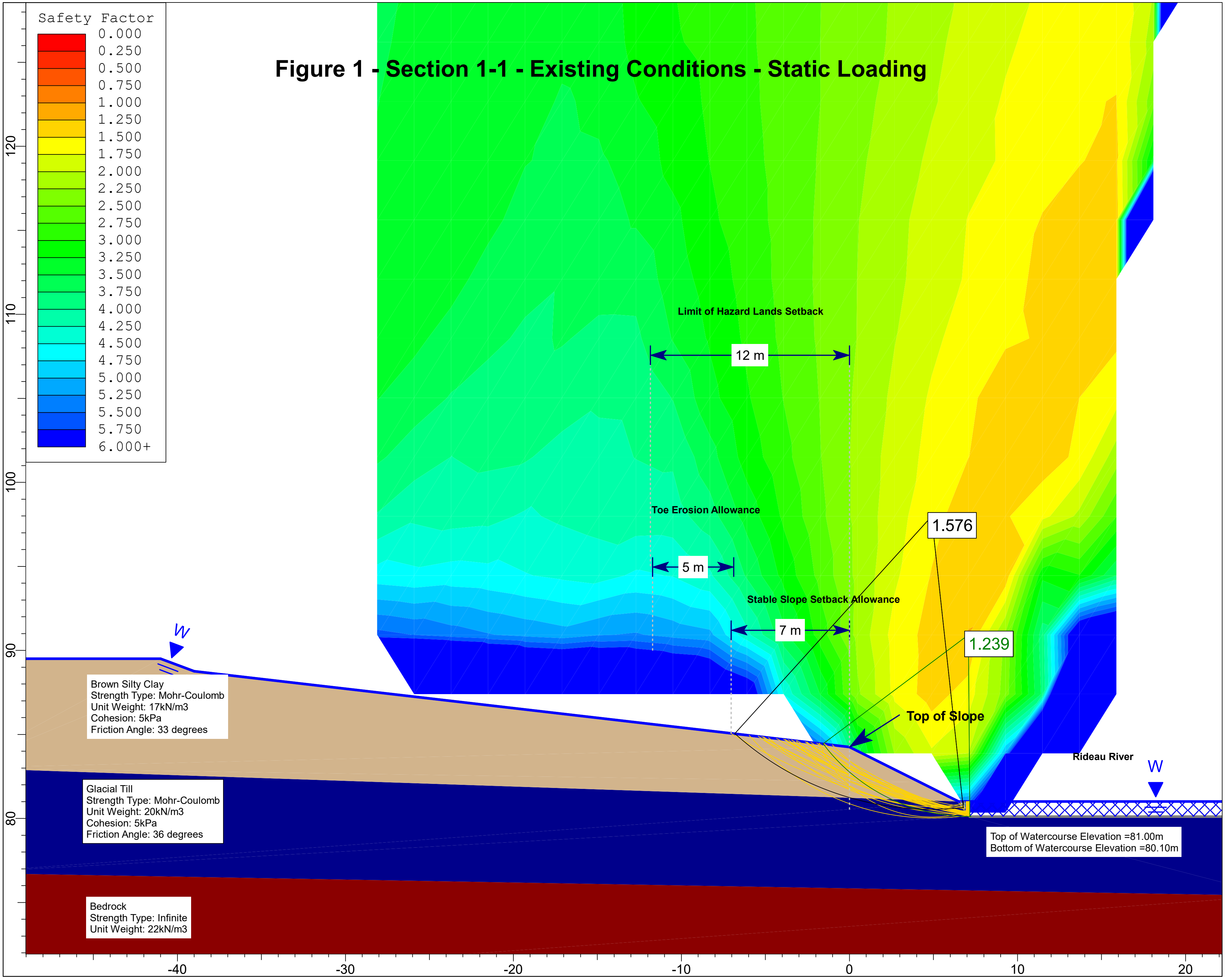


Figure 1 - Section 1-1 - Existing Conditions - Static Loading

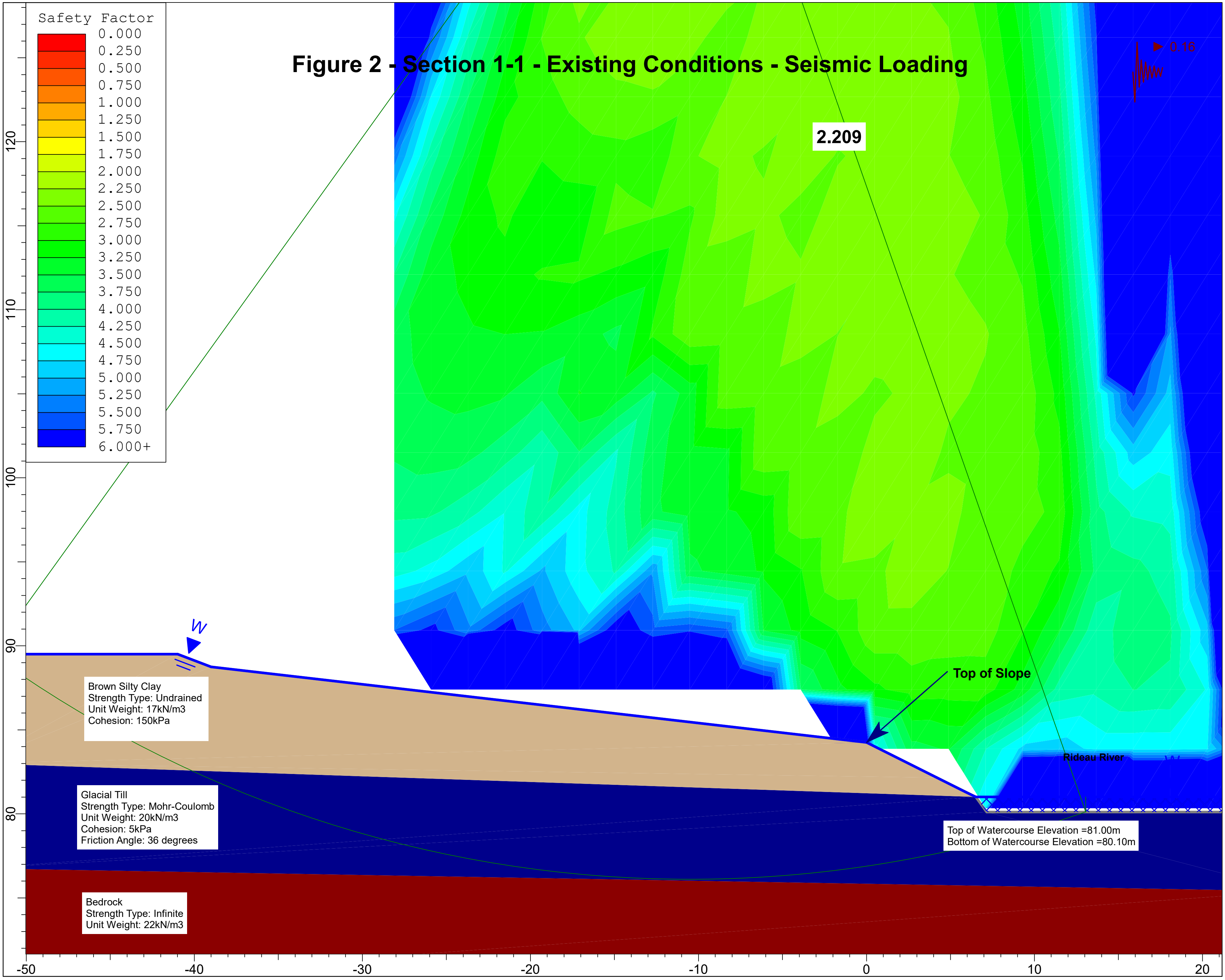


Figure 2 - Section 1-1 - Existing Conditions - Seismic Loading

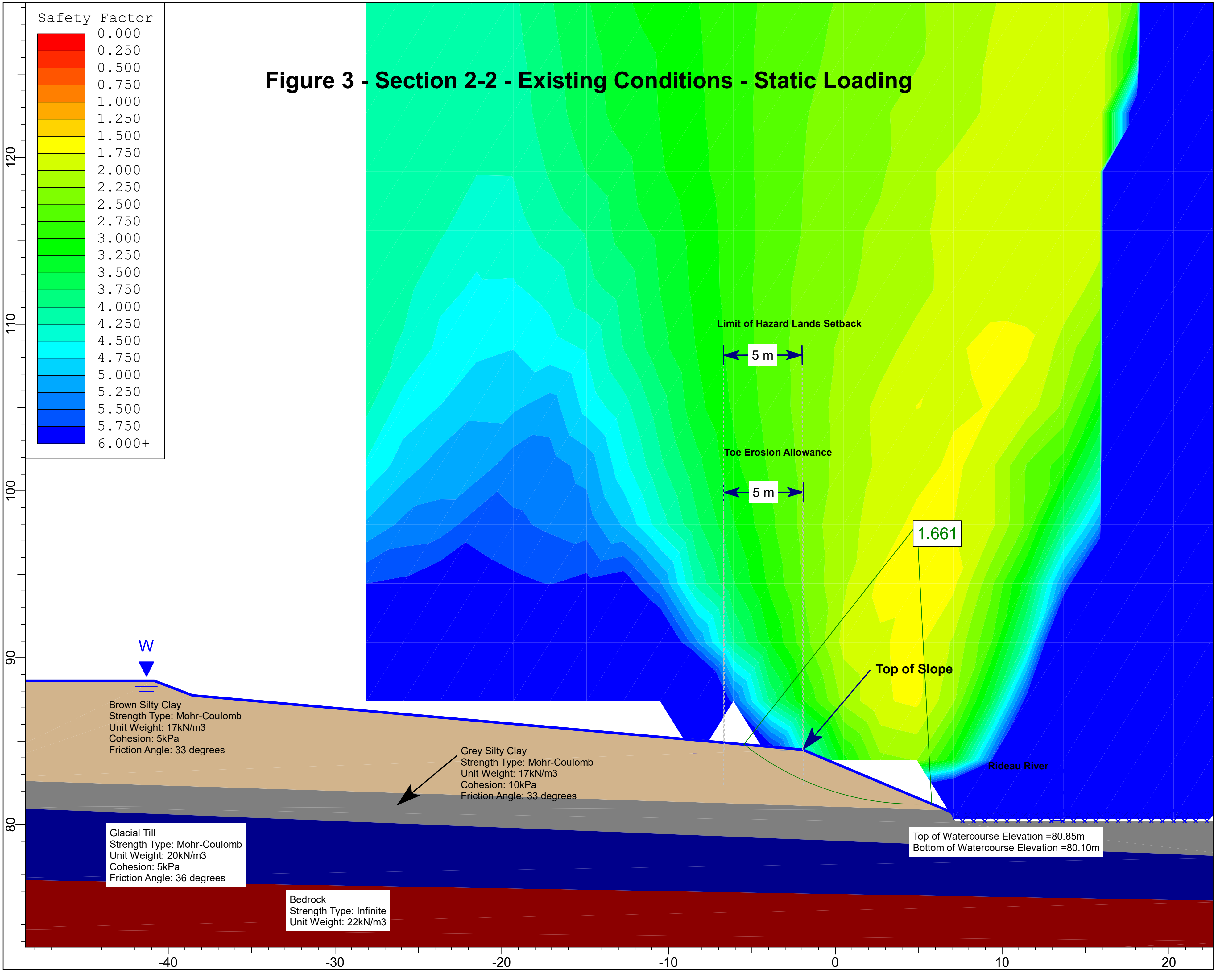


Figure 3 - Section 2-2 - Existing Conditions - Static Loading

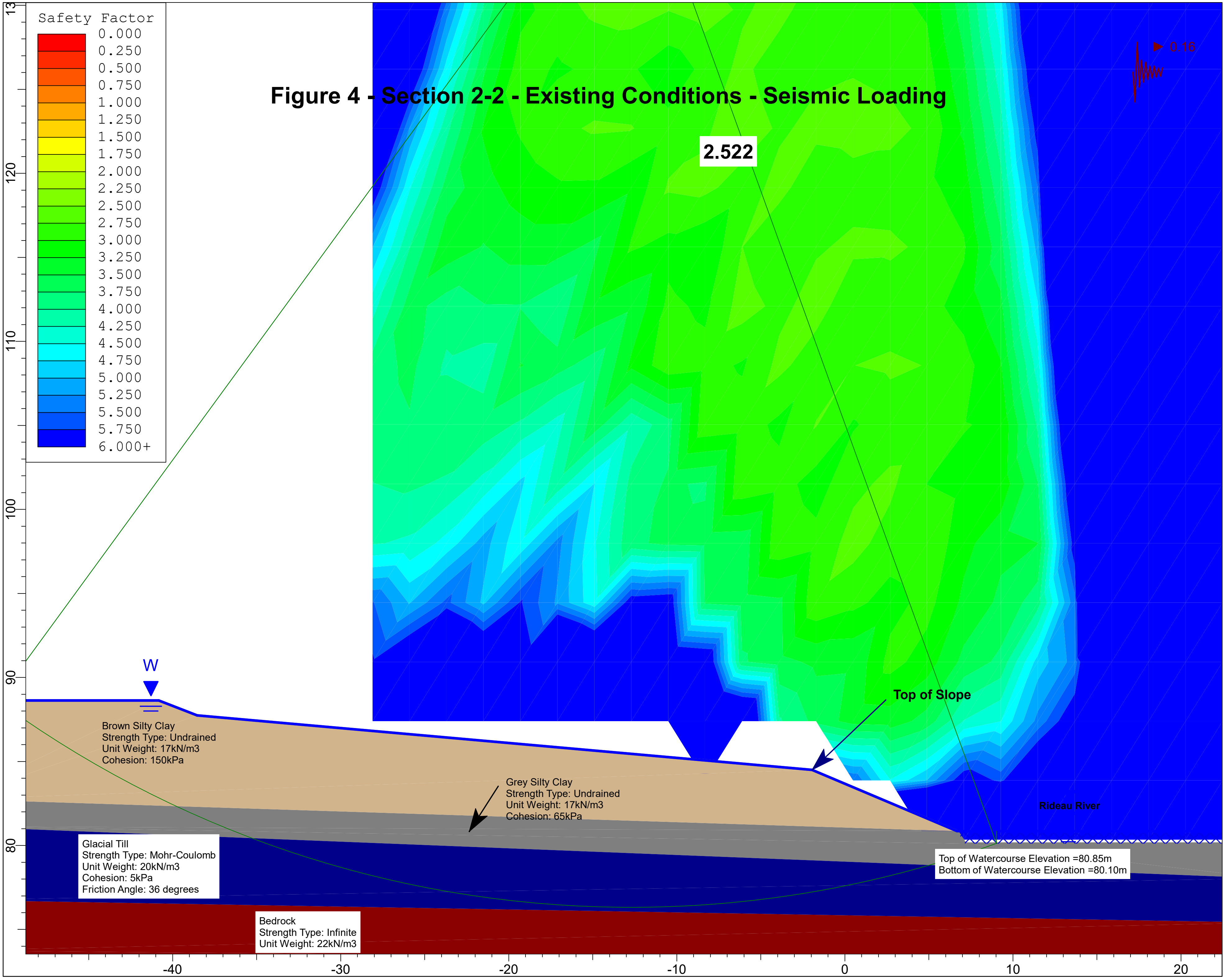


Figure 4 - Section 2-2 - Existing Conditions - Seismic Loading

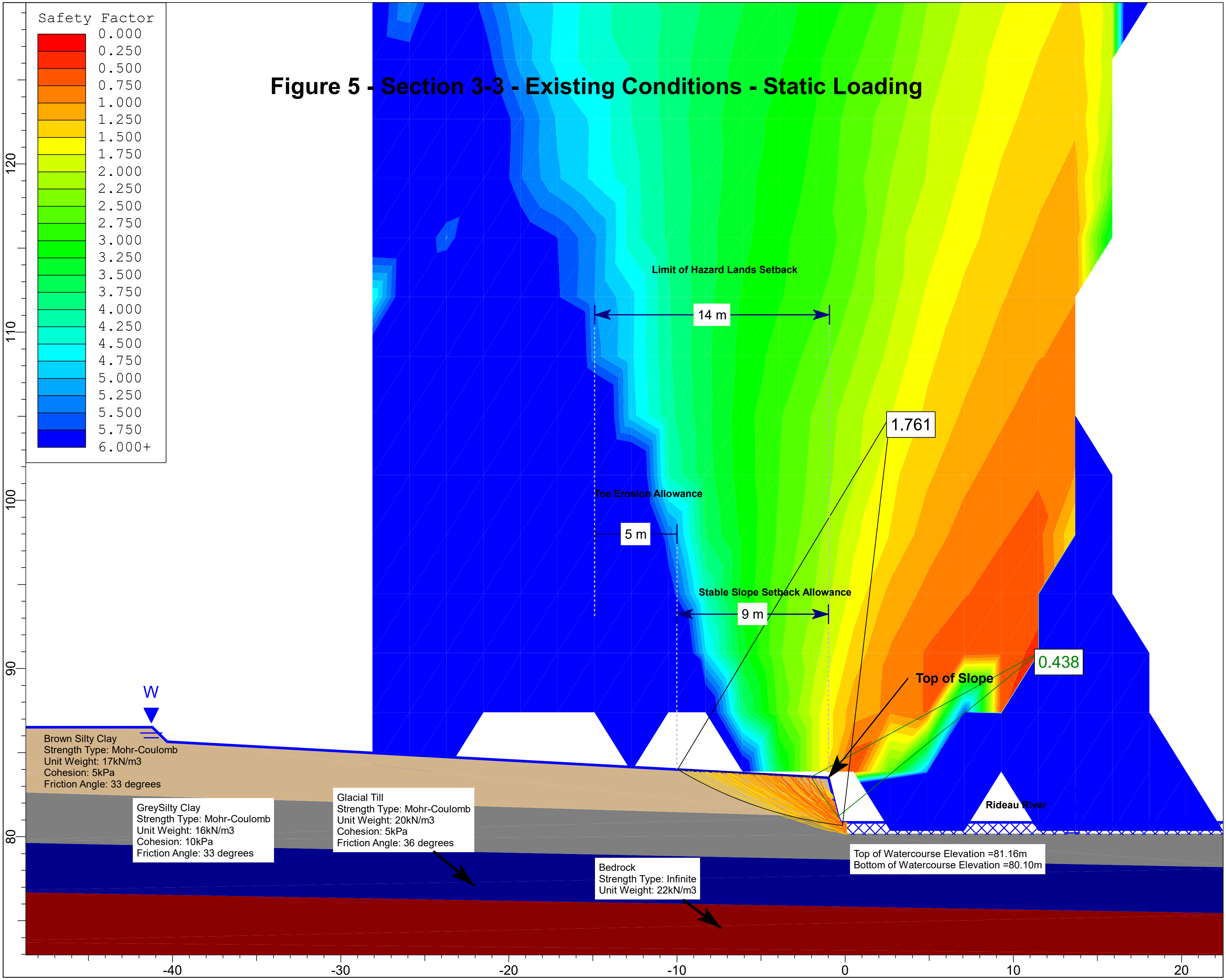
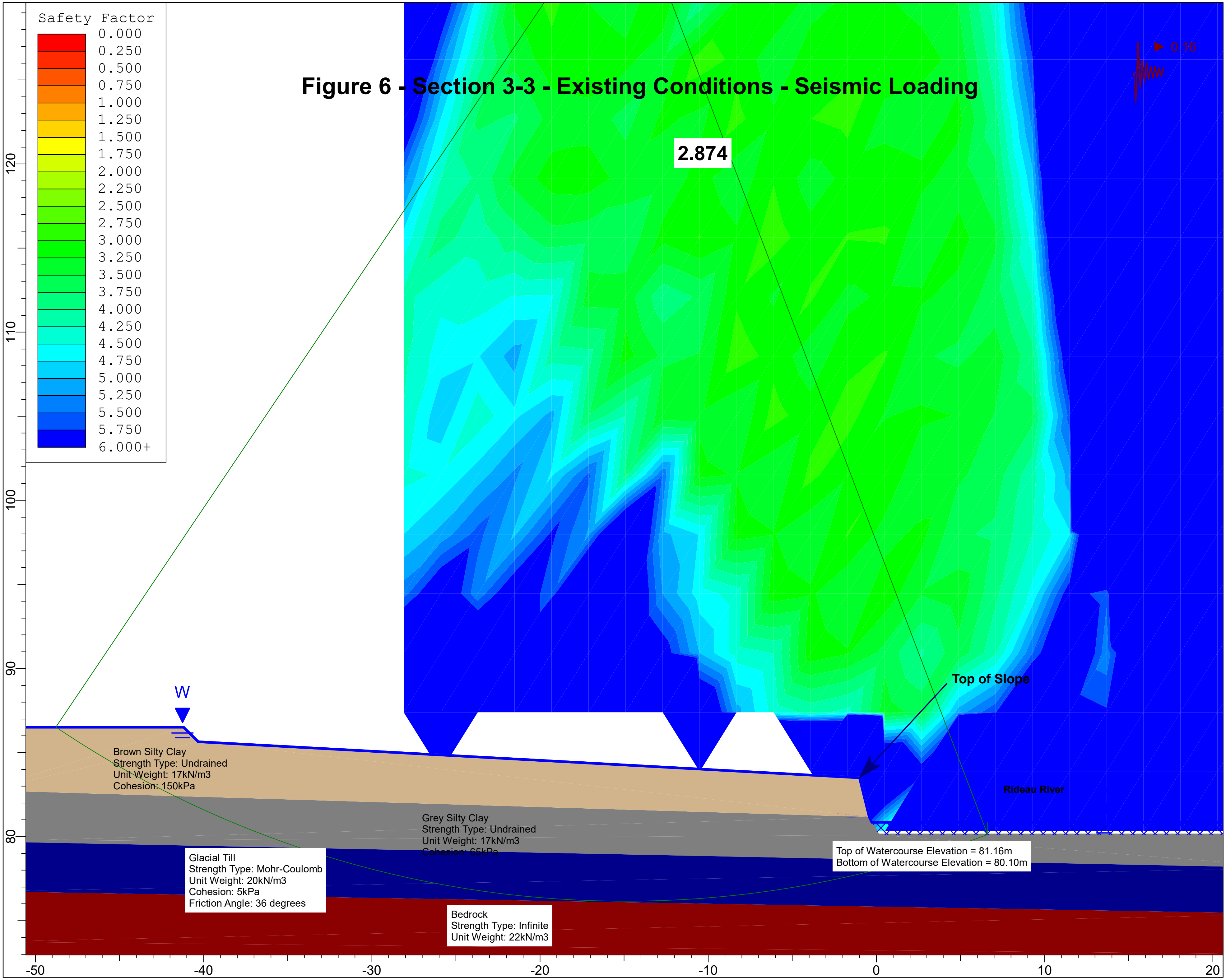
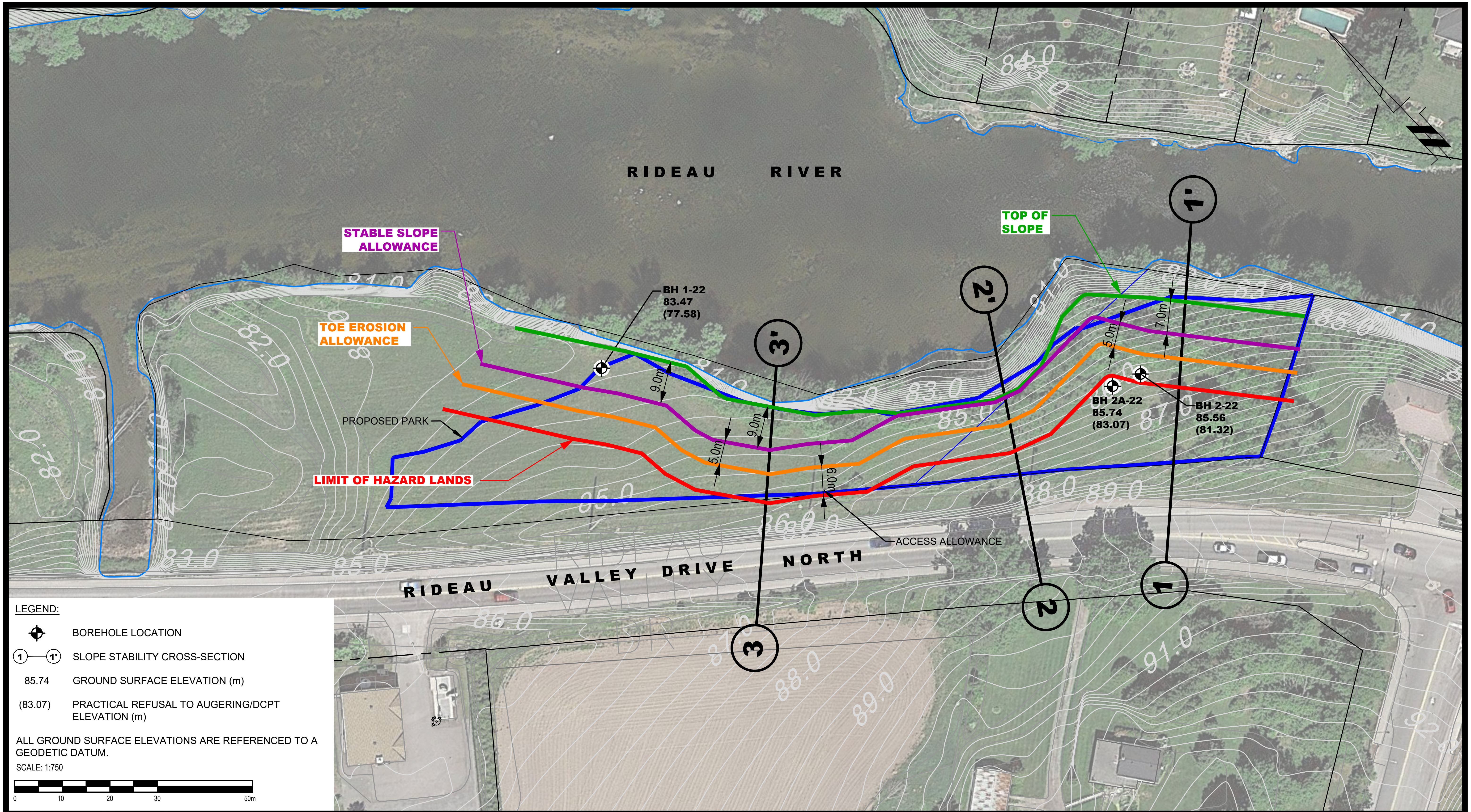


Figure 5 - Section 3-3 - Existing Conditions - Static Loading

Figure 6 - Section 3-3 - Existing Conditions - Seismic Loading





LEGEND:

⌖ BOREHOLE LOCATION

①——① ' SLOPE STABILITY CROSS-SECTION

85.74 GROUND SURFACE ELEVATION (m)

(83.07) PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS
SLOPE STABILITY ANALYSIS
PROPOSED PARK
RIDEAU VALLEY DRIVE
MANOTICK, ONTARIO

Title:

LIMIT OF HAZARD LANDS PLAN

Scale: 1:750
Date: 06/2022
Drawn by: NFRV
Report No.: PG5828-LET.01
Checked by: MS
Dwg. No.: PG5828-2
Approved by: DJG
Revision No.:

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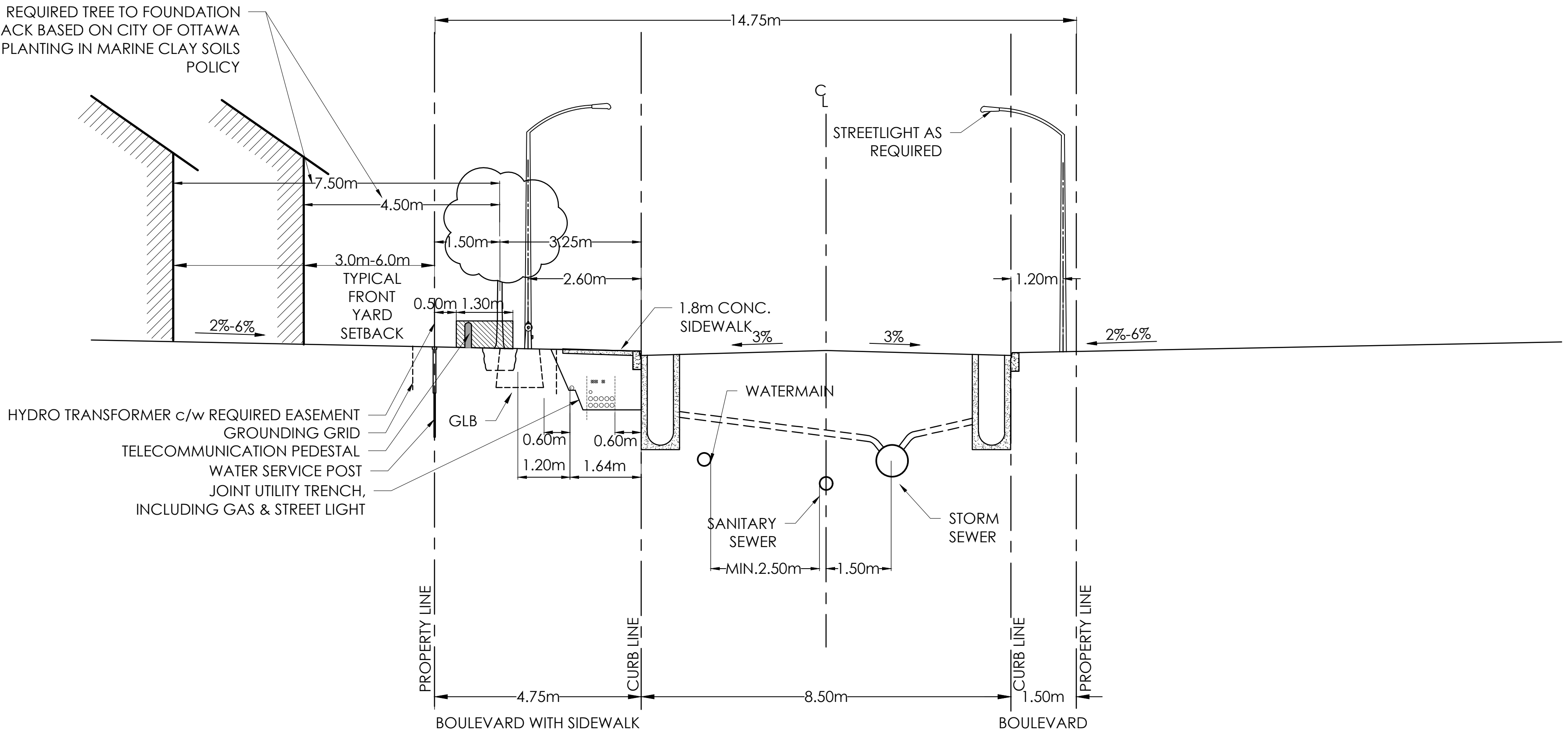
11x17

Appendix G
Pre-vetted City of Ottawa Cross-sections

1. STANDARD CROSS-SECTIONS TO BE READ IN CONJUNCTION WITH THE GENERAL STANDARD CROSS-SECTION NOTES AND OTHER APPLICABLE CITY AND UTILITY PLANS AND DETAILS.
2. 14.75M RIGHT-OF-WAY NOT TO BE USED ON STREETS WITH BUS SERVICE.
3. CONCRETE CURBS TO BE CONSTRUCTED AS PER CITY OF OTTAWA STANDARD DETAILS.
4. TYPICAL FRONT YARD SETBACKS ARE TO BE CLEAR AND UNENCUMBERED OF ANY SUBSURFACE BUILDING ENCROACHMENTS.
5. FIRE HYDRANTS SHALL BE LOCATED ON THE WATERMAIN SIDE OF THE STREET.
6. CATCH BASINS TO BE PER CITY OF OTTAWA DETAIL S2.
7. STREETLIGHTS MAY BE LOCATED ON EITHER SIDE OF THE RIGHT-OF-WAY.
8. GAS MAIN SHALL HAVE A MINIMUM OF 0.6 M CLEARANCE FROM STRUCTURES (E.G. CATCH BASINS AND HYDRANTS) AND 1.2 M FROM TREE ROOT BALL.
9. JOINT-USE UTILITY TRENCH (JUT) UNDER SIDEWALK AS PER DETAIL UDS0049. HELD BY HYDRO OTTAWA.
10. GRADE LEVEL BOX (GLB) AS DRAWN SHOWS GLB3660. EXACT LOCATION TO BE CONFIRMED.
11. THIS CROSS-SECTION CANNOT BE USED WHERE A CONCRETE ENCASED HYDROELECTRIC DUCT OR ANOTHER SEPARATE UTILITY DUCT IS REQUIRED.
12. TREE CLEARANCES TO HYDRO OTTAWA PLANT SHALL FOLLOW GCS0038.
13. CLEARANCES SHOWN ARE MINIMUMS.

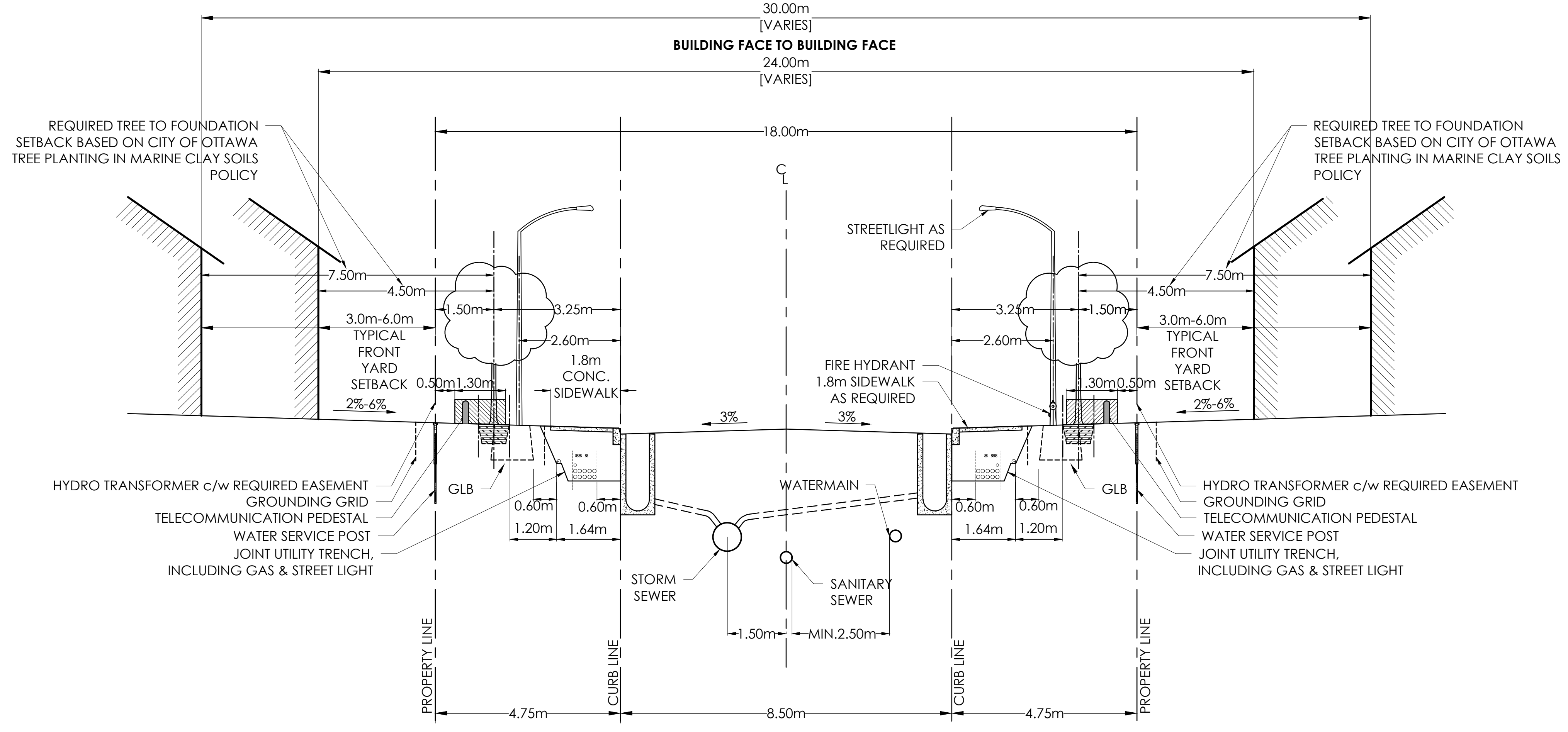


14.75m ROW CROSS SECTION

REV.DATE: AUG. 2022

DWG. No. ROW-14.75

1. STANDARD CROSS-SECTIONS TO BE READ IN CONJUNCTION WITH THE GENERAL STANDARD CROSS-SECTION NOTES AND OTHER APPLICABLE CITY AND UTILITY PLANS AND DETAILS.
2. 18M RIGHT-OF-WAY NOT TO BE USED ON STREETS WITH BUS SERVICE.
3. CONCRETE CURBS TO BE CONSTRUCTED AS PER CITY OF OTTAWA STANDARD DETAILS.
4. TYPICAL FRONT YARD SETBACK IS TO BE CLEAR AND UNENCUMBERED OF ANY SUBSURFACE BUILDING ENCROACHMENTS.
5. FIRE HYDRANTS TO BE LOCATED ON THE WATERMAIN SIDE OF THE STREET.
6. CATCH BASINS TO BE PER CITY OF OTTAWA DETAIL S2.
7. GAS MAIN SHALL HAVE A MINIMUM OF 0.6M CLEARANCE FROM STRUCTURES E.G. CATCH BASINS AND HYDRANTS) AND 1.2 M FROM TREE ROOT BALL.
8. STREETLIGHTS CAN BE LOCATED ON EITHER SIDE OF THE RIGHT-OF-WAY.
9. JOINT-USE UTILITY TRENCH (JUT) UNDER SIDEWALK AS PER DETAIL UDS0049. HELD BY HYDRO OTTAWA.
10. GRADE LEVEL BOX (GLB) AS DRAWN SHOWS GLB3660. EXACT LOCATION TO BE CONFIRMED.
11. THIS CROSS-SECTION CANNOT BE USED WHERE A CONCRETE ENCASED HYDROELECTRIC DUCT OR ANOTHER SEPARATE UTILITY DUCT IS REQUIRED.
12. TREE CLEARANCES TO HYDRO OTTAWA PLANT SHALL FOLLOW GCS0038.
13. CLEARANCES SHOWN ARE MINIMUMS.



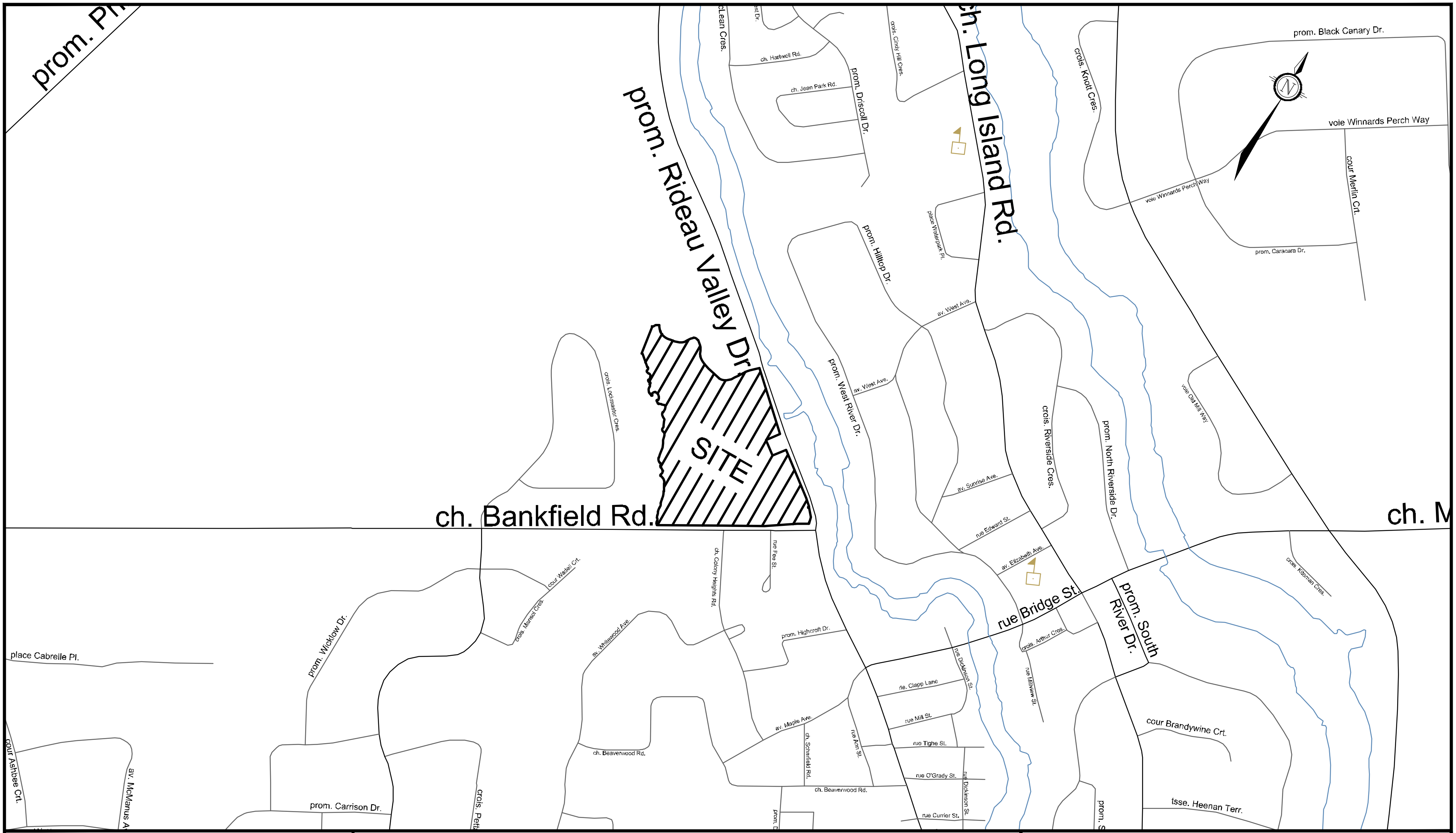
Ottawa

18.0m ROW CROSS SECTION

REV.DATE: AUG. 2022

DWG. No. ROW-18.0

Appendix H
Figures



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

NOVATECH

Engineers, Planners & Landscape Architects

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

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

CITY OF OTTAWA
STINSON LANDS
4386 RIVER ROAD

KEY PLAN

SCALE
NOT TO SCALE

DATE	JOB	FIGURE
MAR 2025	121153	1.1

SHT8X11.DWG - 216mmx279mm

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Ottawa, Ontario, Canada K2M 1P6

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

LEGEND

━ DEVELOPMENT LIMIT
━ BOUNDARY
─ WATERCOURSE

CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

EXISTING CONDITIONS

SCALE NOT TO SCALE

DATE	JOB	FIGURE
MAR 2025	121153	1.2

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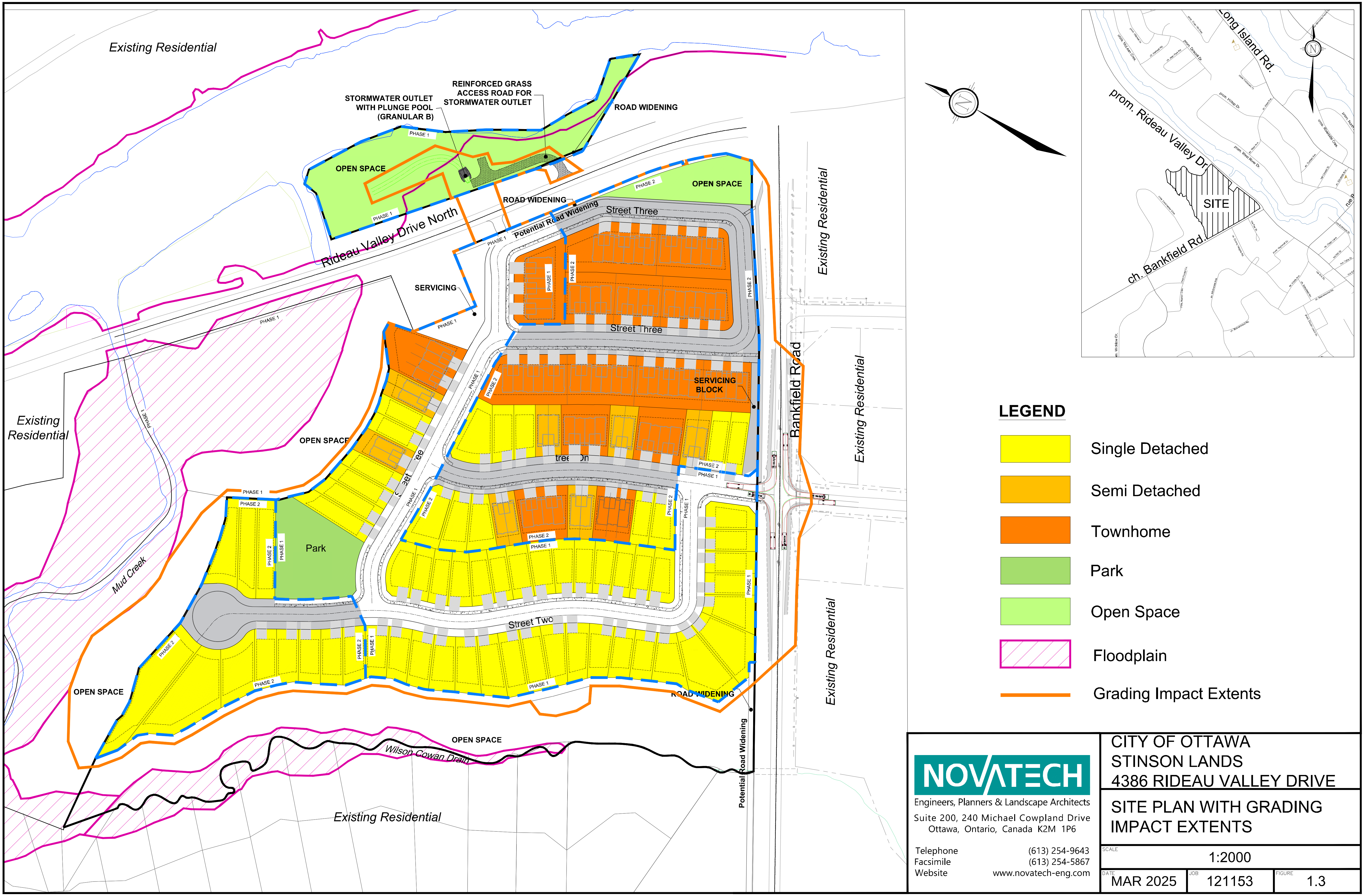

M:\2021\121153\CAD\Design\121153-BP.dwg, FIG 1.3 Site Plan, Dec 06, 2024 - 3:58pm, pkanani

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Ottawa, Ontario, Canada K2M 1P6

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

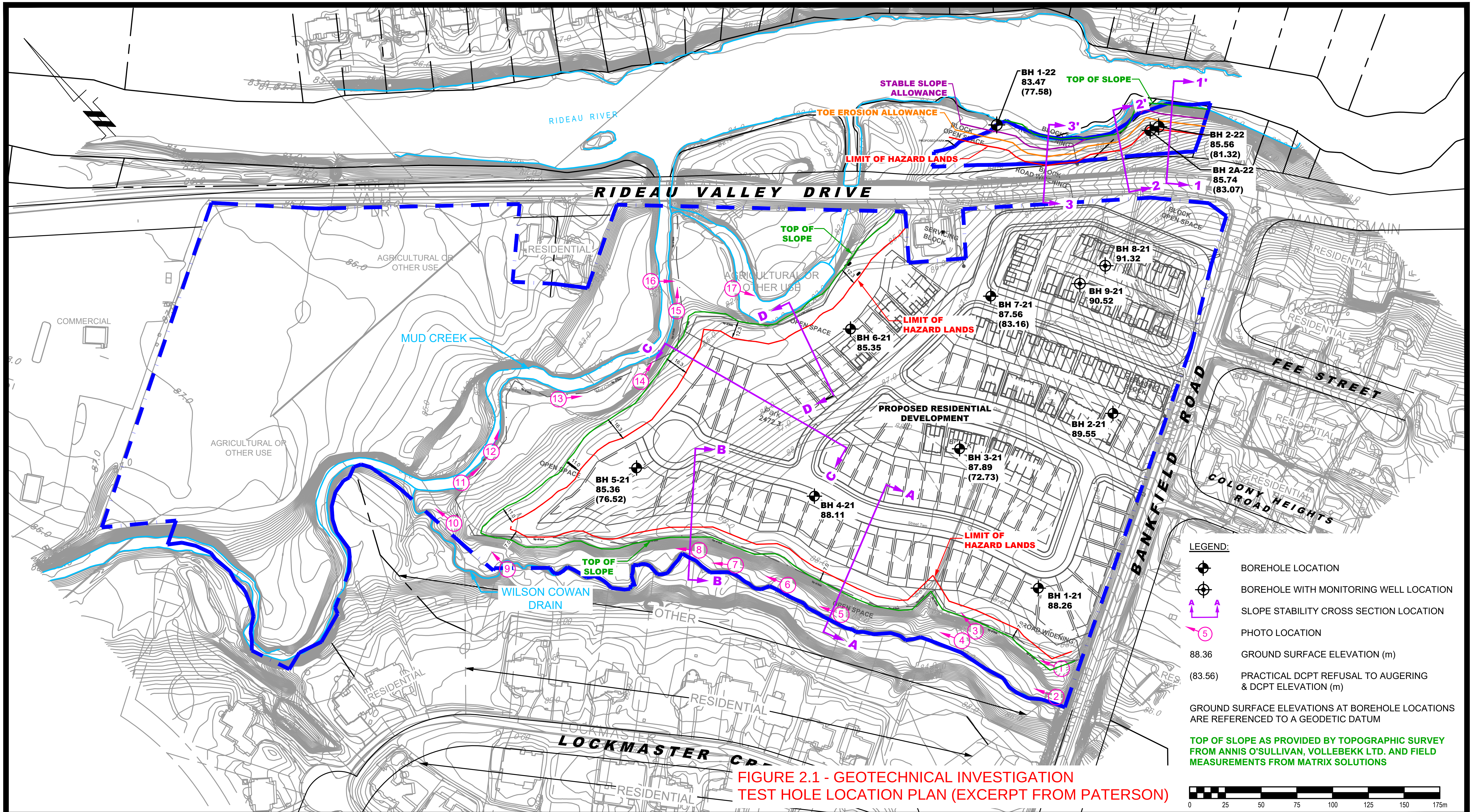
CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

SITE PLAN WITH GRADING
IMPACT EXTENTS

SCALE 1:2000

DATE	JOB	FIGURE
MAR 2025	121153	1.3

SHT11X17.DWG - 279mmX432mm



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

FIGURE 2.1 - GEOTECHNICAL INVESTIGATION TEST HOLE LOCATION PLAN (EXCERPT FROM PATERSON)

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

NO.	REVISIONS	DATE	INITIAL
4	SITE PLAN UPDATED	04/04/2024	SK
3	WATER COURSES NAME UPDATED	20/10/2023	MS
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: KP
Dwg No.: PG5828-1
Approved by: DJG
Revision No.: 4

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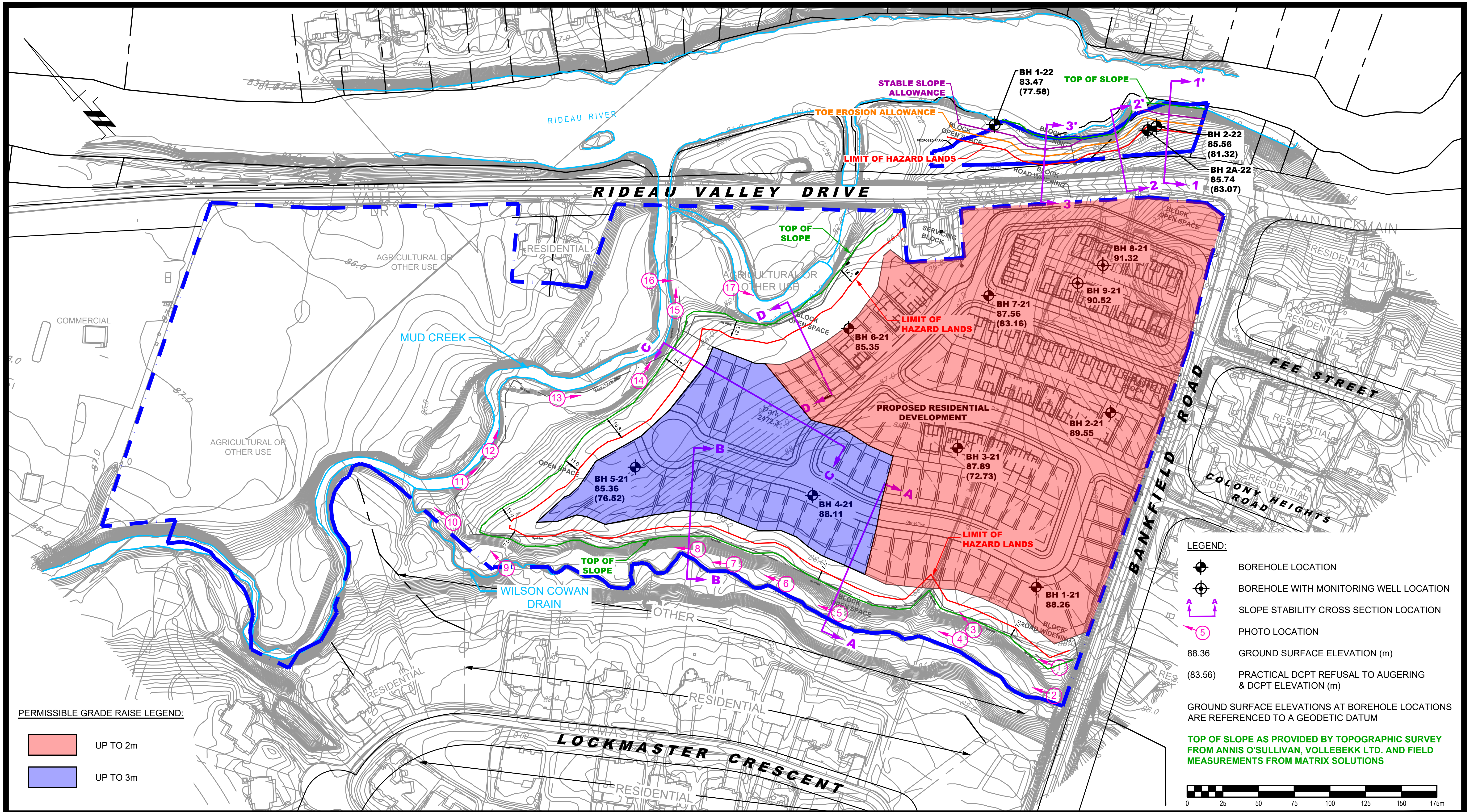

PERMISSIBLE GRADE RAISE LEGEND:

🟥	UP TO 2m
🟦	UP TO 3m

LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
A A	SLOPE STABILITY CROSS SECTION LOCATION
⑤	PHOTO LOCATION
88.36	GROUND SURFACE ELEVATION (m)
(83.56)	PRACTICAL DCPT REFUSAL TO AUGERING & DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

TOP OF SLOPE AS PROVIDED BY TOPOGRAPHIC SURVEY FROM ANNIS O'SULLIVAN, VOLLEBEKK LTD. AND FIELD MEASUREMENTS FROM MATRIX SOLUTIONS

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

NO.	REVISIONS	DATE	INITIAL

UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:2500	Date: 04/2024
Drawn by: NFRV	Report No.: PG5828-1
Checked by: KP	Dwg No.: PG5828-3
Approved by: DJG	Revision No.:

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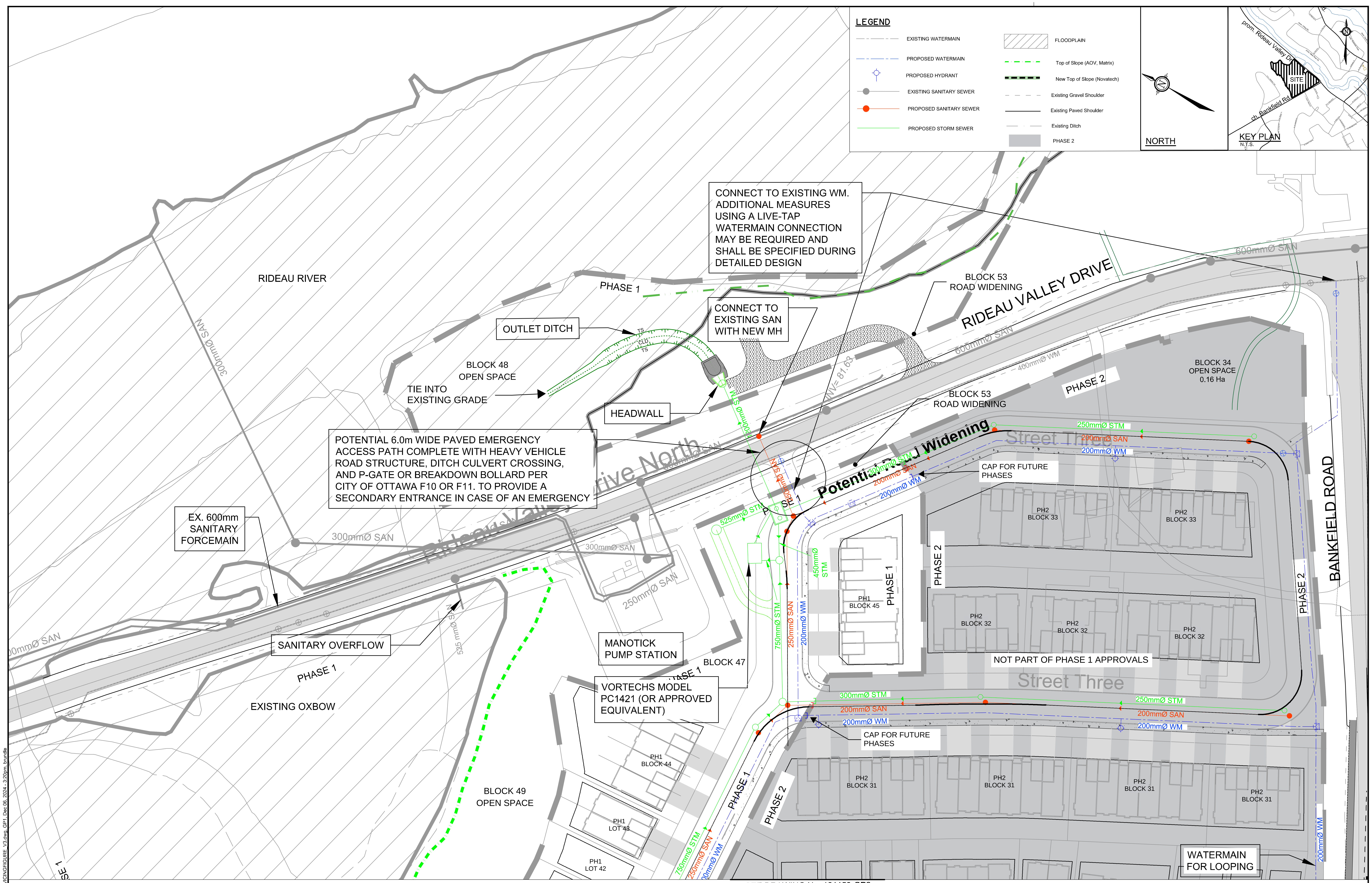


SEE DRAWING No. 121153-GP2

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.

No.	REVISION	DATE	BY
4.	ISSUED FOR RE-SUBMISSION REVIEW	MAR 3/25	BHB
3.	ISSUED FOR RE-SUBMISSION REVIEW	DEC 20/24	BHB
2.	ISSUED FOR RE-SUBMISSION REVIEW	APR 10/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE
1:500

1:500
0 5 10 15 20

DESIGN BPR
CHECKED BHB
DRAWN BPR
CHECKED BHB
APPROVED BHB

FOR REVIEW ONLY

NOVATECH
Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

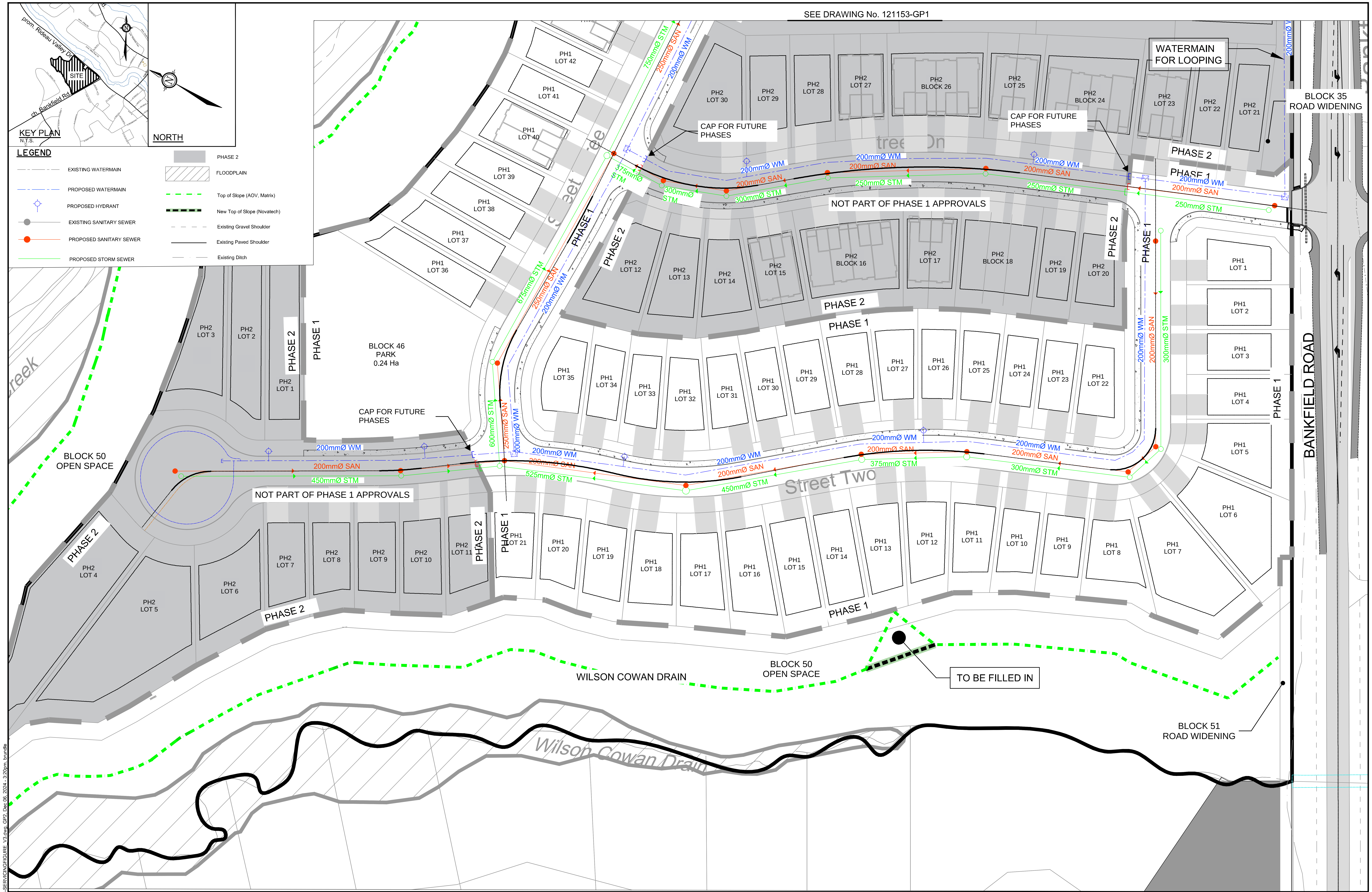
DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES 1

PROJECT No. 121153-00
REV REV #4
DRAWING No. 121153-GP1

PLANA1.DWG - 841mmx594mm

M:\2021\121153\CAD\Design\Figures\121153-SERVICINGFIGURE_V3.dwg, GP1, Dec 06, 2024 - 3:20pm, brundle

SEE DRAWING No. 121153-GP1



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
4.	ISSUED FOR RE-SUBMISSION REVIEW	MAR 3/25	BHB
3.	ISSUED FOR RE-SUBMISSION REVIEW	DEC 20/24	BHB
2.	ISSUED FOR RE-SUBMISSION REVIEW	APR 10/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE
1:500

1:500
0 5 10 15 20

DESIGN BPR
CHECKED BHB
DRAWN BPR
CHECKED BHB
APPROVED BHB

FOR REVIEW ONLY

NOVATECH
Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES 2

PROJECT No. 121153-00
REV REV #4
DRAWING No. 121153-GP2

PLANA1.DWG - 841mmx594mm

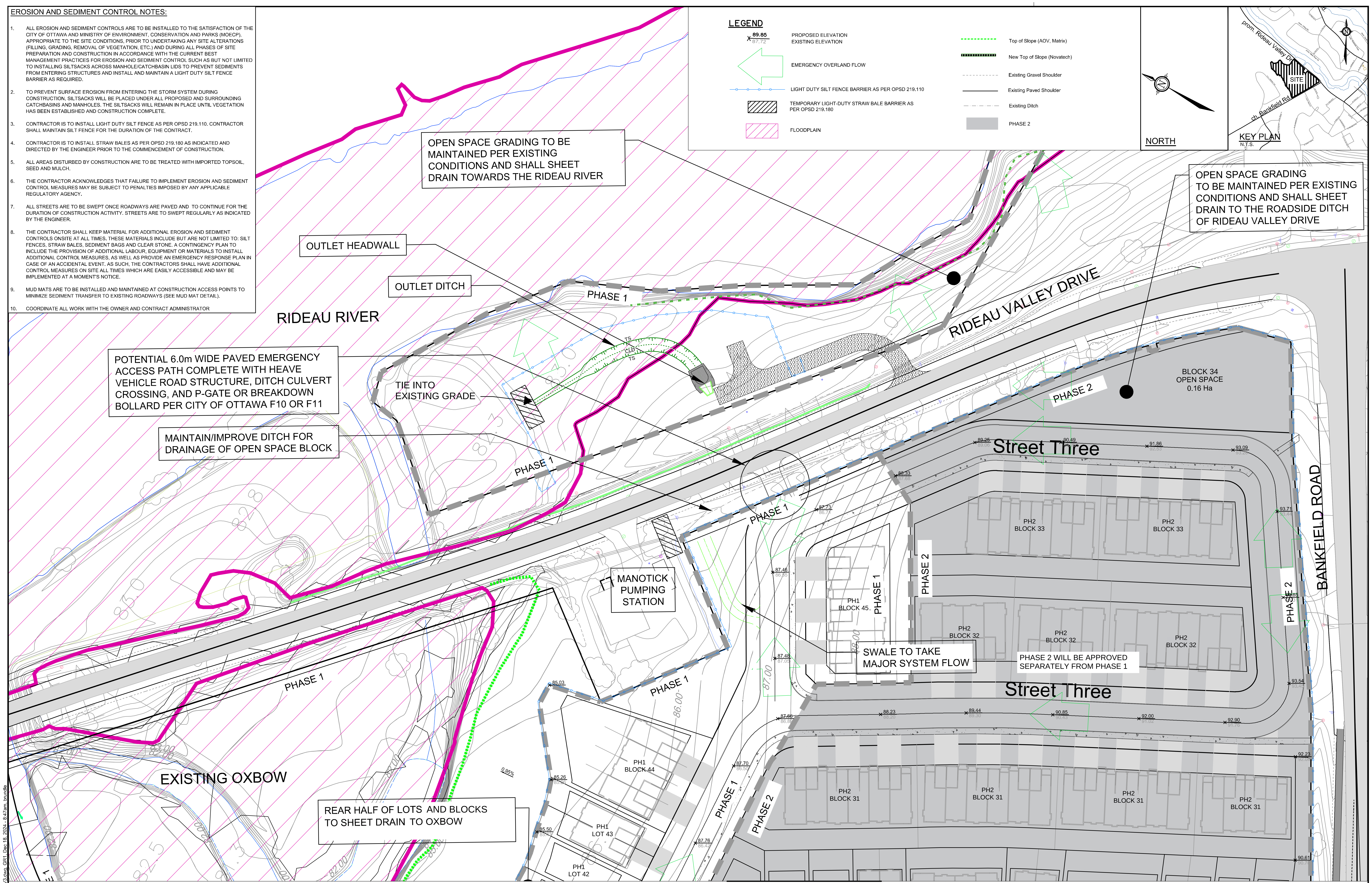
M:\2021\121153\CAD\Design\Figures\121153-SERVICINGFIGURE_V3.dwg, GP2, Dec 06, 2024 - 3:20pm, brundle

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
■	PHASE 2



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
5.	ISSUED FOR RE-SUBMISSION REVIEW	MAR 3/25	BHB
4.	ISSUED FOR RE-SUBMISSION REVIEW	DEC 13/24	BHB
3.	ISSUED FOR RE-SUBMISSION REVIEW	AUG 08/24	BHB
2.	ISSUED FOR RE-SUBMISSION REVIEW	APR 11/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE
1:500

1:500
0 5 10 15 20

DESIGN BPR
CHECKED BHB
DRAWN BPR
CHECKED BHB
APPROVED BHB

FOR REVIEW ONLY

NOVATECH
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Ottawa, Ontario, Canada K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GRADING, EROSION, & SEDIMENT CONTROL PLAN 1

PROJECT No. 121153-00
REV REV #5
DRAWING No. 121153-GR1

PLANA1.DWG - 841mmx594mm

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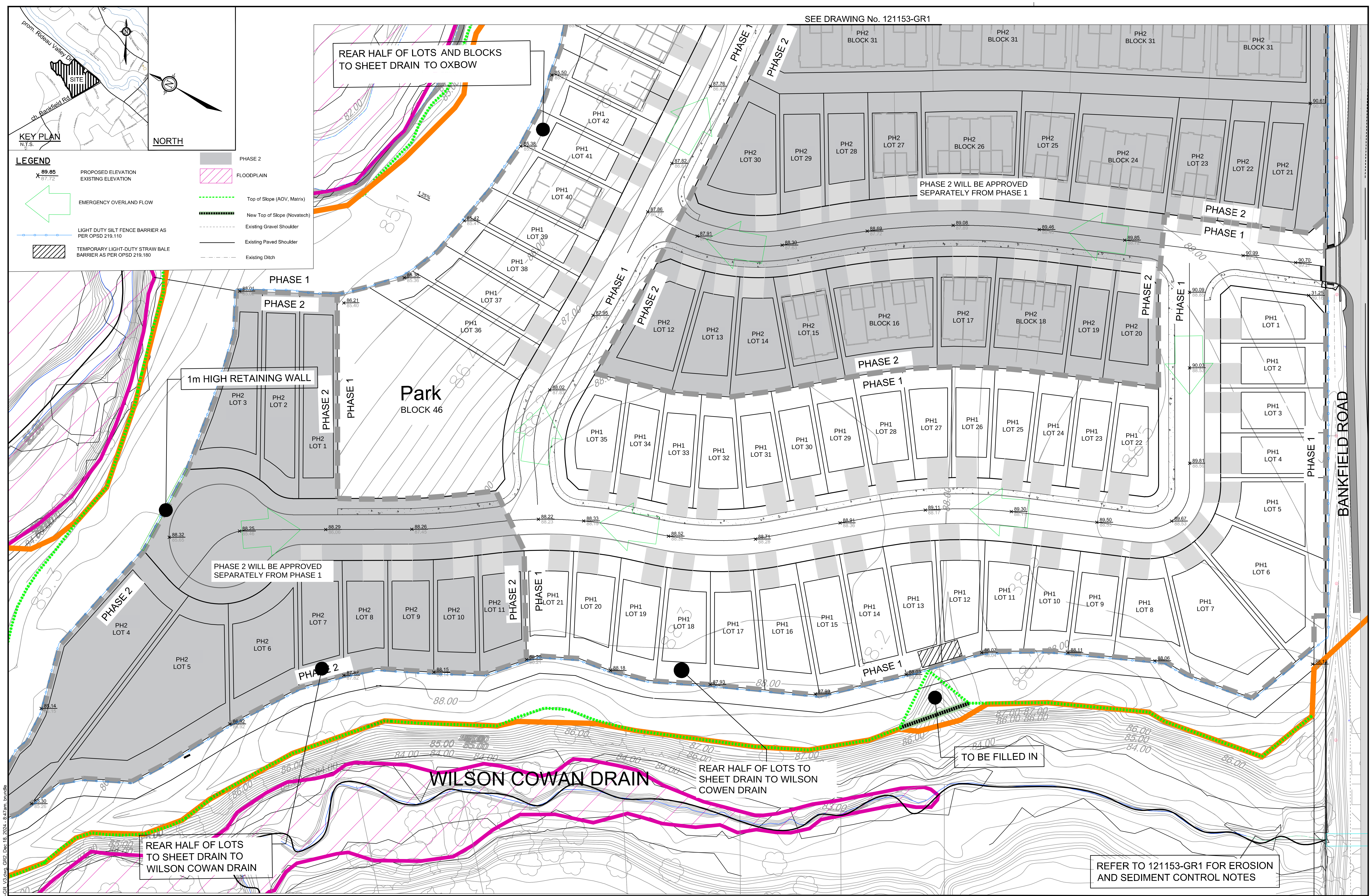

REAR HALF OF LOTS AND BLOCKS TO SHEET DRAIN TO OXBOW

PHASE 2 WILL BE APPROVED SEPARATELY FROM PHASE 1

PHASE 2 WILL BE APPROVED SEPARATELY FROM PHASE 1

REAR HALF OF LOTS TO SHEET DRAIN TO WILSON COWEN DRAIN

REAR HALF OF LOTS TO SHEET DRAIN TO WILSON COWAN DRAIN

REFER TO 121153-GR1 FOR EROSION AND SEDIMENT CONTROL NOTES

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
5.	ISSUED FOR RE-SUBMISSION REVIEW	MAR 3/25	BHB
4.	ISSUED FOR RE-SUBMISSION REVIEW	DEC 13/24	BHB
3.	ISSUED FOR RE-SUBMISSION REVIEW	OCT 23/24	BHB
2.	ISSUED FOR RE-SUBMISSION REVIEW	APR 11/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE
1:500

DESIGN BPR
CHECKED BHB
DRAWN BPR
CHECKED BHB
APPROVED BHB

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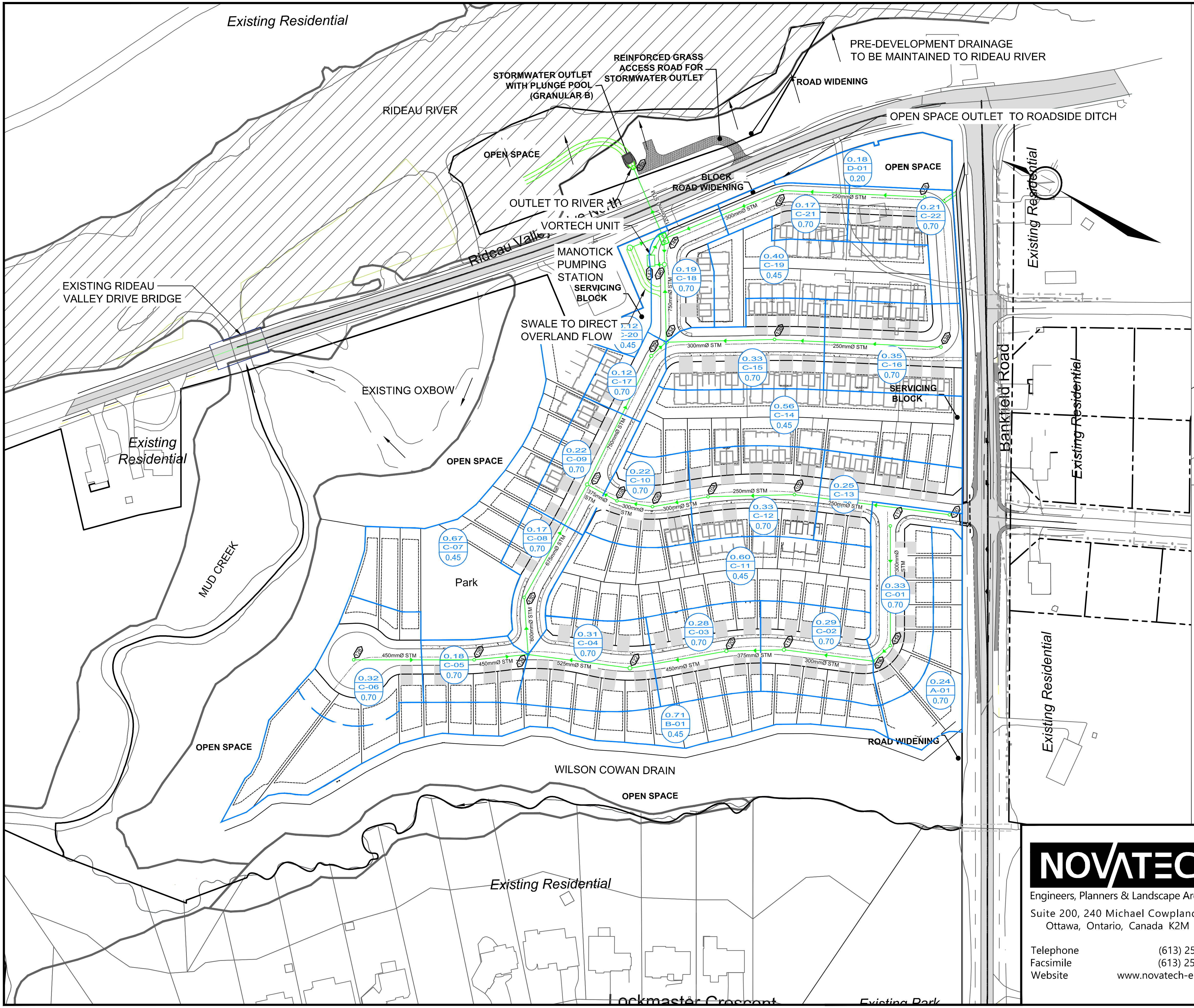
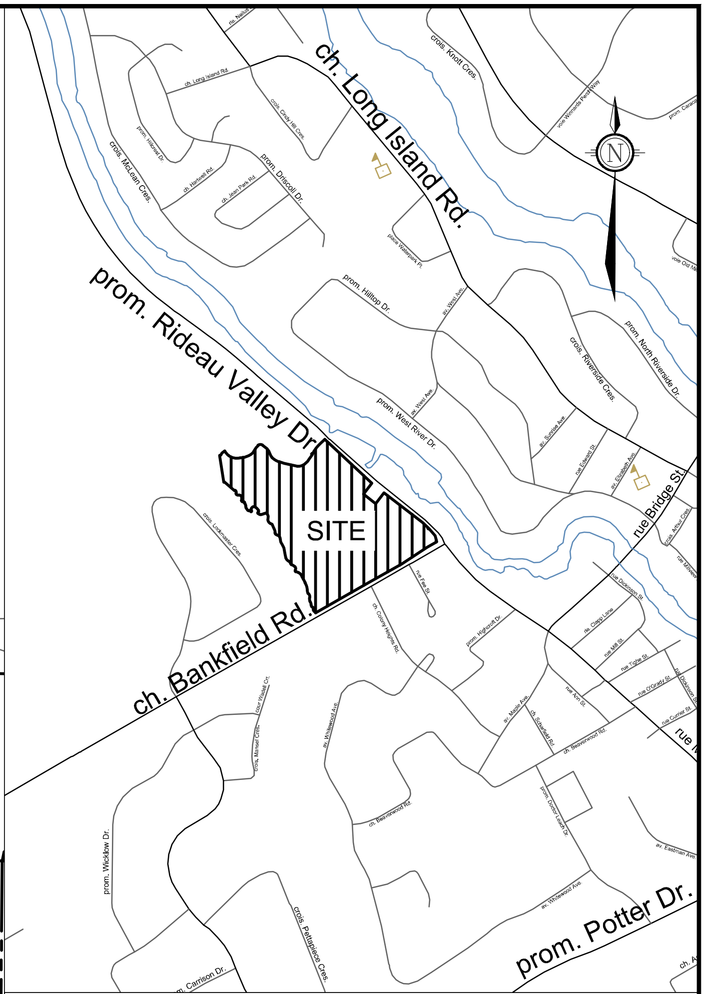
LOCATION
CITY OF OTTAWA - 4386 RIDEAU VALLEY DRIVE (N)
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GRADING, EROSION, & SEDIMENT CONTROL PLAN 2

PROJECT No. 121153-00
REV REV # 5
DRAWING No. 121153-GR2





LEGEND

0.34 C-15 0.70 DRAINAGE AREA (hectares)
AREA ID
RUN-OFF COEFFICIENT

STORM DRAINAGE AREA

EXISTING STORM SEWER AND MH

PROPOSED STORM SEWER AND MH

▨ FLOODPLAIN

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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAI VALLEY DRIVE

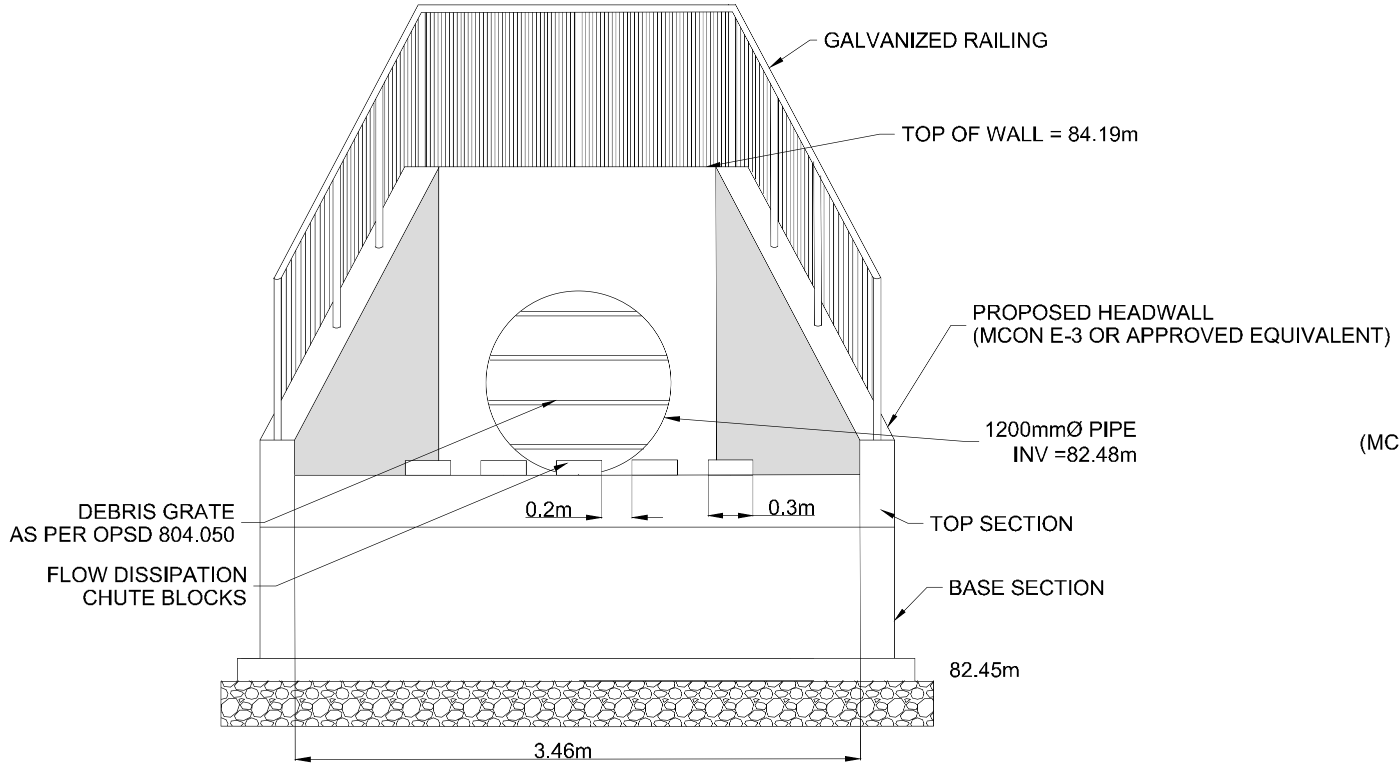
POST-DEVELOPMENT STORM DRAINAGE AREA PLAN

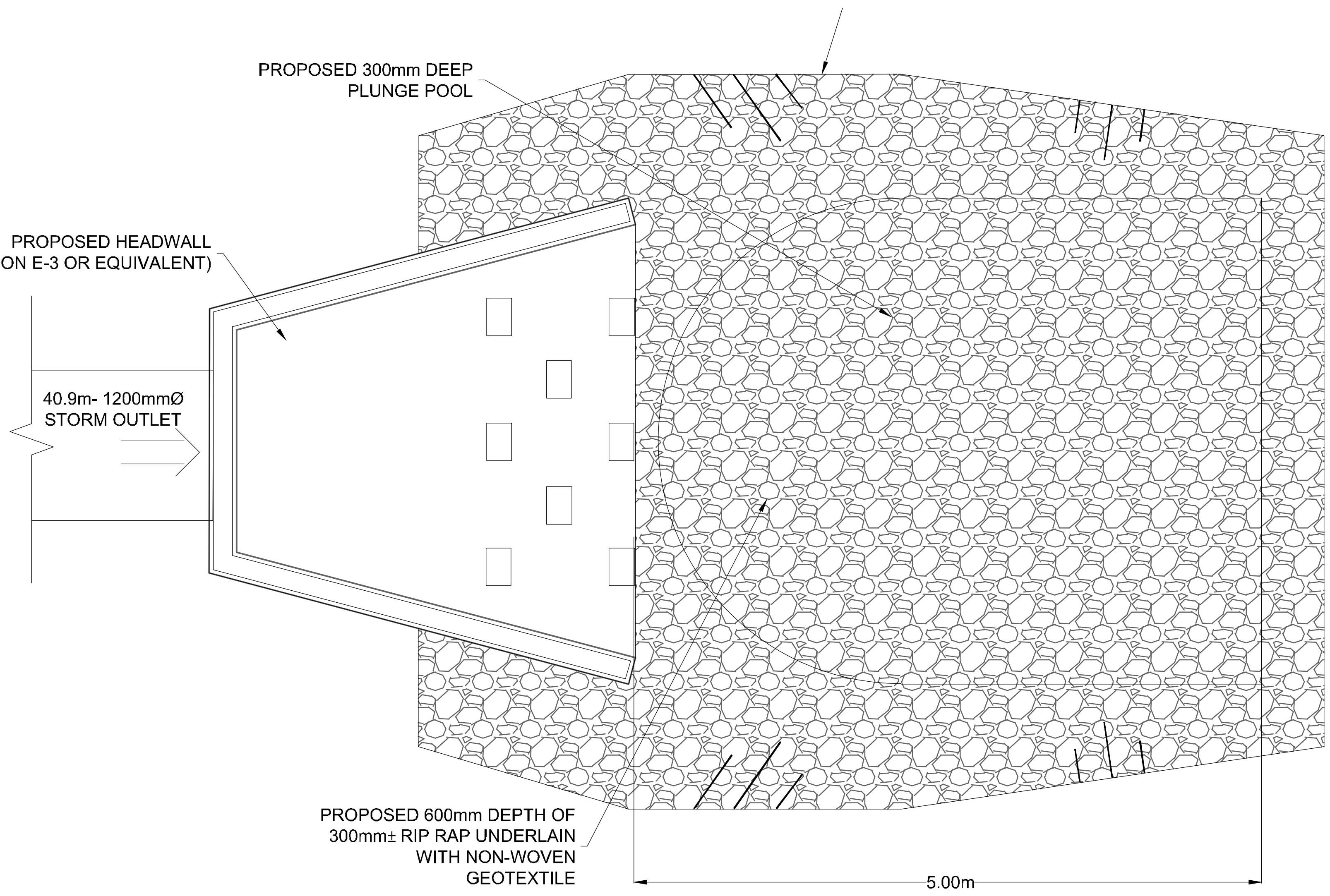
SCALE 1:2000

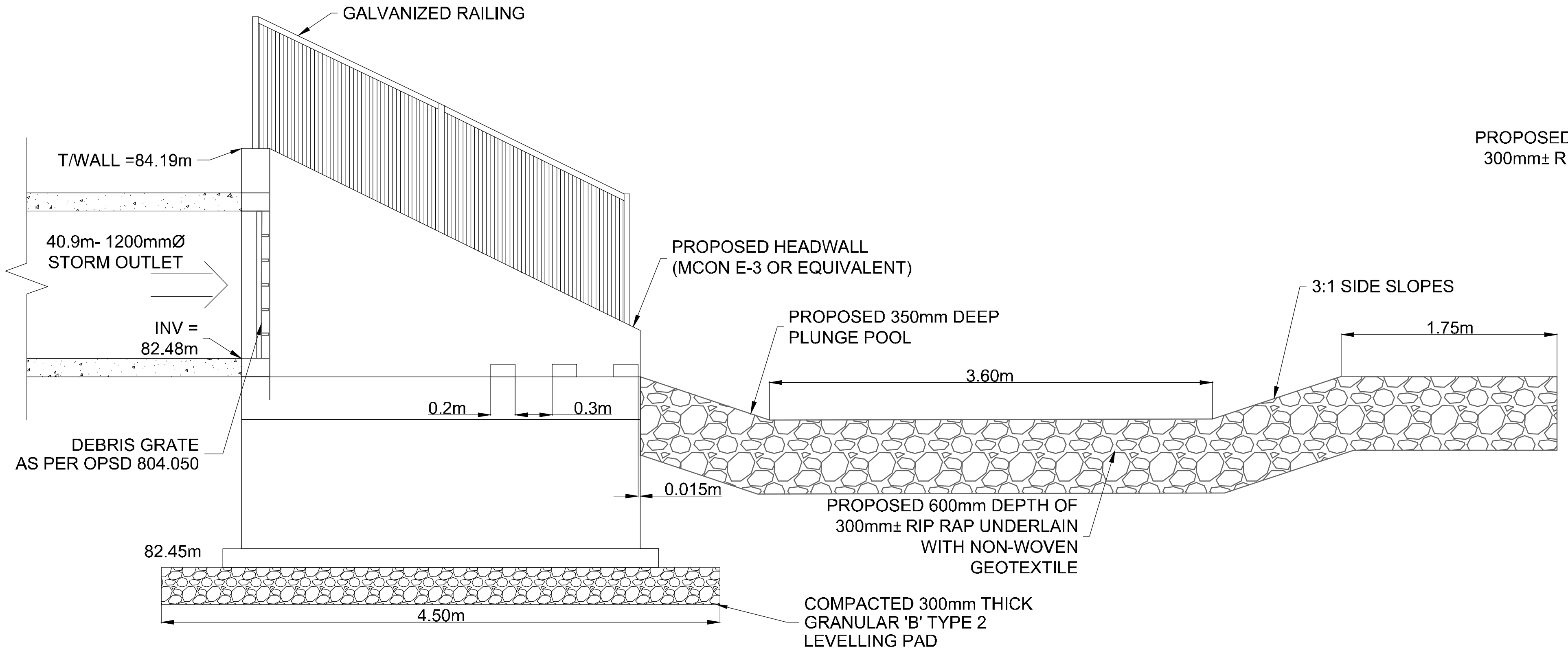
DATE	JOB	FIGURE
MAR 2025	121153	4.1

M:\2021\121153\CAD\Design\121153-STM.dwg, FIG 4.1, Dec 17, 2024 - 10:19am, brundle

SHT11X17.DWG - 279mmX432mm







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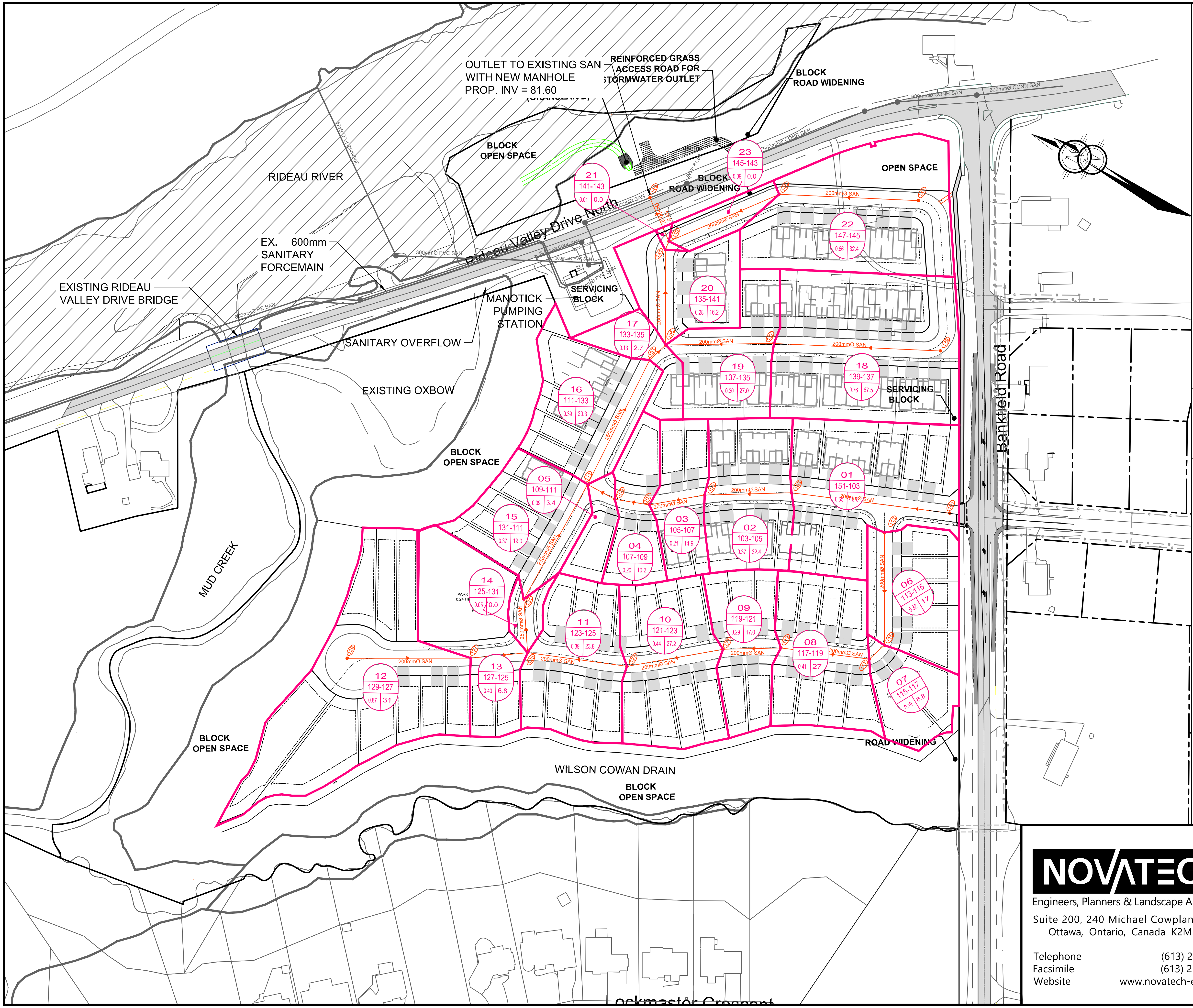
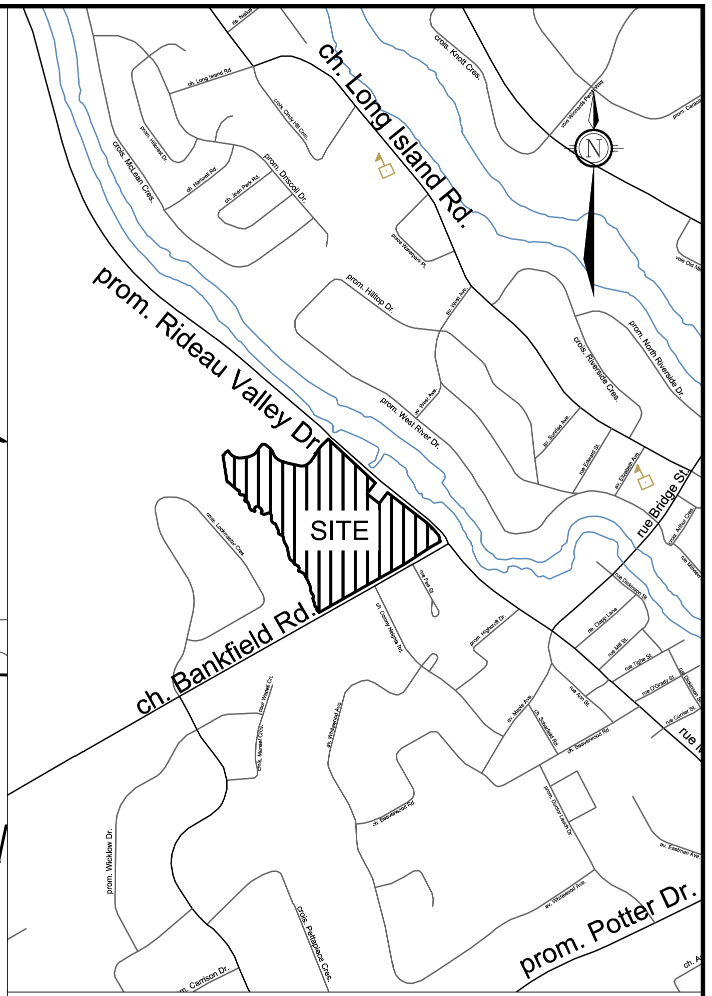
CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

PROPOSED STORM OUTLET
WITH PLUNGE POOL

SCALE
1:50

DATE	JOB	FIGURE
MAR 2025	121153	121153-OUTLET





LEGEND

01	AREA ID
101-103	MANHOLE TO MANHOLE
0.65	44.0	POPULATION EQUIVALENT / AREA IN HECTARES

- 0.65 — AREA IN HECTARES
- 44.0 — POPULATION EQUIVALENT

▬ SANITARY DRAINAGE AREA BOUNDARY

—●— *EXISTING SANITARY SEWER*

—●— PROPOSED SANITARY SEWER

▨ FLOODPLAIN

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CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

SANITARY DRAINAGE AREA PLAN

SCALE 1:2000

DATE	JOB	FIGURE
MAR 2025	121153	5.2

M:\2021\121153\CAD\Design\121153-SAN.dwg, FIG 5.2, Dec 17, 2024 - 10:14am, brundle

SHT11X17.DWG - 279mmX432mm

Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

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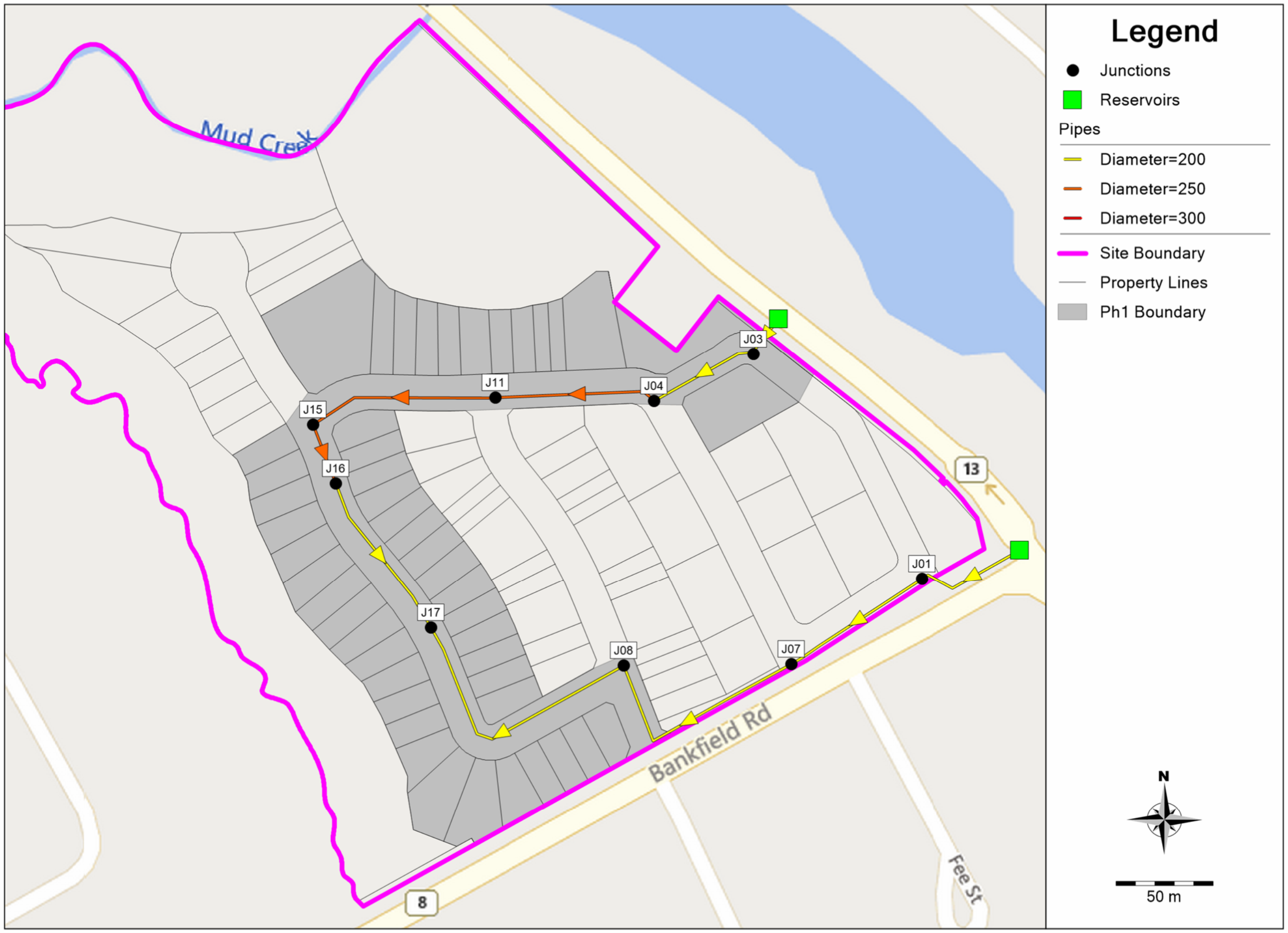

<u>Proposed Watermain Sizing, Layout and Junction IDs</u>

Date: 2024/04/08
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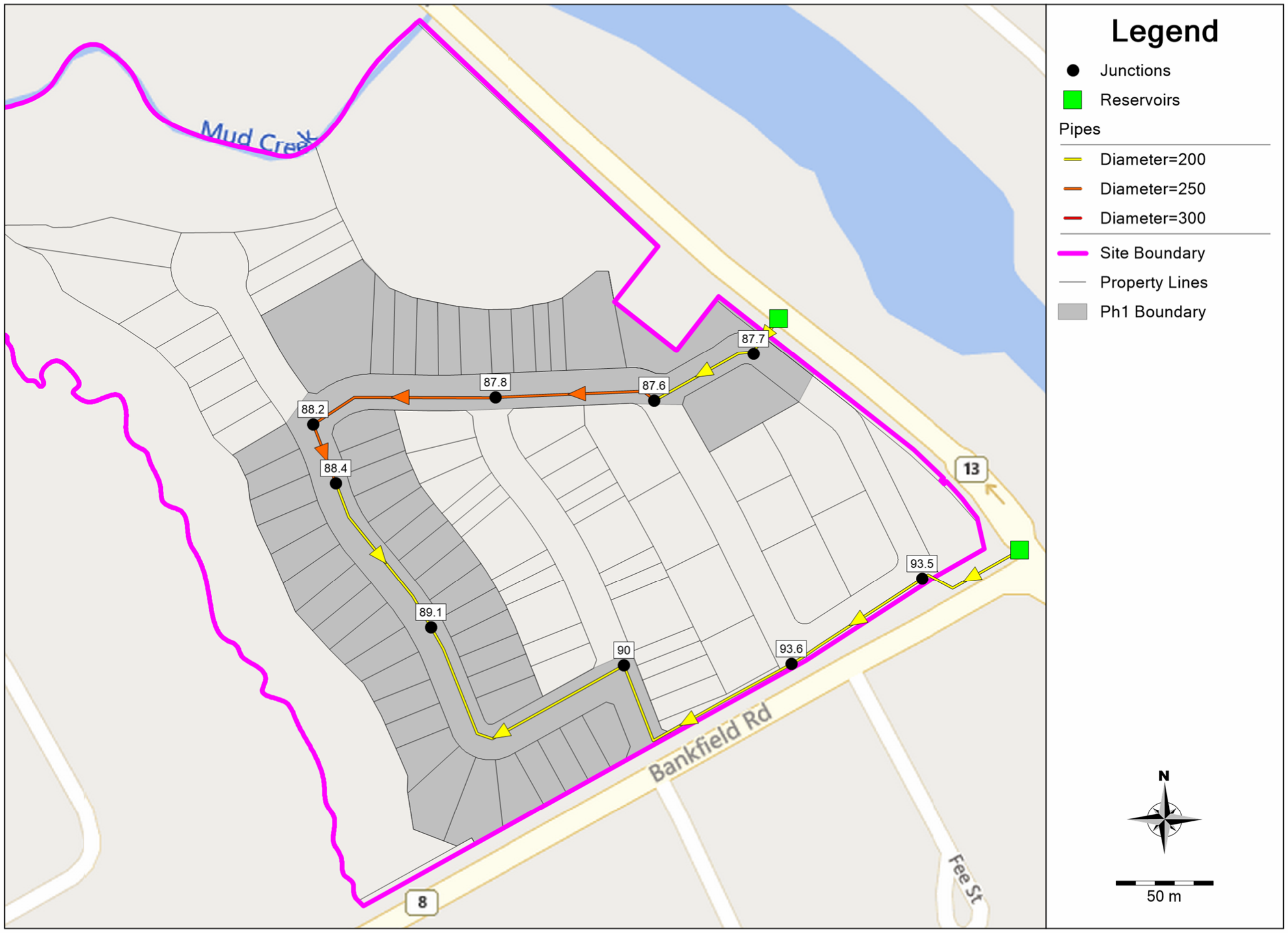
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



Ground Elevations (m)

Date: 2024/04/08
M:\2021\121153\DATA\Calculations\Sewer Calcs\Water_PCSWMM\Images\Ph1\Model Schematics.docx

Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

NOVATECH
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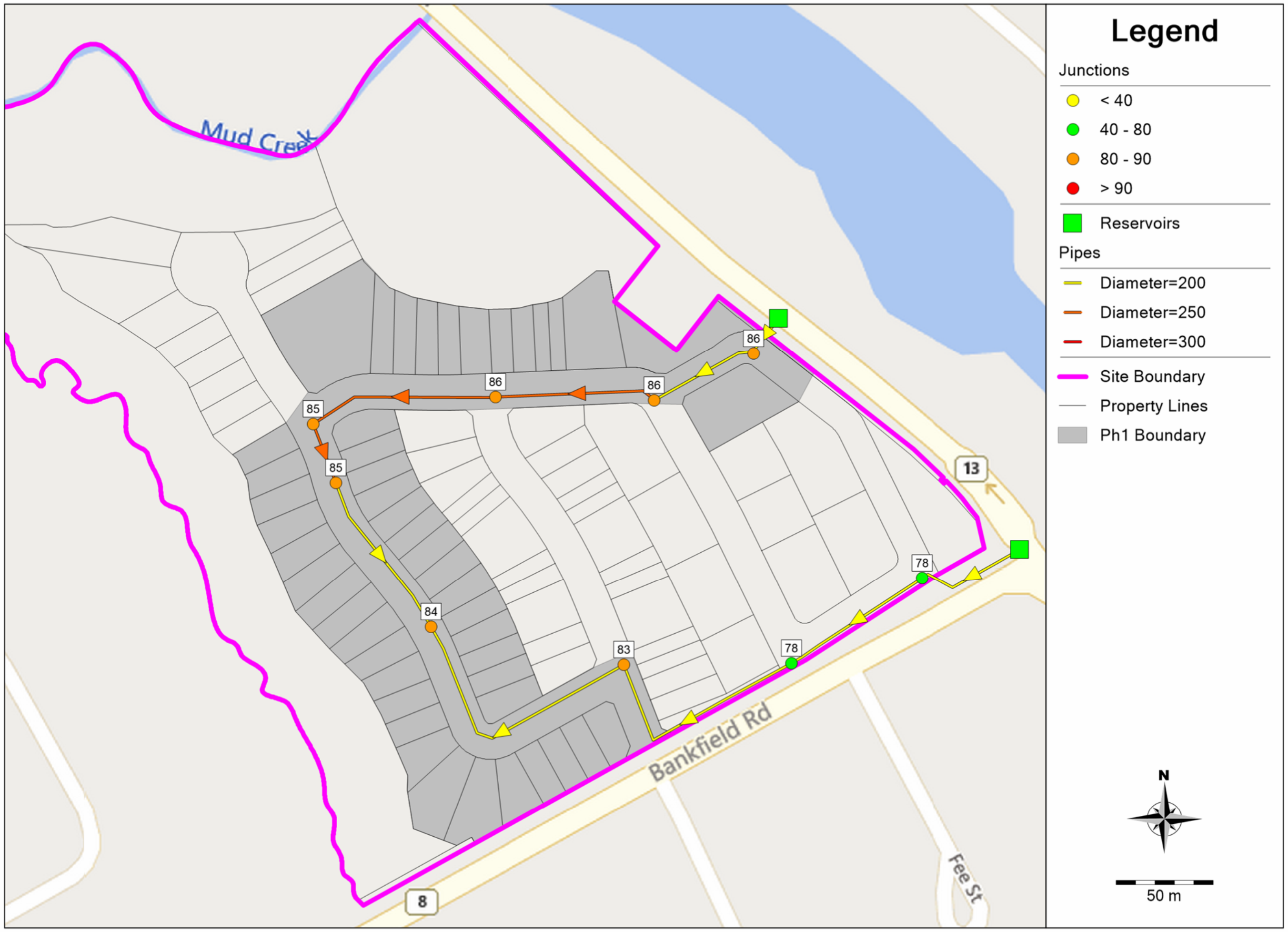

Maximum Pressure During AVDY Conditions – Future

Date: 2024/04/08
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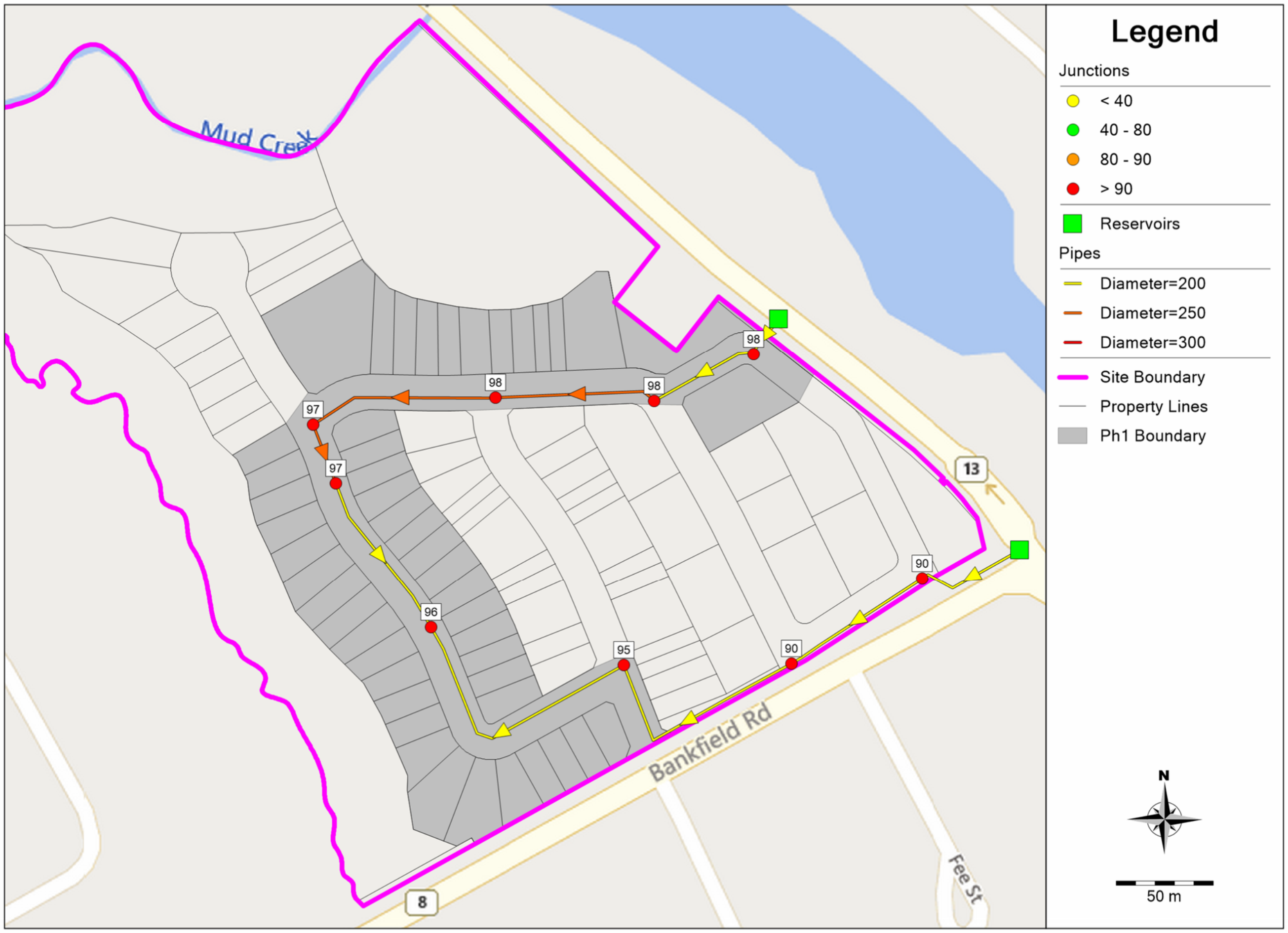
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



Maximum Pressure During AVDY Conditions – Existing

Date: 2024/04/08
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Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

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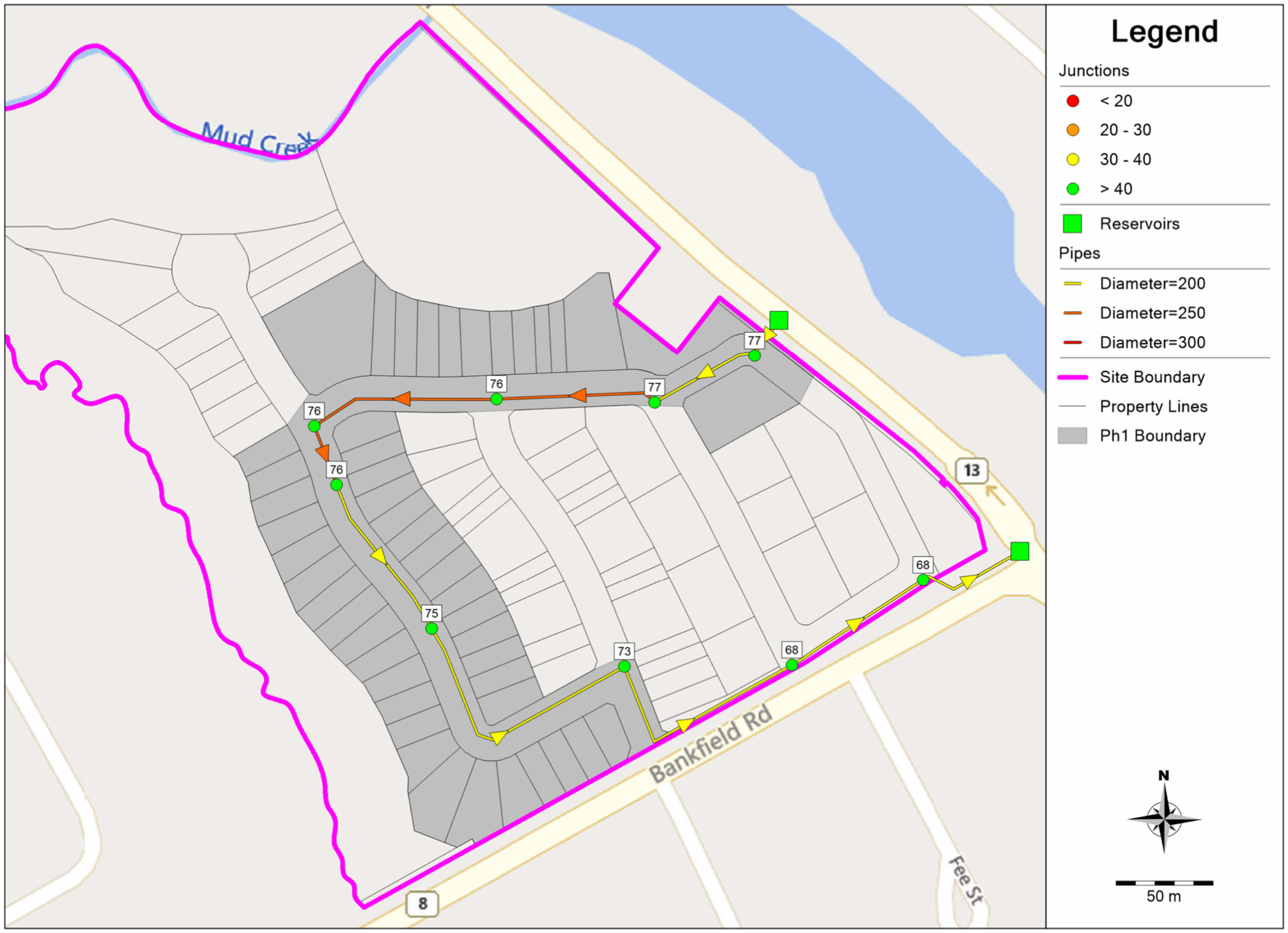

Minimum Pressure During PKHR Conditions – Future

Date: 2024/04/08
M:\2021\121153\DATA\Calculations\Sewer Calcs\Water_PCSWMM\Images\Ph1\Model Schematics.docx

Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

NOVATECH
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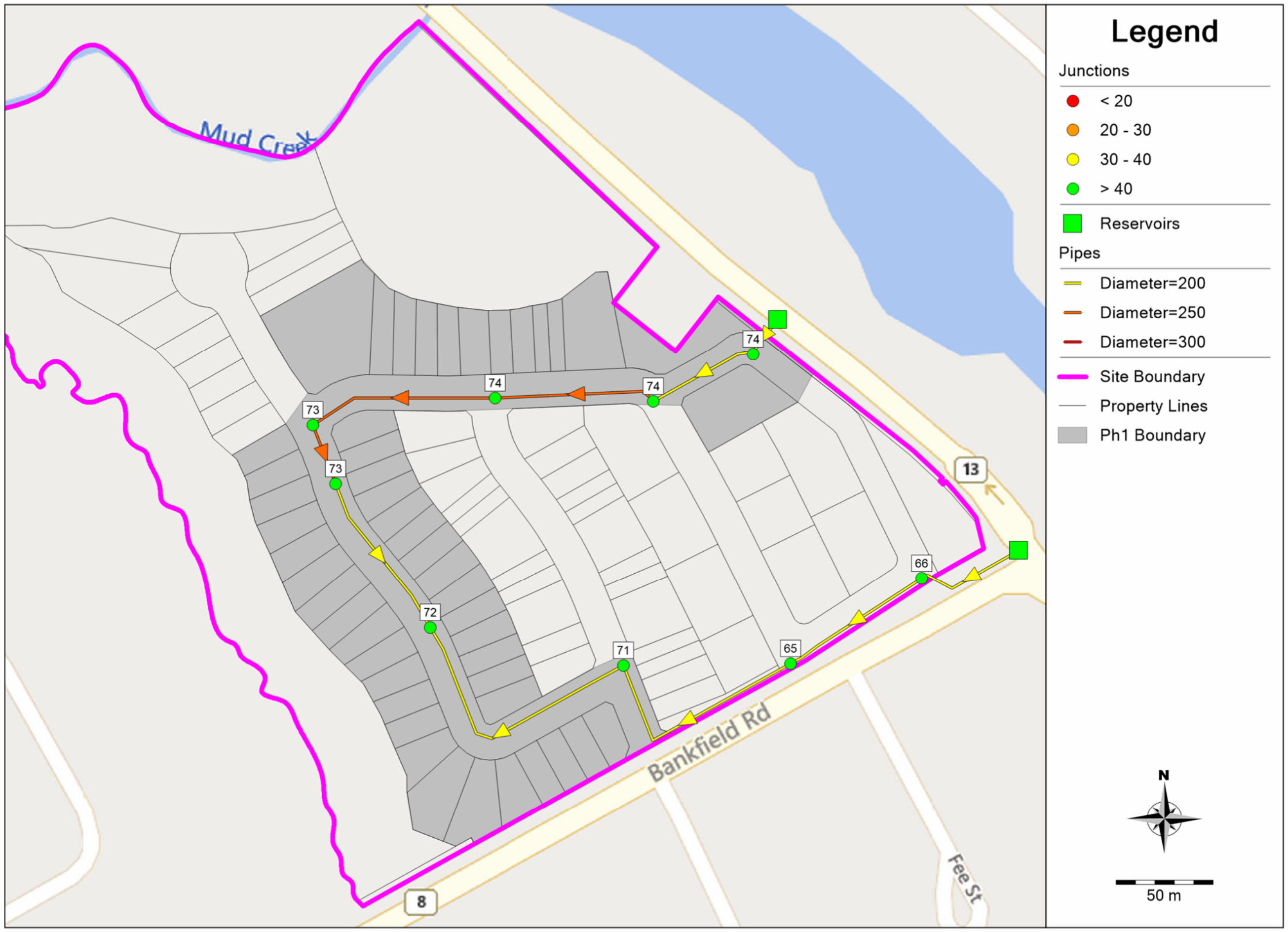
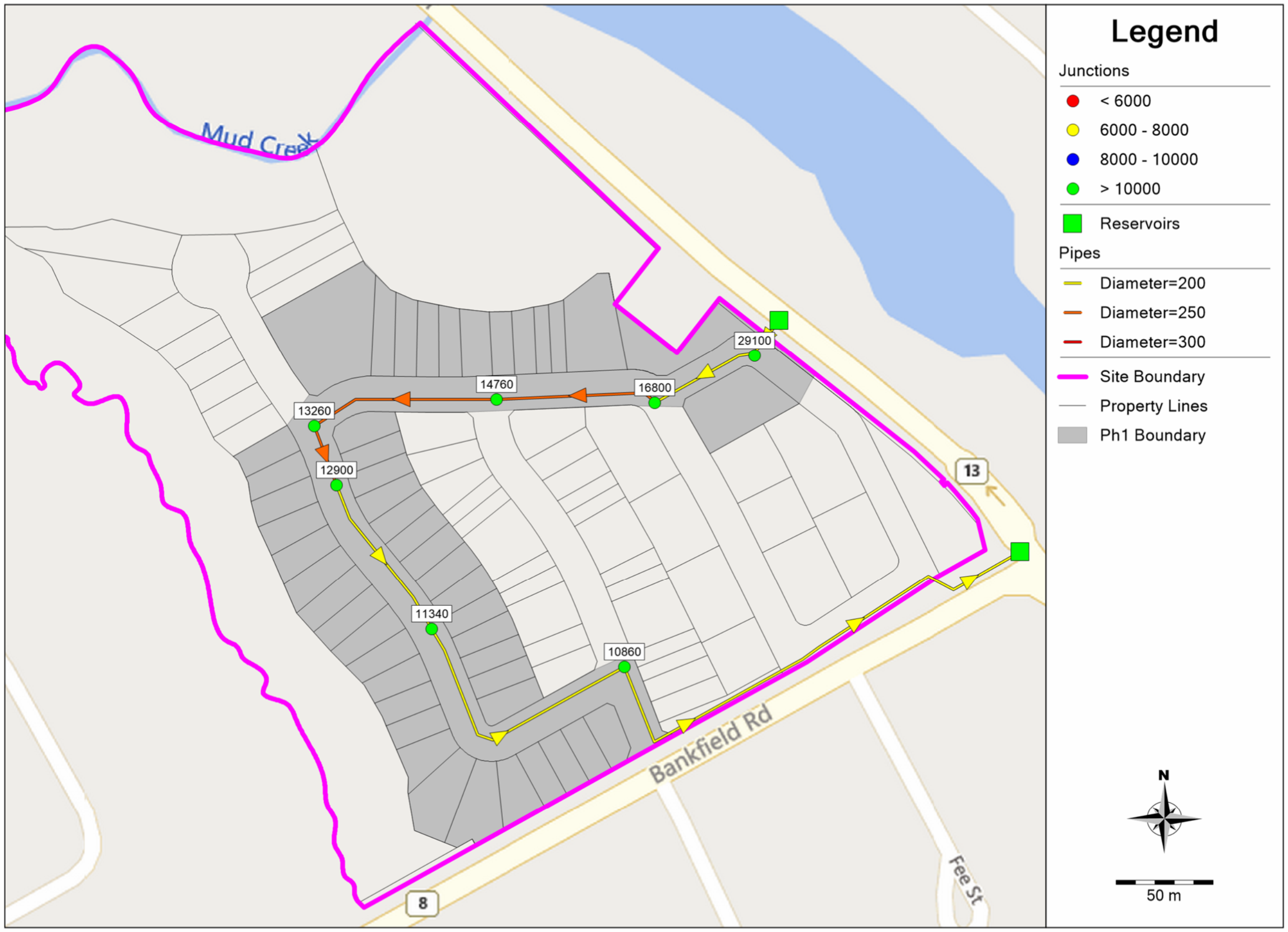


<u>Minimum Pressure During PKHR Conditions – Existing</u>

Date: 2024/04/08
M:\2021\121153\DATA\Calculations\Sewer Calcs\Water_PCSWMM\Images\Ph1\Model Schematics.docx

Stinson Lands (121153) – Ph1
PCSWMM Model Schematic



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

Date: 2024/04/08
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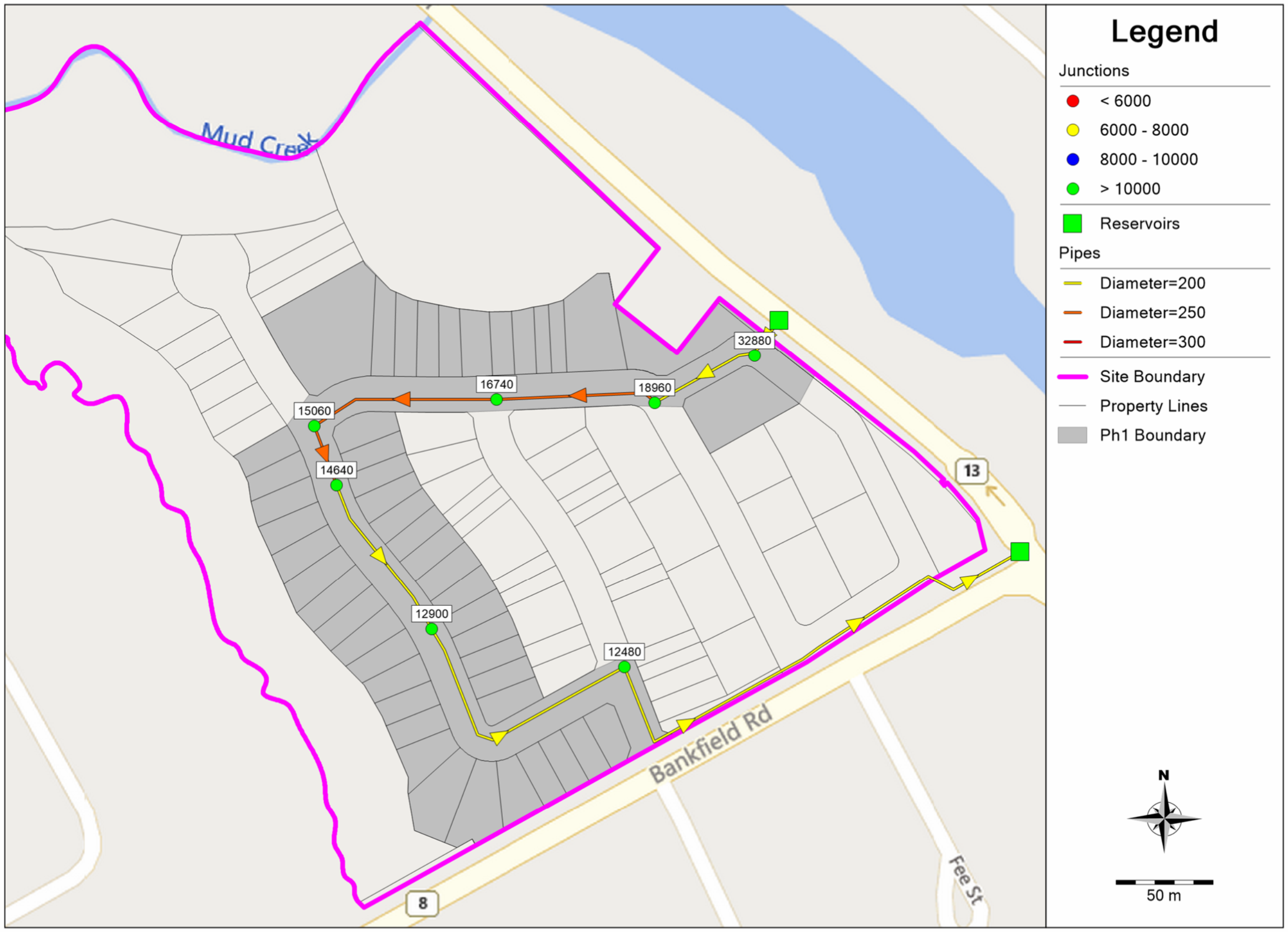
Stinson Lands (121153) – Ph1
PCSWMM Model Schematic

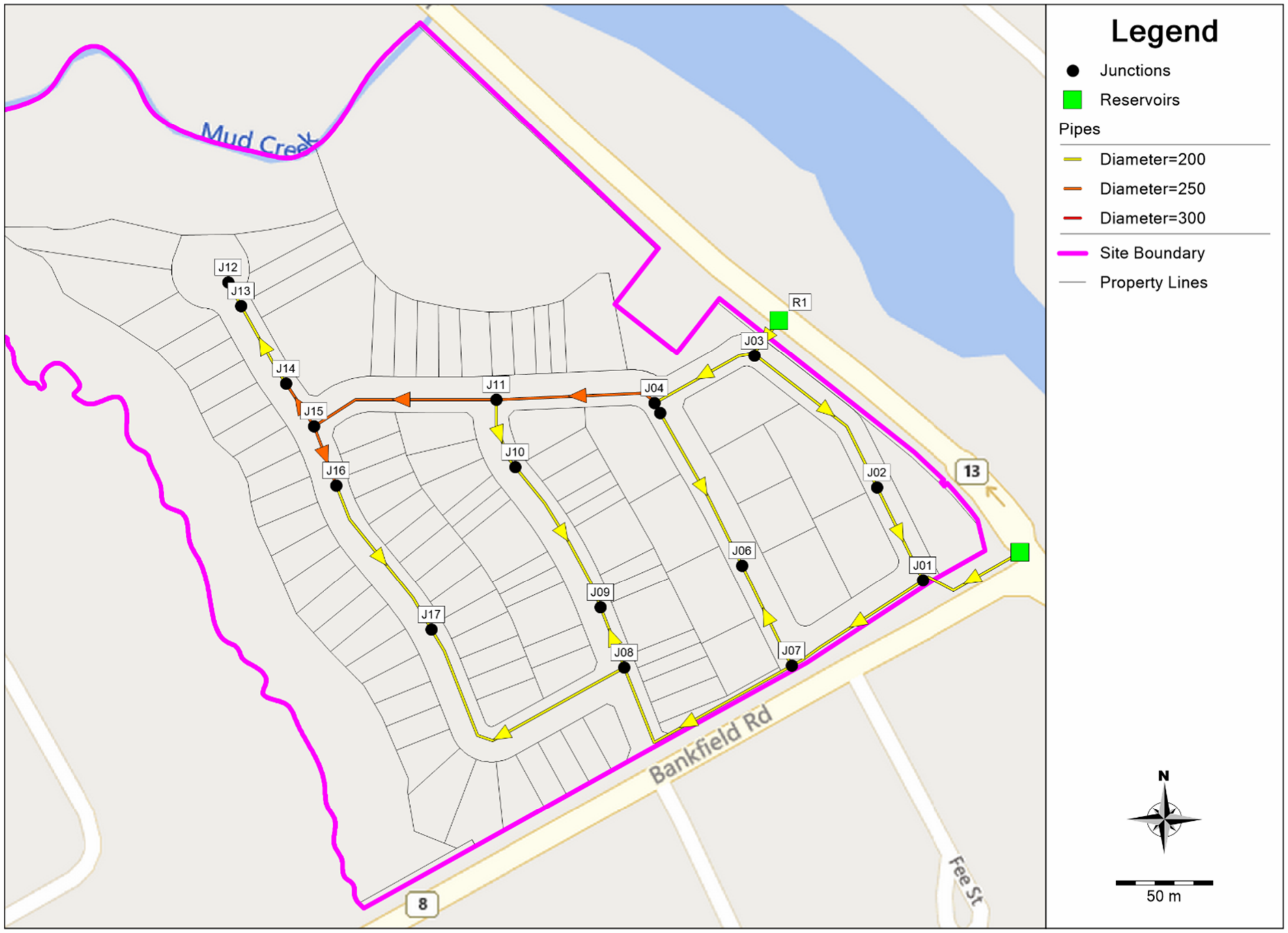


Available Flow at 20psi During MXDY+FF Conditions – Existing

Date: 2024/04/08
M:\2021\121153\DATA\Calculations\Sewer Calcs\Water_PCSWMM\Images\Ph1\Model Schematics.docx

Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic

Engineers, Planners & Landscape Architects



Proposed Watermain Sizing, Layout and Junction IDs

Date: 2024/12/19
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Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic

NOVATECH
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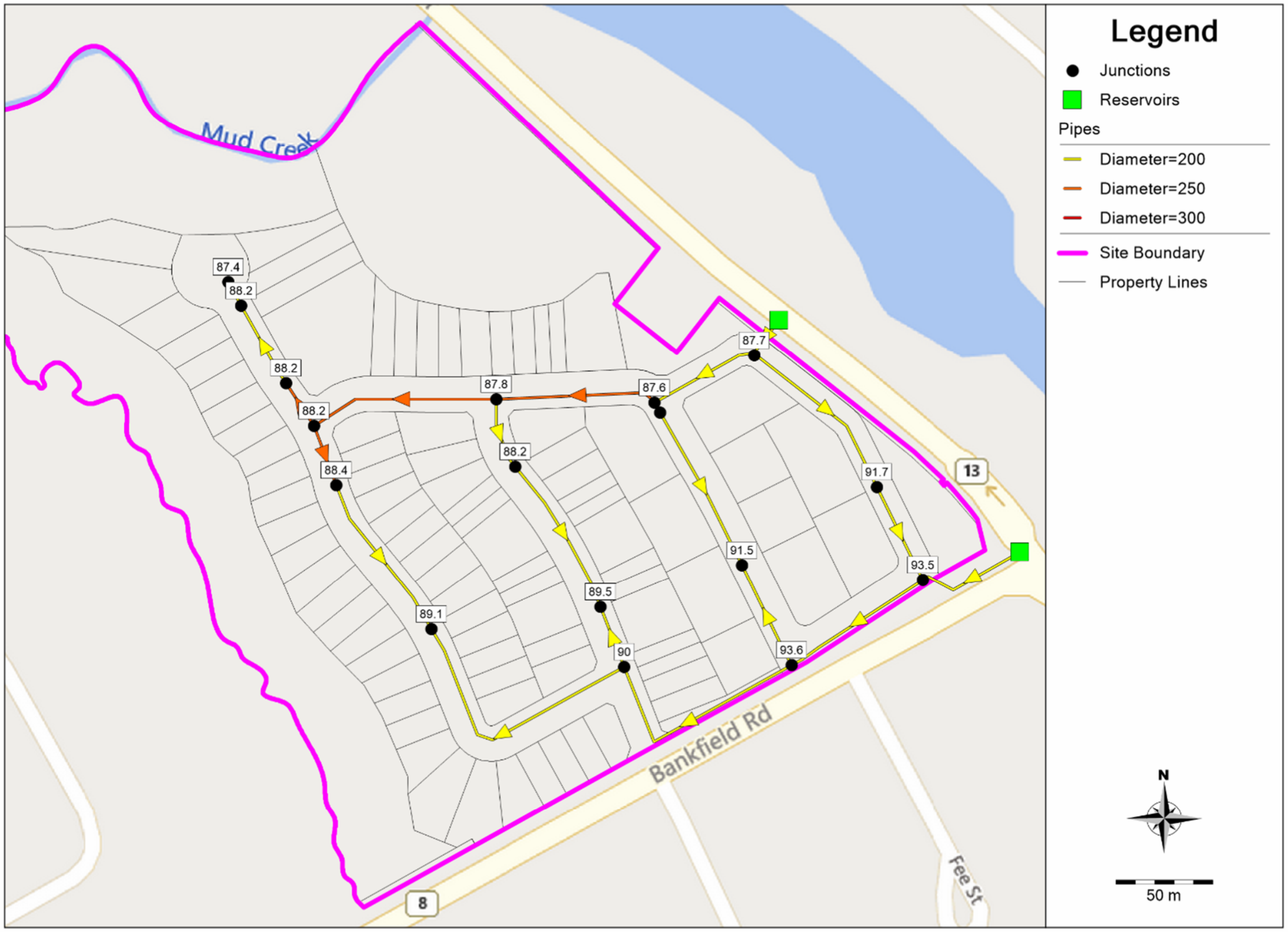

Ground Elevations (m)

Date: 2024/12/19
M:\2021\121153\DATA\Calculations\Sewer Calcs\Water_PCSWMM\Images\Ph2\Model Schematics.docx

Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic

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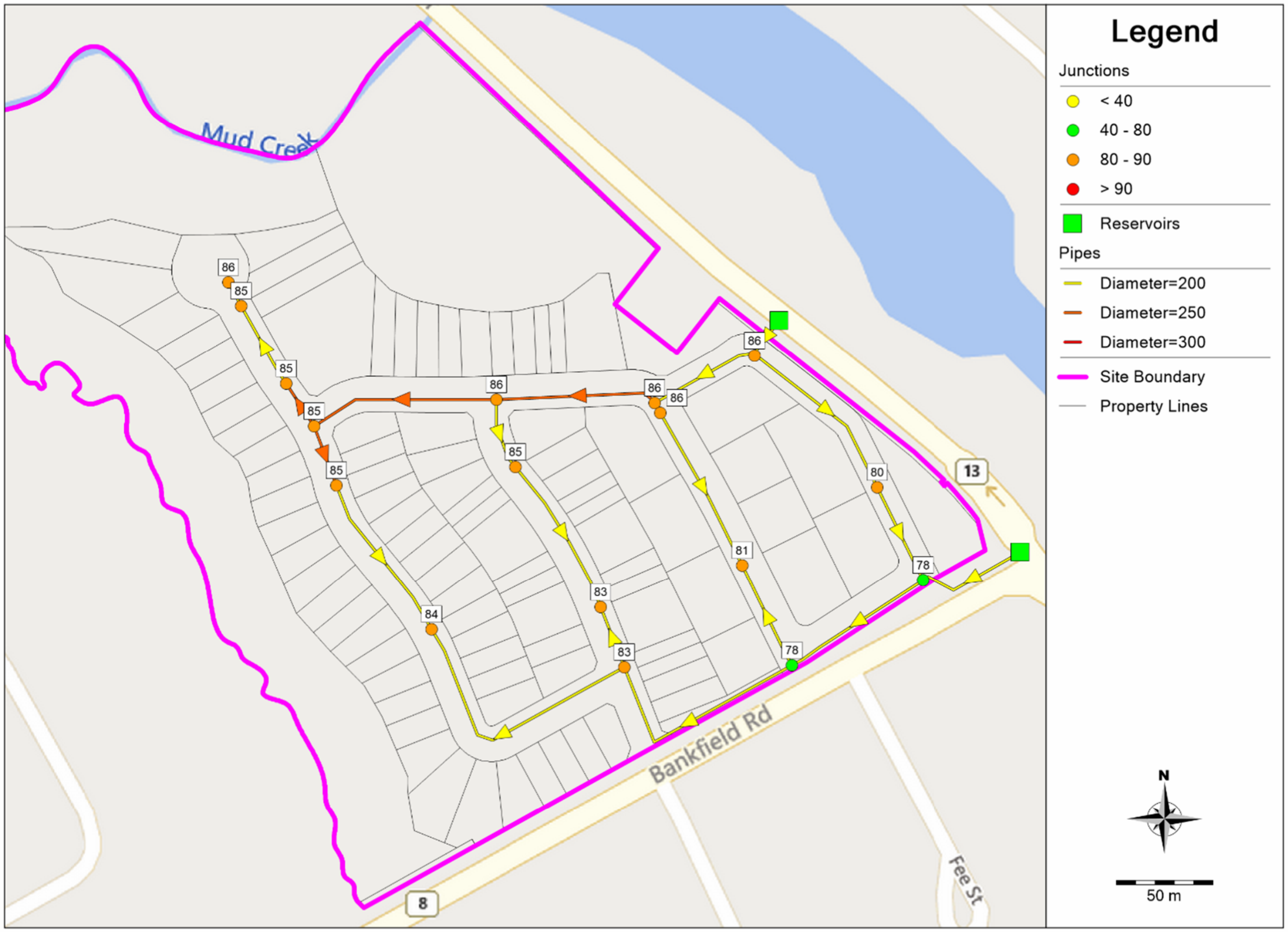

Maximum Pressure During AVDY Conditions – Future

Date: 2024/12/19
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Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic

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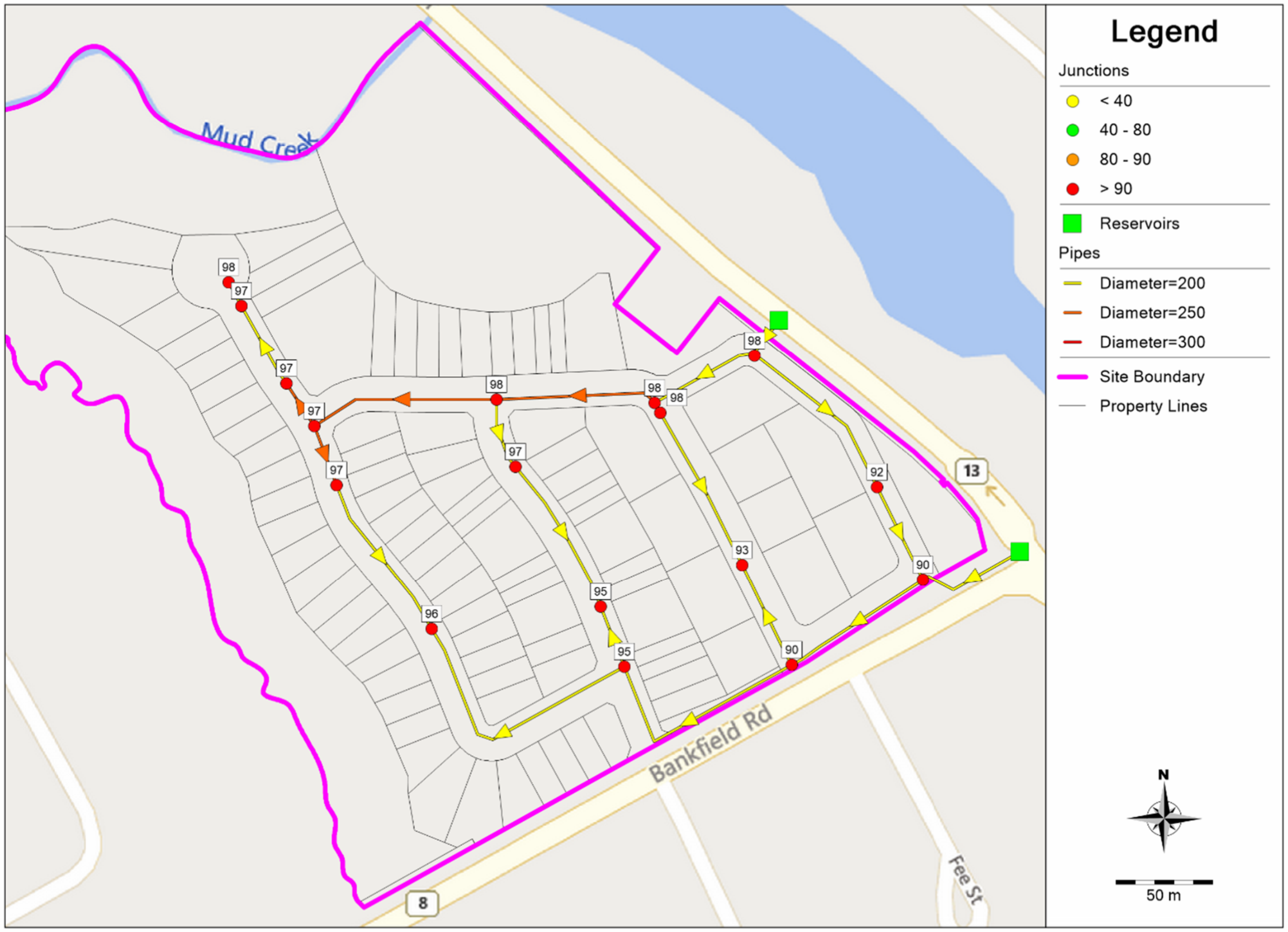

Maximum Pressure During AVDY Conditions – Existing

Date: 2024/12/19
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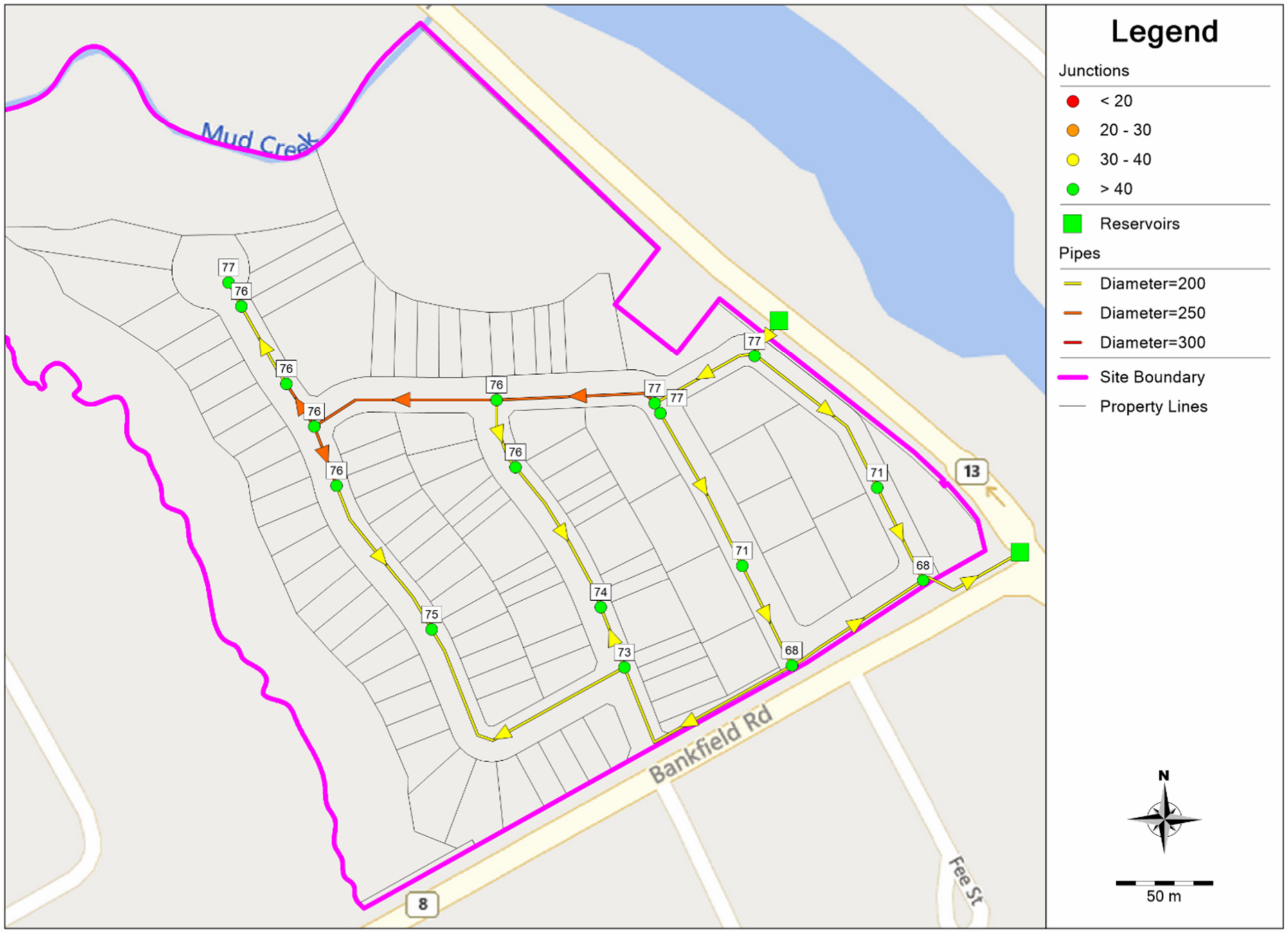
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



Minimum Pressure During PKHR Conditions – Future

Date: 2024/12/19
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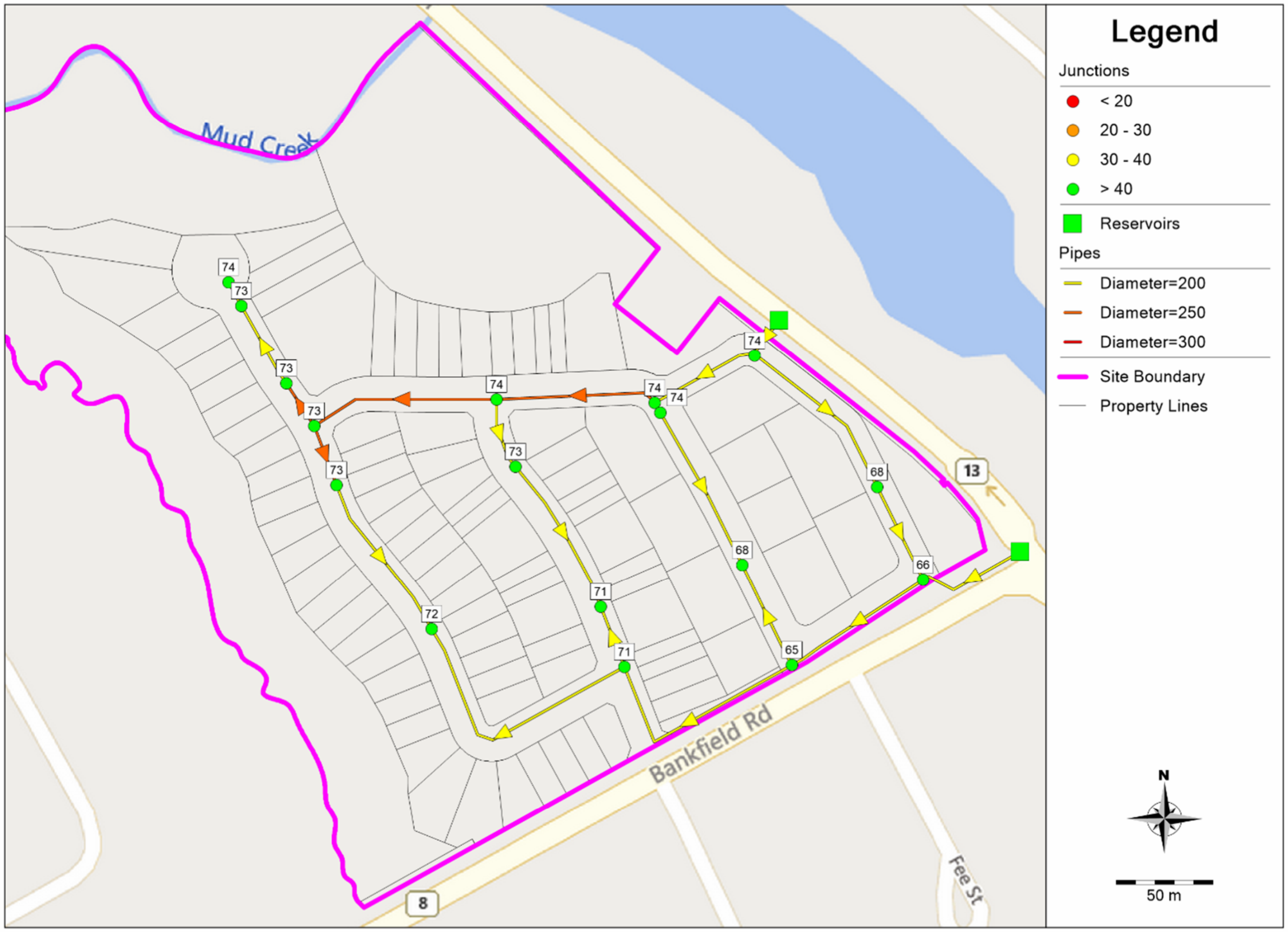
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



Minimum Pressure During PKHR Conditions – Existing

Date: 2024/12/19
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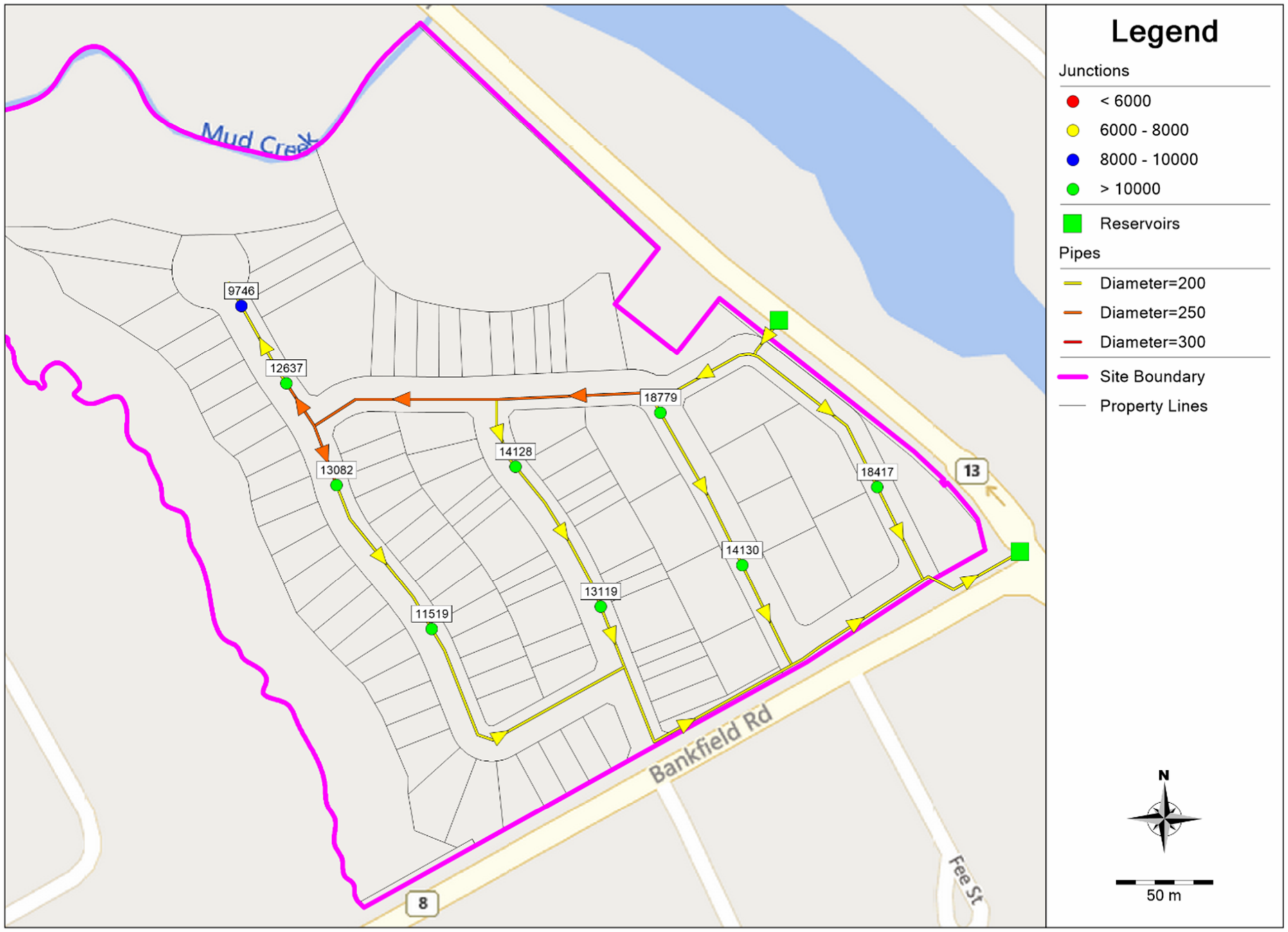
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



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

Date: 2024/12/19
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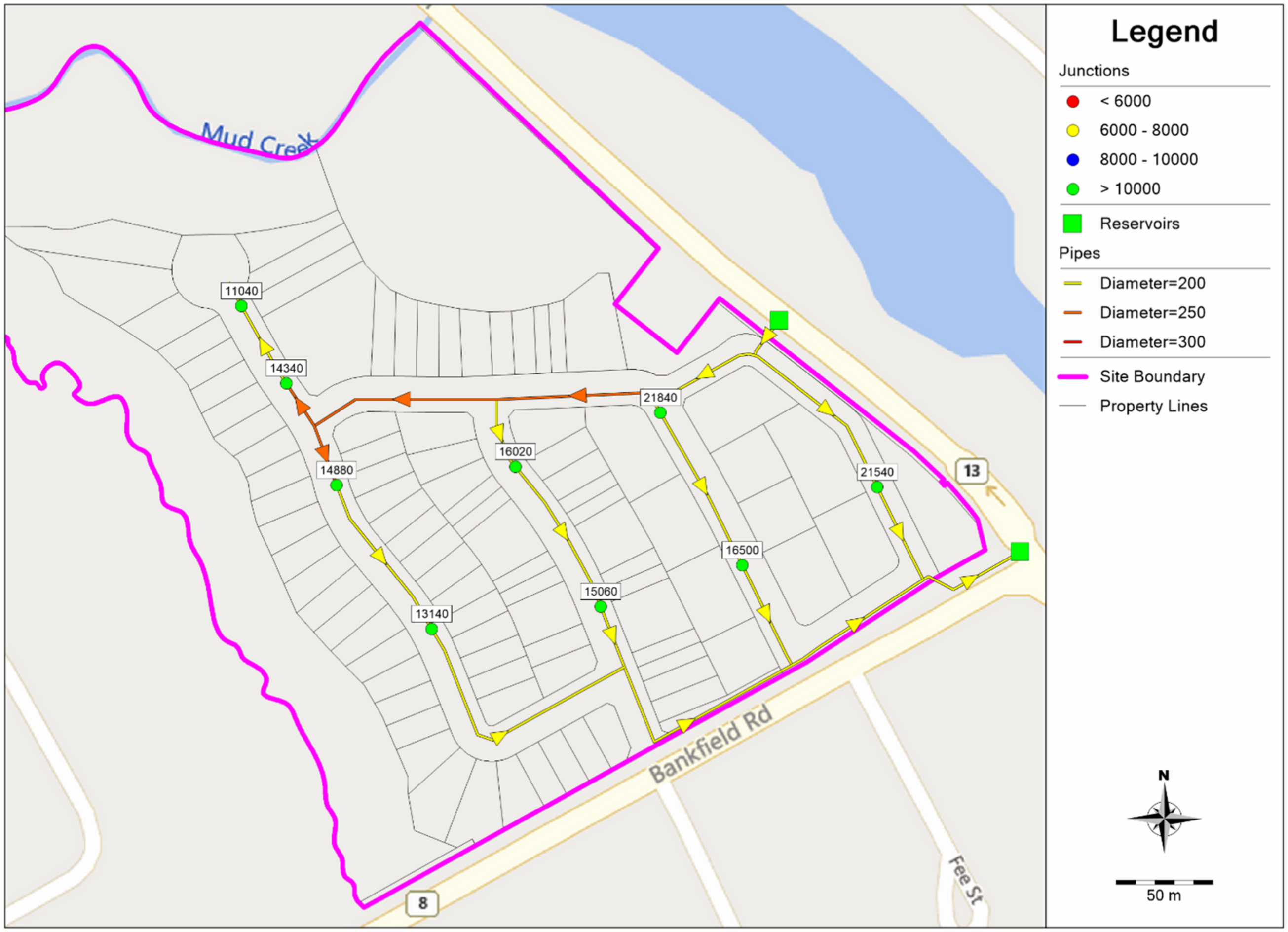
Stinson Lands (121153) – Ph1 + Ph2
PCSWMM Model Schematic



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

Date: 2024/12/19
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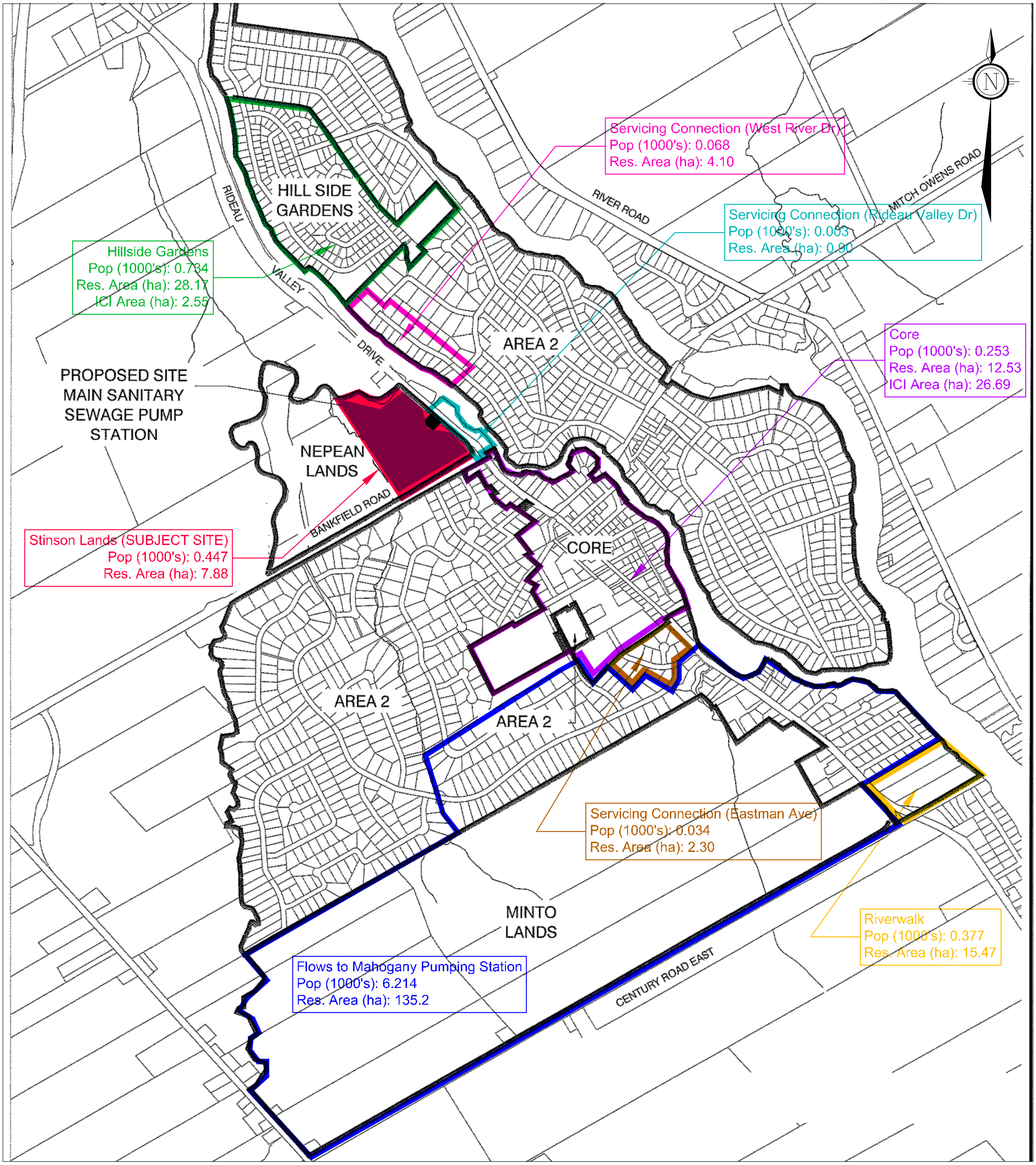

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Ottawa, Ontario, Canada K2M 1P6

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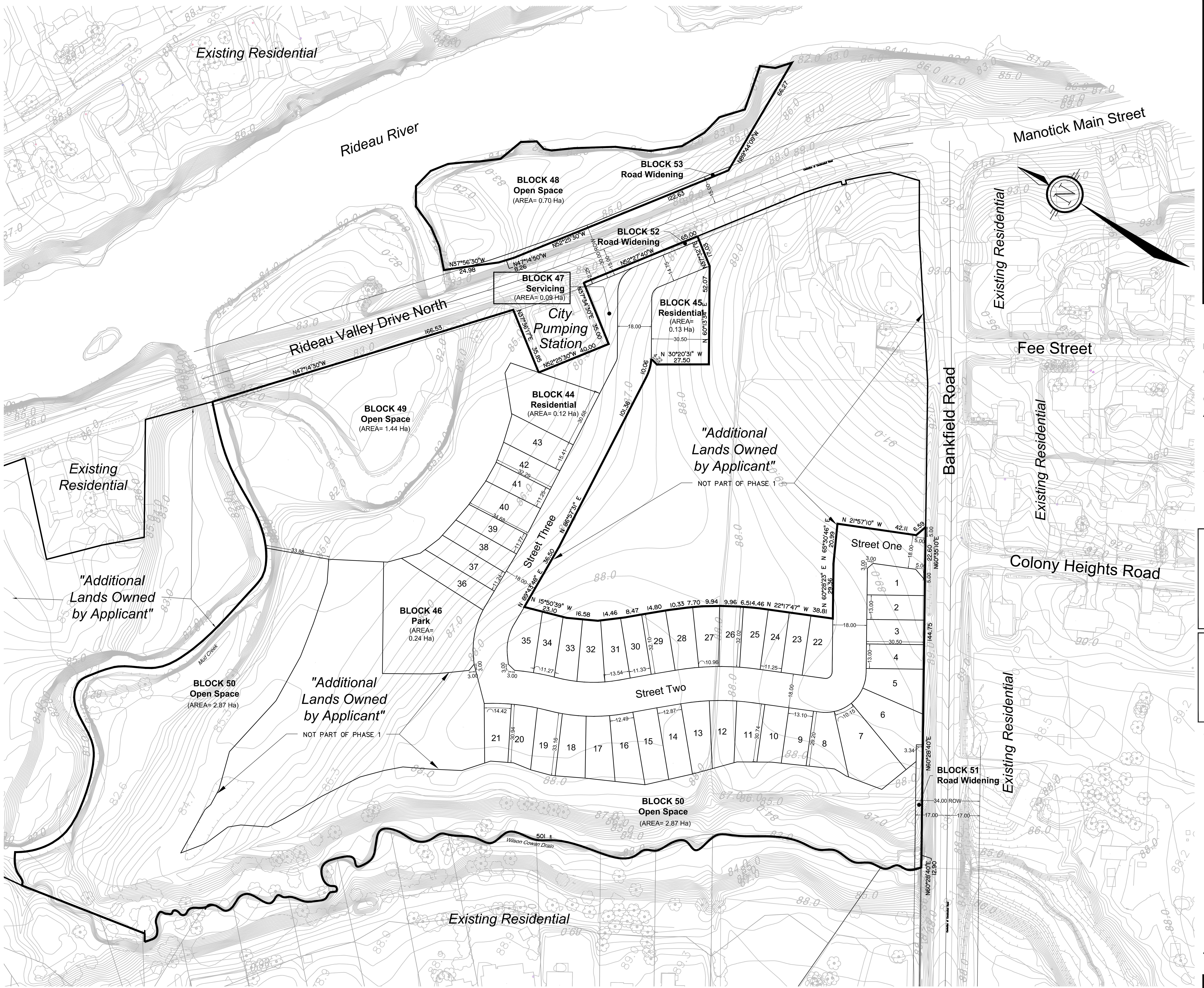
Facsimile	(613) 254-5867
Website	www.novatech-eng.com

MANOTICK PS SERVICING AREAS

STINSON LANDS SUBDIVISION

DATE	JOB	FIGURE
MAR 2025	121153	FIG 5.1

SHT8X11.DWG - 216mmx279mm



KEY MAP
NOT TO SCALE

METRIC : MEASUREMENTS SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

DRAFT PLAN OF SUBDIVISION OF

PART OF LOT 1 CONCESSION 1 (RIDEAU FRONT), PART OF LOT 1 CONCESSION 2 (RIDEAU FRONT) AND PART OF THE ROAD ALLOWANCE BETWEEN CONCESSIONS 1 AND 2

Geographic Township of Nepean
CITY OF OTTAWA
SCALE:
1 : 1000
DATE: MARCH, 2025

SURVEYOR'S CERTIFICATE

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AND THEIR RELATIONSHIP TO ADJOINING LANDS ARE CORRECTLY SHOWN.

DATED March 13, 2025

E. H. Herweyer
ONTARIO LAND SURVEYOR

ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
ONTARIO LAND SURVEYORS 22459-21

OWNER'S CERTIFICATE

WE, UNIFORM URBAN DEVELOPMENTS, BEING THE REGISTERED OWNER(S), HEREBY AUTHORIZE NOVATECH TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE CITY OF OTTAWA FOR REVIEW AND APPROVAL.

DATED March 13, 2025

John MacDougall
John MacDougall
Uniform Urban Developments

ADDITIONAL INFORMATION REQUIRED UNDER SECTION 51 (17) OF THE PLANNING ACT.

A) The boundaries of the land proposed to be subdivided, certified by an Ontario land Surveyor;
As shown on Draft Plan

B) The locations, widths & names of the proposed highways within the proposed subdivision & of existing highways on which the proposed subdivision abuts;
As shown on Draft Plan

C) On a small keyplan, on a scale of not less than 1cm to 100m, all of the land adjacent to the proposed subdivision that is owned by the applicant or in which the applicant has an interest, every subdivision adjacent to the proposed subdivision & the relationship of the boundaries of the land to be subdivided to the boundaries of the township lot of other original grant of which the land forms the whole part;
As Shown on Draft Plan

D) The purpose for which the proposed lots are to be used;
Residential, Open Space, and Park shown on Draft Plan

E) The existing uses of all adjoining lands;
Residential, Open Space, and Park shown on Draft Plan

F) The approximate dimensions & layout of the proposed lots;
As shown on Draft Plan

G) Natural & artificial features such as buildings or other structures or installations, railways, highways, watercources, drainage ditches, wetlands & wooded areas within or adjacent to the land proposed to be subdivided;
As shown on Draft Plan

H) The availability and nature of domestic water supplies;
Development will be supplied with full municipal piped water service

I) The nature & porosity of the soil;
Silty Clay

J) Existing contours or elevations as may be required to determine the grade of the highways and the drainage of the land proposed to be subdivided;
Contours shown on Draft Plan

K) The municipal services available or to be available to the land proposed to be subdivided;
Development will be supplied with full sanitary and storm water sewer services.

L) The nature & extent of any restrictions affecting the land proposed to be subdivided, including restrictive covenants or easements. 1994, c. 23, s. 30; 1996, c. 4, s. 28 (3).;
As shown on Draft Plan.

4386 RIDEAU VALLEY DRIVE - PHASE 1

SUBJECT TO THE CONDITIONS, IF ANY, SET FORTH IN OUR LETTER DATED ______________, THIS DRAFT PLAN IS APPROVED BY THE CITY OF OTTAWA UNDER SECTION 51 OF THE PLANNING ACT THIS ______ DAY OF ________, 20____

ADAM BROWN, MANAGER
DEVELOPMENT REVIEW RURAL ,
PLANNING, INFRASTRUCTURE AND ECONOMIC
DEVELOPMENT DEPARTMENT , CITY OF OTTAWA

NOVATECH
Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

PROJECT No. 121153

D07-16-22-0026

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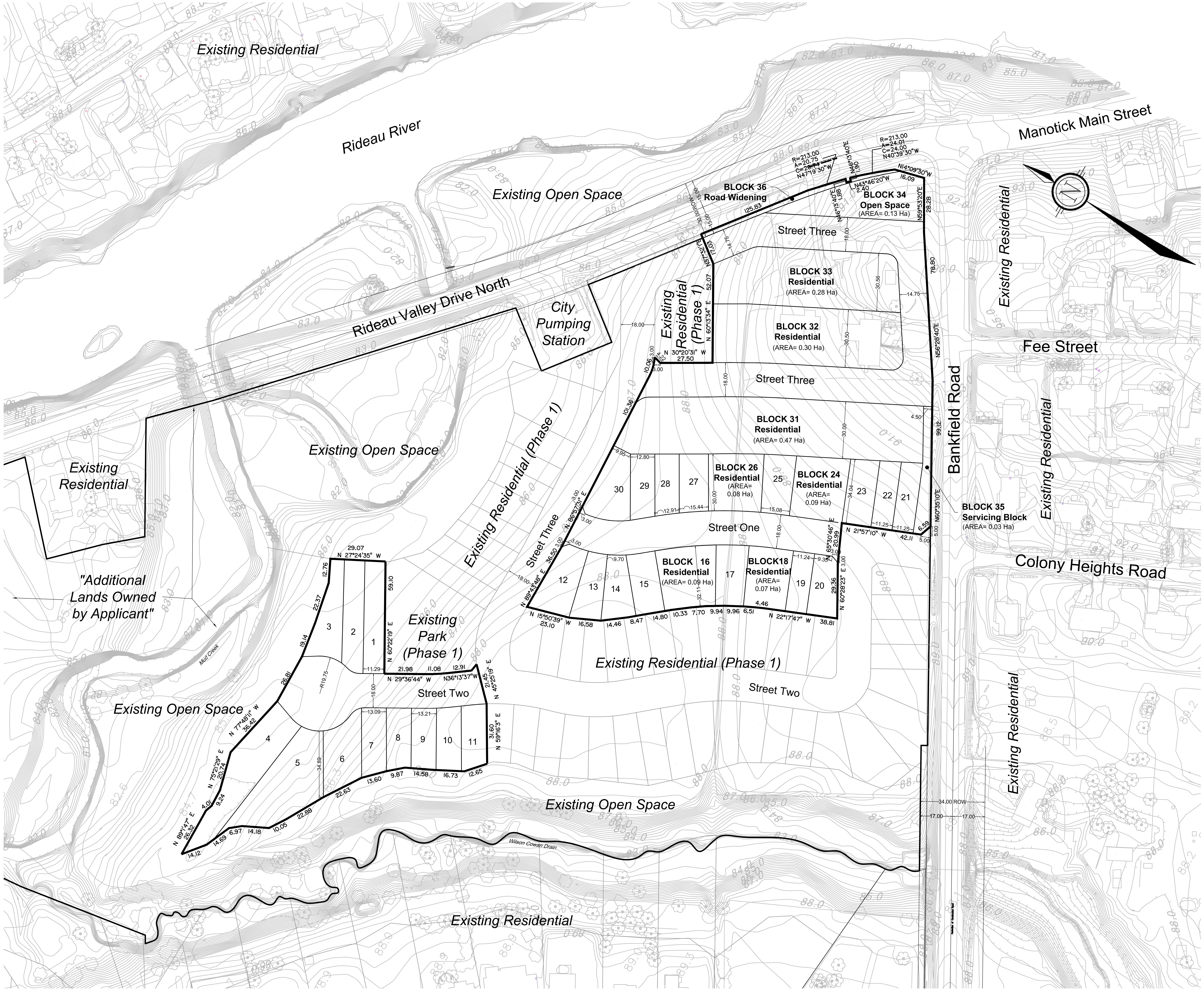

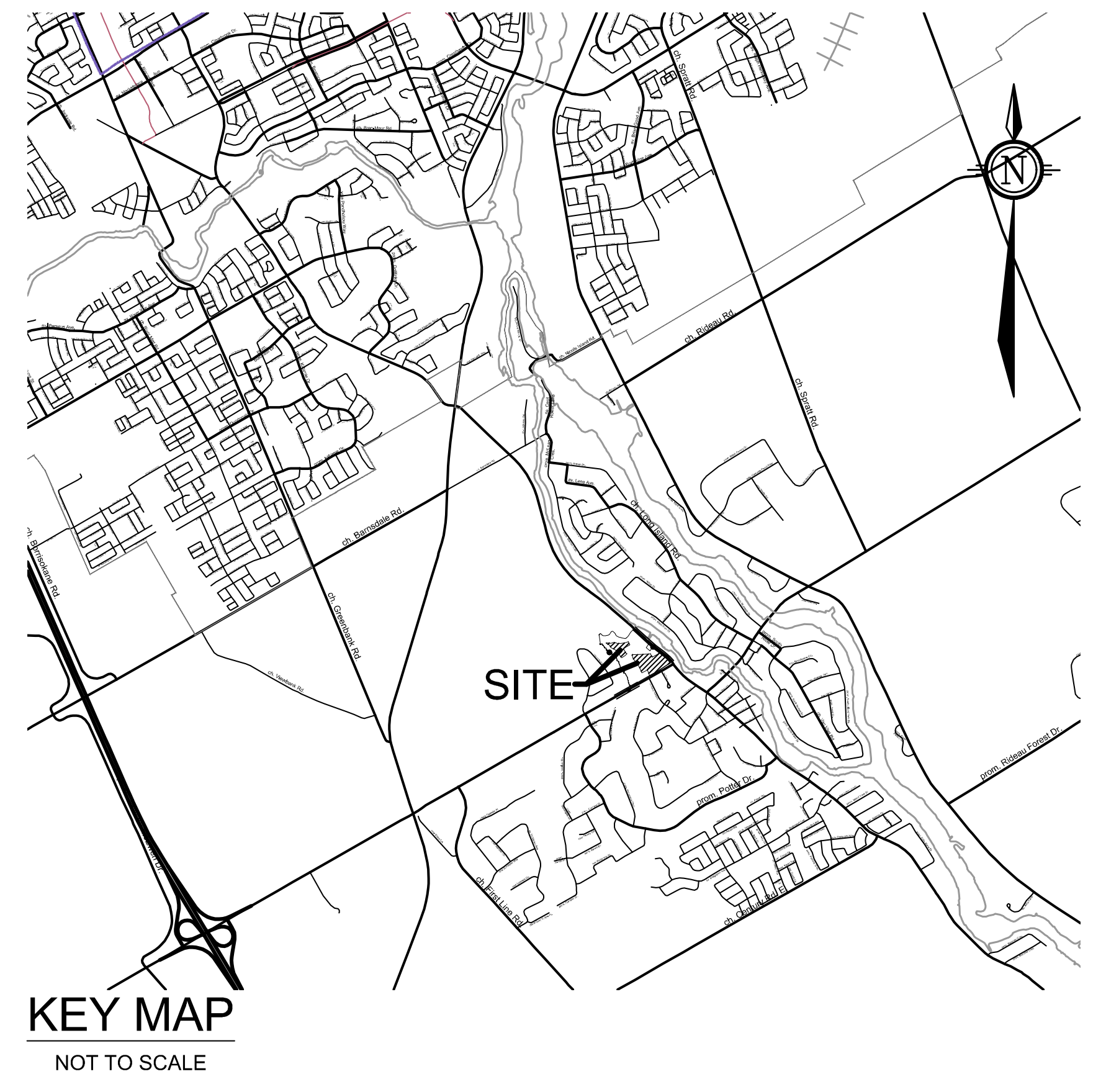

KEY MAP

NOT TO SCALE

METRIC : MEASUREMENTS SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

DRAFT PLAN OF SUBDIVISION OF

PART OF LOT 1
CONCESSION 1 (RIDEAU FRONT),
PART OF LOT 1
CONCESSION 2 (RIDEAU FRONT)
AND PART OF THE ROAD ALLOWANCE
BETWEEN CONCESSIONS 1 AND 2

Geographic Township of Nepean
CITY OF OTTAWA
SCALE:
1 : 1000 0 10 20 30 40
DATE: MARCH, 2025

SURVEYOR'S CERTIFICATE

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED AND THEIR RELATIONSHIP TO ADJOINING LANDS ARE CORRECTLY SHOWN.

DATED March 13, 2025

E. H. Herweyer
ONTARIO LAND SURVEYOR

ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
ONTARIO LAND SURVEYORS 22459-21

OWNER'S CERTIFICATE

WE, UNIFORM URBAN DEVELOPMENTS, BEING THE REGISTERED OWNER(S), HEREBY AUTHORIZE NOVATECH TO PREPARE AND SUBMIT THIS DRAFT PLAN OF SUBDIVISION TO THE CITY OF OTTAWA FOR REVIEW AND APPROVAL.

DATED March 13, 2025

John MacDougall

John MacDougall
Uniform Urban Developments

ADDITIONAL INFORMATION REQUIRED UNDER SECTION 51 (17) OF THE PLANNING ACT.

A) The boundaries of the land proposed to be subdivided, certified by an Ontario land Surveyor;
As shown on Draft Plan

B) The locations, widths & names of the proposed highways within the proposed subdivision & of existing highways on which the proposed subdivision abuts;
As shown on Draft Plan

C) On a small keyplan, on a scale of not less than 1cm to 100m, all of the land adjacent to the proposed subdivision that is owned by the applicant or in which the applicant has an interest, every subdivision adjacent to the proposed subdivision & the relationship of the boundaries of the land to be subdivided to the boundaries of the township lot of other original grant of which the land forms the whole part;
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Residential, and Open Space shown on Draft Plan

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H) The availability and nature of domestic water supplies;
Development will be supplied with full municipal piped water service

I) The nature & porosity of the soil;
Silty Clay

J) Existing contours or elevations as may be required to determine the grade of the highways and the drainage of the land proposed to be subdivided;
Contours shown on Draft Plan

K) The municipal services available or to be available to the land proposed to be subdivided;
Development will be supplied with full sanitary and storm water sewer services.

L) The nature & extent of any restrictions affecting the land proposed to be subdivided, including restrictive covenants or easements. 1994, c. 23, s. 30; 1996, c. 4, s. 28 (3).;
As shown on Draft Plan.

4386 RIDEAU VALLEY DRIVE - PHASE 2

SUBJECT TO THE CONDITIONS, IF ANY, SET FORTH IN OUR LETTER DATED ______________, THIS DRAFT PLAN IS APPROVED BY THE CITY OF OTTAWA UNDER SECTION 51 OF THE PLANNING ACT THIS ______ DAY OF ________, 20____

ADAM BROWN, MANAGER
DEVELOPMENT REVIEW RURAL ,
PLANNING, INFRASTRUCTURE AND ECONOMIC
DEVELOPMENT DEPARTMENT , CITY OF OTTAWA

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