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Stinson Lands 4386 Rideau Valley Drive Conceptual Site Servicing & Stormwater Management Report

**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

Novatech File: 121153

April 22, 2024

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 Drive 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 Bahiā, M.Eng., P. Eng.
Senior Project Manager | Land Development

cc: Ryan McDougall / Annibale Ferro, Uniform Urban Developments

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1.0 INTRODUCTION

1.1 Background

This report 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**.

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 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**. 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 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**. The proposed development concept is shown on **Figure 1.3 – Site Plan**.

Table 1.1: Land Use, Development Potential, and Yield

Unit Type	Number of Units	Area
Singles	62	3.07
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 ha

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.

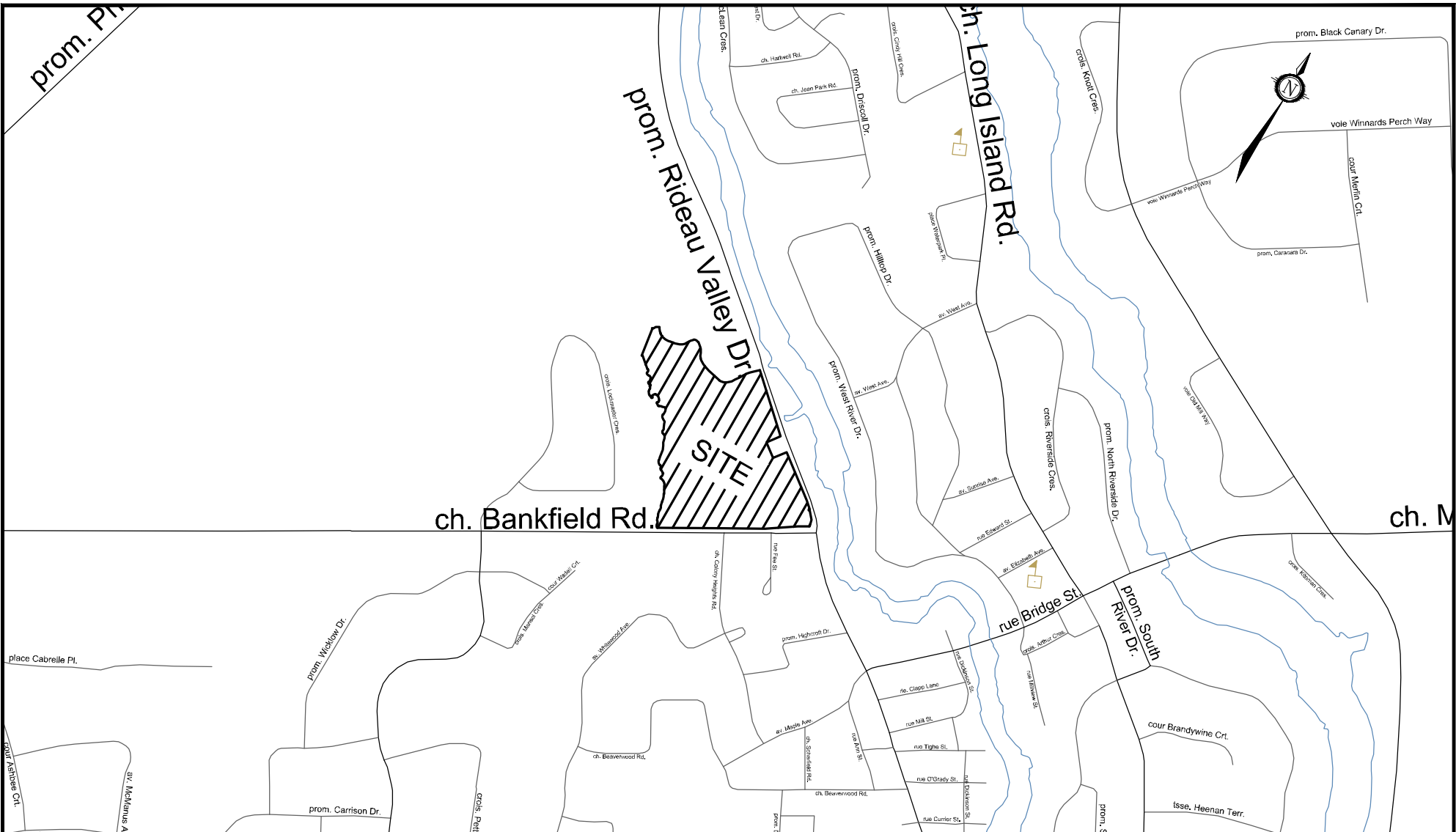
1.3 Report Objective

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**.

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



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
AUG 2022	121153	1.1



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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

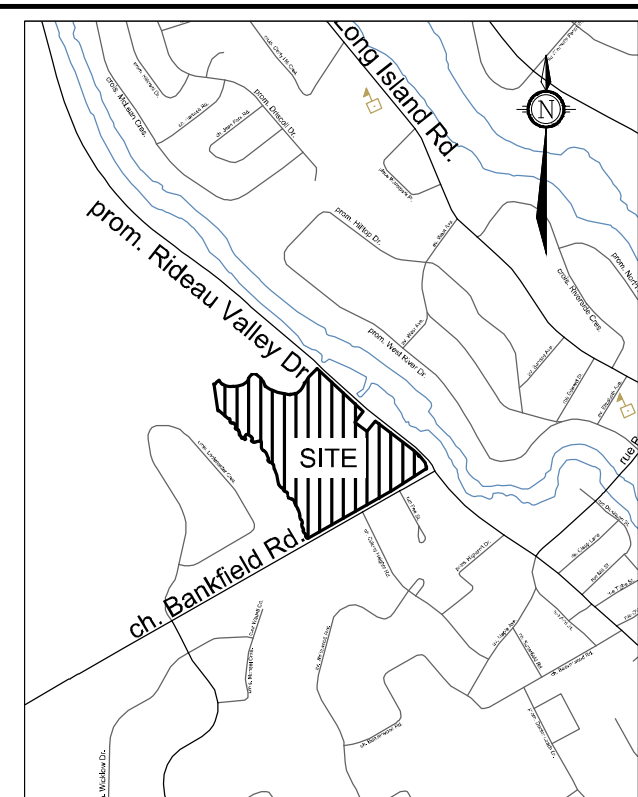
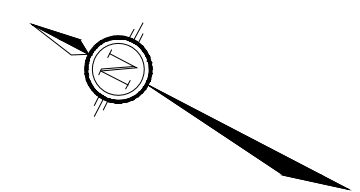
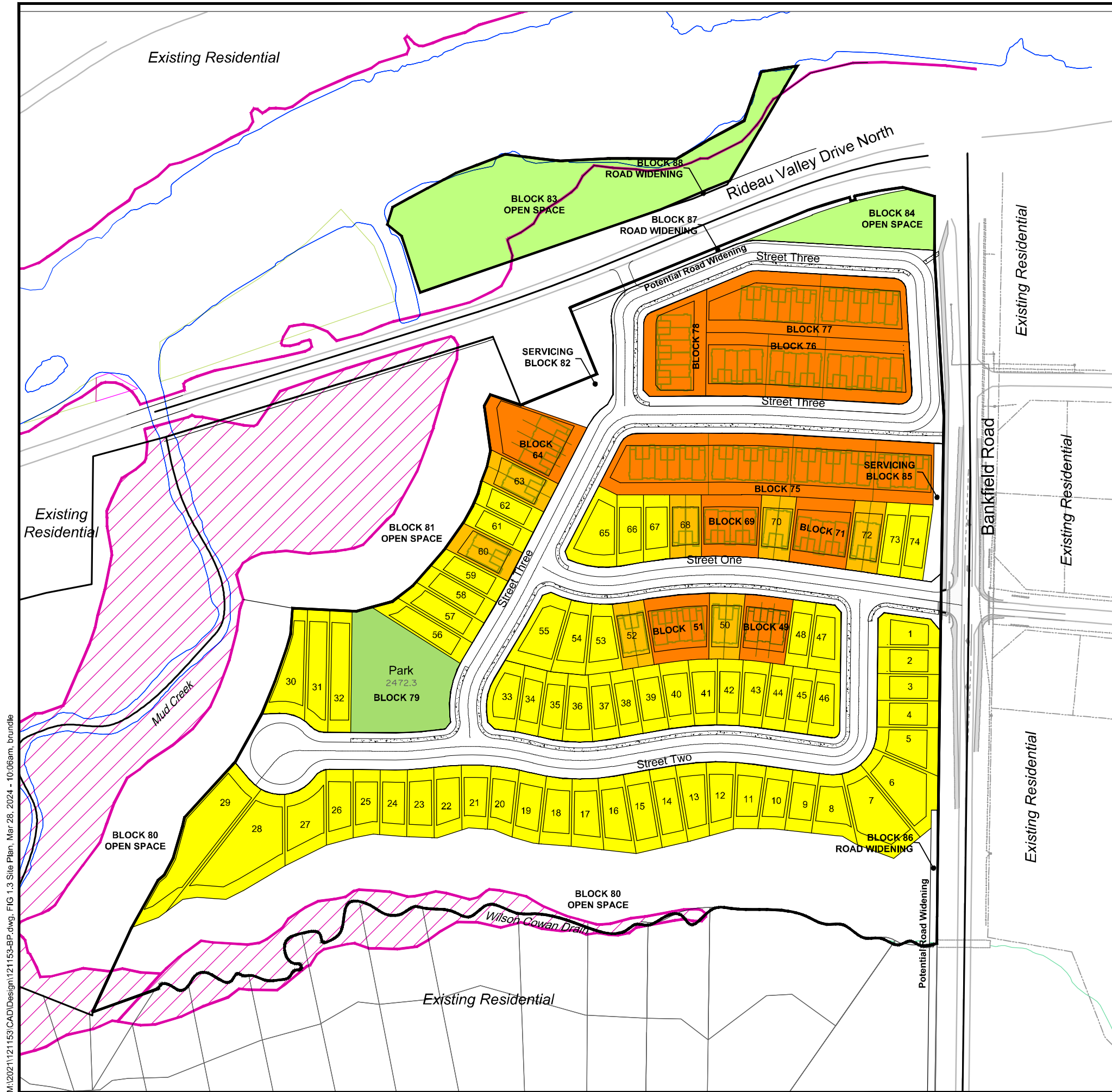
LEGEND

- DEVELOPMENT LIMIT
- BOUNDARY
- WATERCOURSE





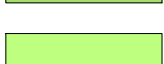

**CITY OF OTTAWA
 STINSON LANDS
 4386 RIDEAU VALLEY DRIVE**

EXISTING CONDITIONS

SCALE		
NOT TO SCALE		
DATE	JOB	FIGURE
APR 2024	121153	1.2



LEGEND

	Single Detached
	Semi Detached
	Townhome
	Park
	Open Space
	Floodplain

M:\2021\121153\CAD\Design\121153-BP.dwg, FIG 1.3 Site Plan, Mar 28, 2024 - 10:06am, brundle



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 RIDEAU VALLEY DRIVE

SITE PLAN

SCALE	1:2000	
DATE	APR 2024	FIGURE 1.3
JOB	121153	

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. 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.

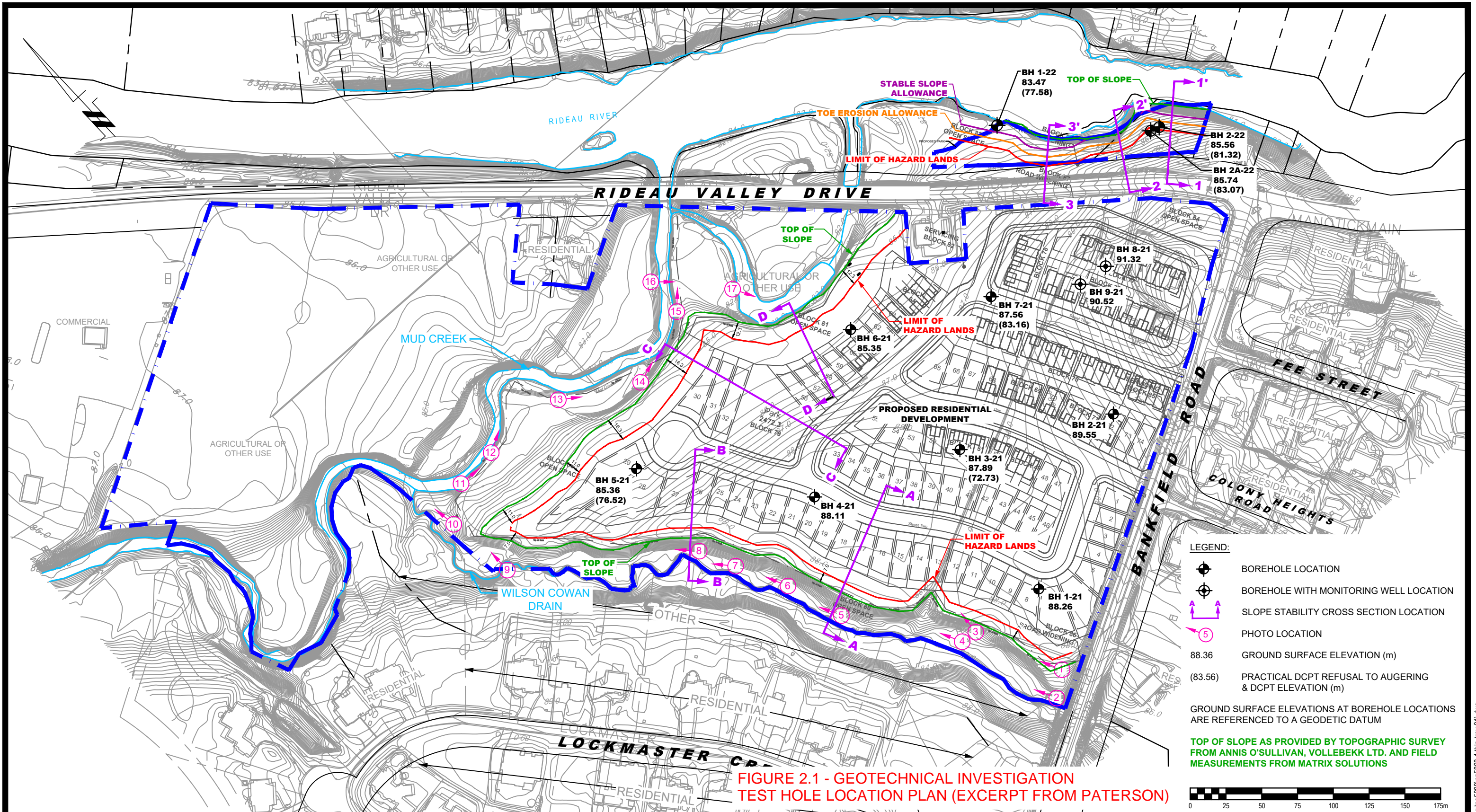


FIGURE 2.1 - GEOTECHNICAL INVESTIGATION TEST HOLE LOCATION PLAN (EXCERPT FROM PATERSON)



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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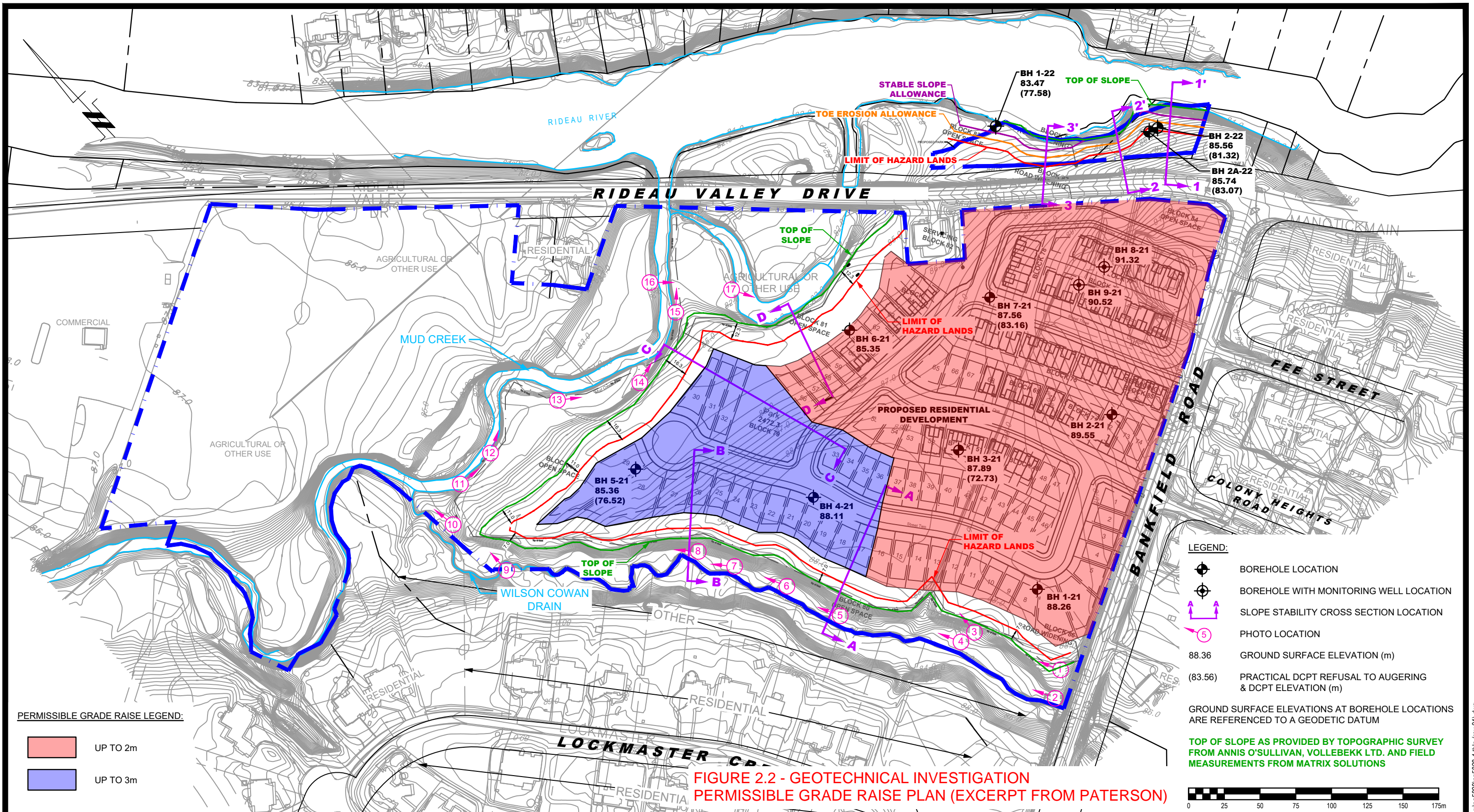


FIGURE 2.2 - GEOTECHNICAL INVESTIGATION PERMISSIBLE GRADE RAISE PLAN (EXCERPT FROM PATERSON)

PERMISSIBLE GRADE RAISE LEGEND:

- UP TO 2m
- UP TO 3m

LEGEND:

- BOREHOLE LOCATION
 - BOREHOLE WITH MONITORING WELL LOCATION
 - SLOPE STABILITY CROSS SECTION LOCATION
 - 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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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.

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.

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 82, 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.

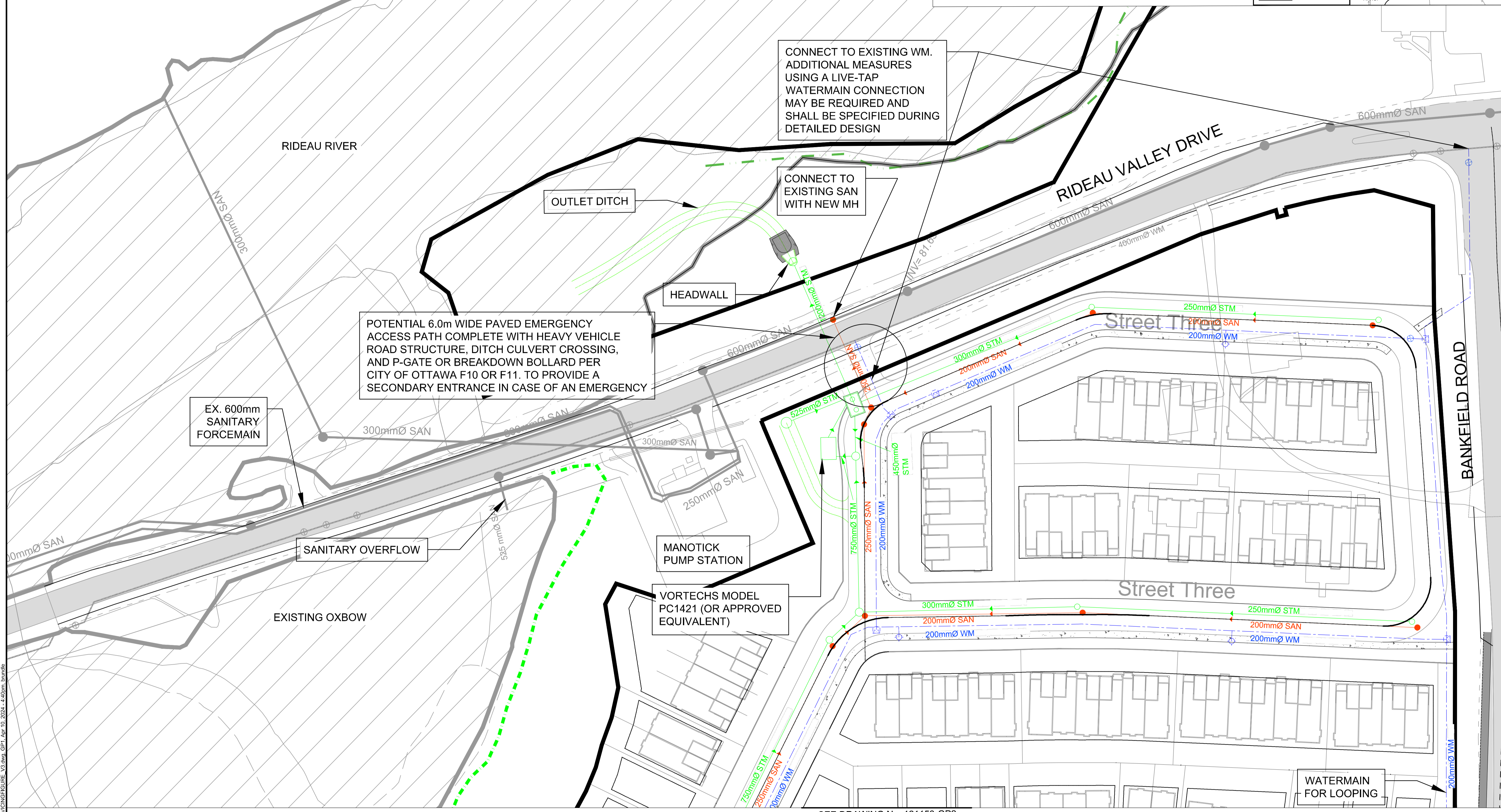
Existing Residential

LEGEND

- EXISTING WATERMAIN
- PROPOSED WATERMAIN
- PROPOSED HYDRANT
- EXISTING SANITARY SEWER
- PROPOSED SANITARY SEWER
- PROPOSED STORM SEWER
- FLOODPLAIN
- Top of Slope (AOV, Matrix)
- New Top of Slope (Novatech)
- Existing Gravel Shoulder
- Existing Paved Shoulder
- Existing Ditch

KEY PLAN
N.T.S.

NORTH



SEE DRAWING No. 121153-GP2

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMANS, 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 RE-SUBMISSION REVIEW	APR 10/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

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FOR REVIEW ONLY

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PROFESSIONAL ENGINEER

B. C. SWEET
100206031
APRIL 22, 2024
PROVINCE OF ONTARIO

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

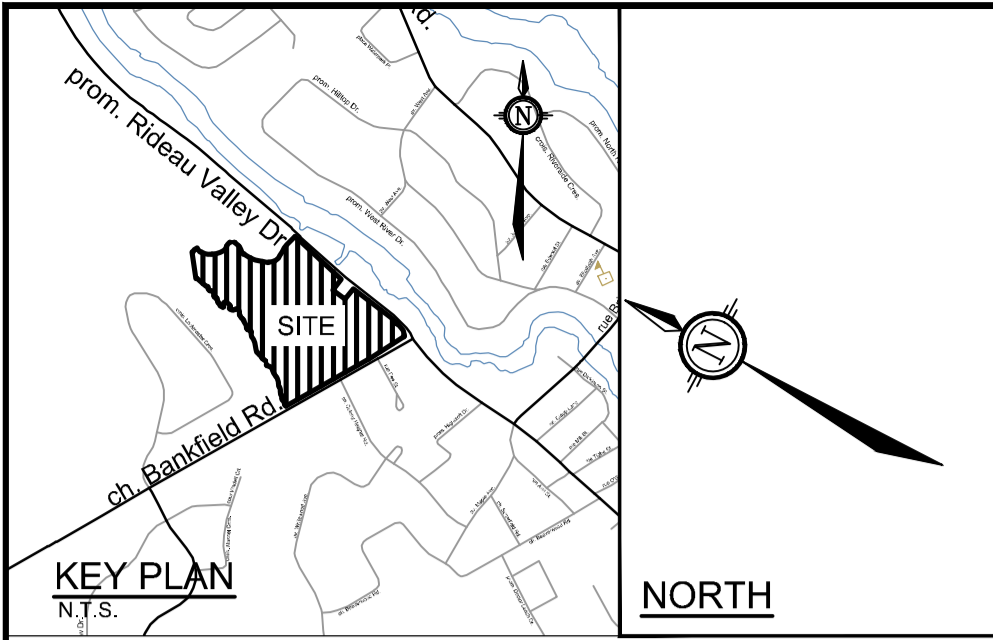
LOCATION
CITY OF OTTAWA
STINSON LANDS SUBDIVISION

DRAWING NAME
CONCEPTUAL GENERAL PLAN OF SERVICES

PROJECT No. 121153-00

REV #2

DRAWING No. 121153-GP1



LEGEND

	EXISTING WATERMAIN		FLOODPLAIN
	PROPOSED WATERMAIN		Top of Slope (AOV, Matrix)
	PROPOSED HYDRANT		New Top of Slope (Novatech)
	EXISTING SANITARY SEWER		Existing Gravel Shoulder
	PROPOSED SANITARY SEWER		Existing Paved Shoulder
	PROPOSED STORM SEWER		Existing Ditch



NOTE:
 THE POSITION OF ALL POLE LINES, CONDUITS, WATERMANS, 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 RE-SUBMISSION REVIEW	APR 10/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

SCALE	
1:500	

DESIGN	
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 STINSON LANDS SUBDIVISION	
DRAWING NAME CONCEPTUAL GENERAL PLAN OF SERVICES	
PROJECT No. 121153-00	REV #2
DRAWING No. 121153-GP2	

M:\2021\121153\CAD\Design\Figures\121153-SERVINGFIGURE_V3.dwg, GP2, Mar 29, 2024, 3:34pm, bounelle

EROSION AND SEDIMENT CONTROL NOTES:

- ALL EROSION AND SEDIMENT CONTROLS ARE TO BE INSTALLED TO THE SATISFACTION OF THE CITY OF OTTAWA AND MINISTRY OF ENVIRONMENT, CONSERVATION AND PARKS (MECP), 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.
- 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.
- CONTRACTOR IS TO INSTALL LIGHT DUTY SILT FENCE AS PER OPSD 219.110. CONTRACTOR SHALL MAINTAIN SILT FENCE FOR THE DURATION OF THE CONTRACT.
- CONTRACTOR IS TO INSTALL STRAW BALES AS PER OPSD 219.180 AS INDICATED AND DIRECTED BY THE ENGINEER PRIOR TO THE COMMENCEMENT OF CONSTRUCTION.
- ALL AREAS DISTURBED BY CONSTRUCTION ARE TO BE TREATED WITH IMPORTED TOPSOIL, SEED AND MULCH.
- THE CONTRACTOR ACKNOWLEDGES THAT FAILURE TO IMPLEMENT EROSION AND SEDIMENT CONTROL MEASURES MAY BE SUBJECT TO PENALTIES IMPOSED BY ANY APPLICABLE REGULATORY AGENCY.
- ALL STREETS ARE TO BE SWEEPED ONCE ROADWAYS ARE PAVED AND TO CONTINUE FOR THE DURATION OF CONSTRUCTION ACTIVITY. STREETS ARE TO BE SWEEPED REGULARLY AS INDICATED BY THE ENGINEER.
- THE CONTRACTOR SHALL KEEP MATERIAL FOR ADDITIONAL EROSION AND SEDIMENT CONTROL ON-SITE 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 AT ALL TIMES WHICH ARE EASILY ACCESSIBLE AND MAY BE IMPLEMENTED AT A MOMENT'S NOTICE.
- MUD MATS ARE TO BE INSTALLED AND MAINTAINED AT CONSTRUCTION ACCESS POINTS TO MINIMIZE SEDIMENT TRANSFER TO EXISTING ROADWAYS (SEE MUD MAT DETAIL).
- COORDINATE ALL WORK WITH THE OWNER AND CONTRACT ADMINISTRATOR

LEGEND

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

NORTH

KEY PLAN
N.T.S.

RIDEAU RIVER

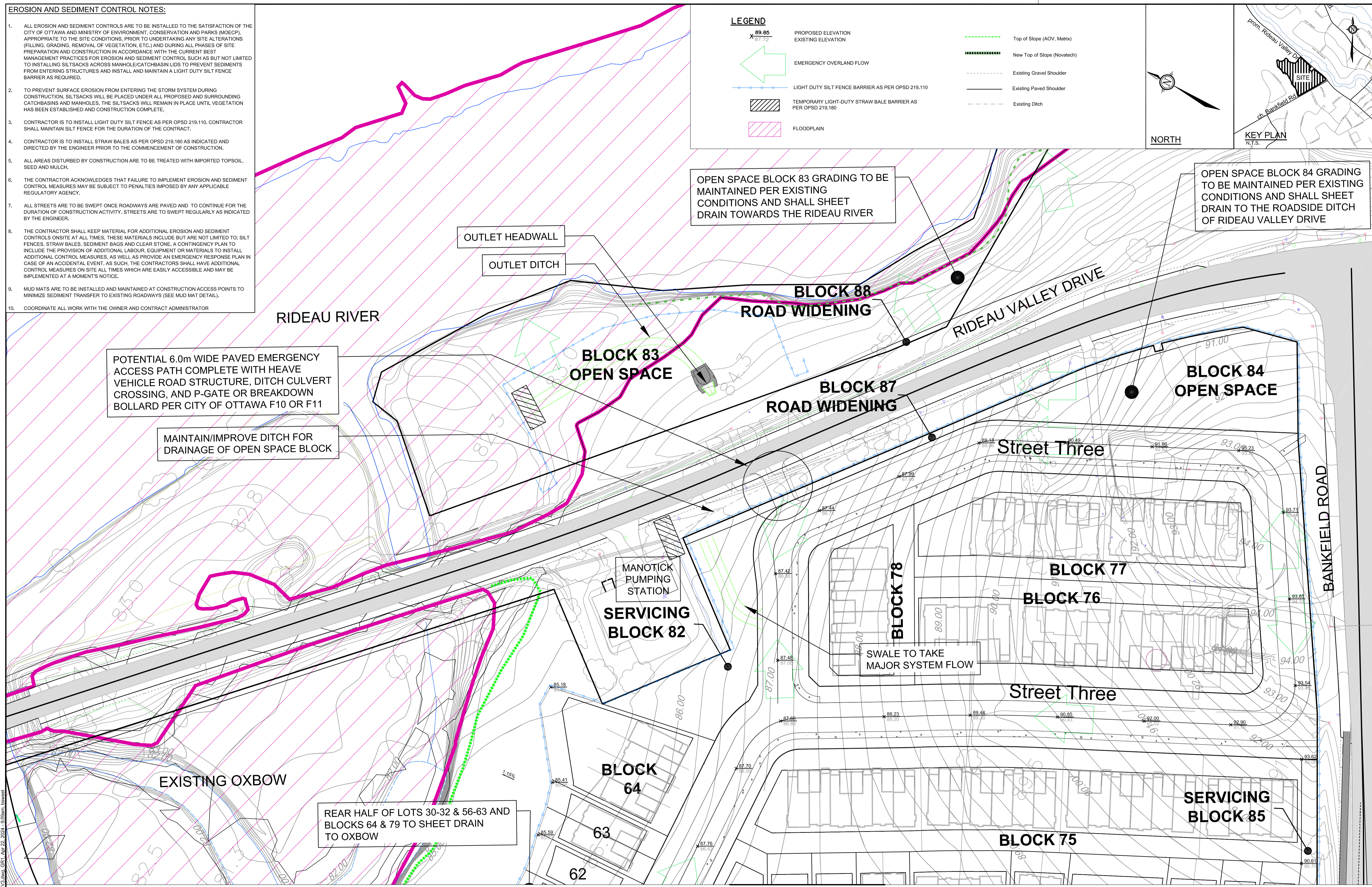
OPEN SPACE BLOCK 83 GRADING TO BE MAINTAINED PER EXISTING CONDITIONS AND SHALL SHEET DRAIN TOWARDS THE RIDEAU RIVER

OPEN SPACE BLOCK 84 GRADING TO BE MAINTAINED PER EXISTING CONDITIONS AND SHALL SHEET DRAIN TO THE ROADSIDE DITCH OF RIDEAU VALLEY DRIVE

POTENTIAL 6.0m WIDE PAVED EMERGENCY ACCESS PATH COMPLETE WITH HEAVE VEHICLE ROAD STRUCTURE, DITCH CULVERT CROSSING, AND P-GATE OR BREAKDOWN BOLLARD PER CITY OF OTTAWA F10 OR F11

MAINTAIN/IMPROVE DITCH FOR DRAINAGE OF OPEN SPACE BLOCK

REAR HALF OF LOTS 30-32 & 56-63 AND BLOCKS 64 & 79 TO SHEET DRAIN TO OXBOW

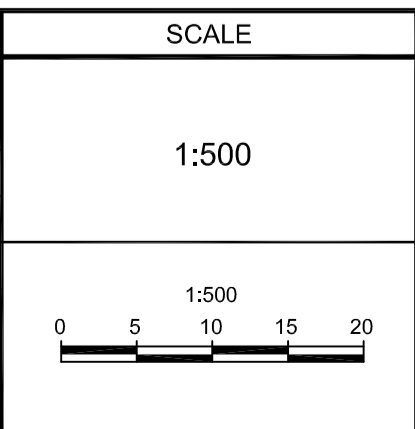


SEE DRAWING No. 121153-GR2

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMANS, 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 RE-SUBMISSION REVIEW	APR 11/24	BHB
1.	ISSUED FOR REVIEW	JAN 6/23	BHB

DESIGN	SCALE
BPR	1:500
BHB	1:500
BPR	1:500
BHB	1:500
BHB	1:500
BHB	1:500



FOR REVIEW ONLY

DESIGN: BPR, BHB, BPR, BHB, BHB, BHB

CHECKED: BPR, BHB, BPR, BHB, BHB, BHB

APPROVED: BHB

PROFESSIONAL ENGINEER
B. C. SWEET
100206031
APRIL 22, 2024
PROVINCE OF ONTARIO

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, STINSON LANDS SUBDIVISION

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

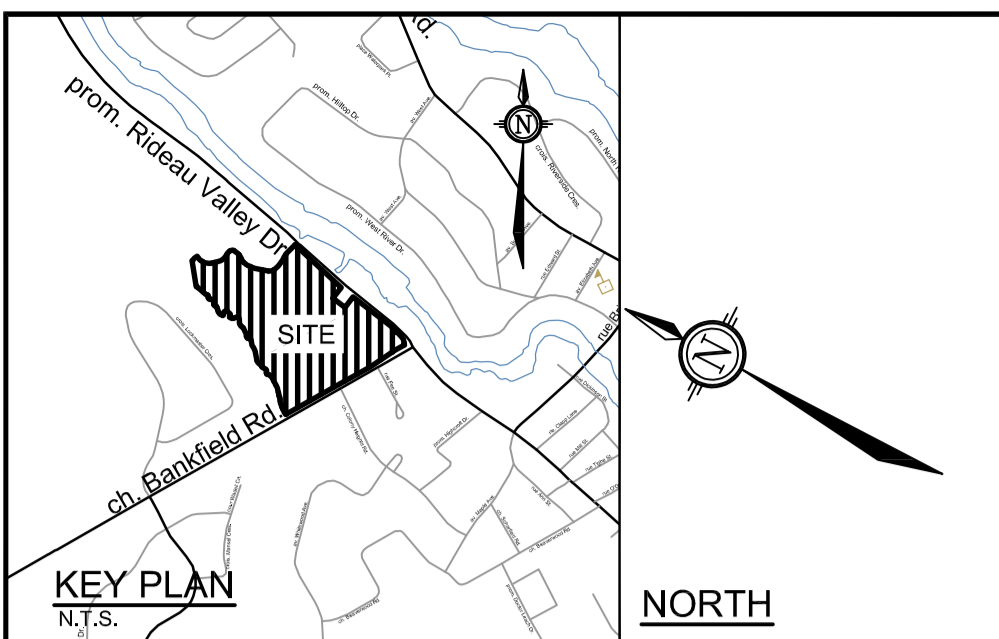
PROJECT No.: 121153-00

REV: REV #2

DRAWING No.: 121153-GR1

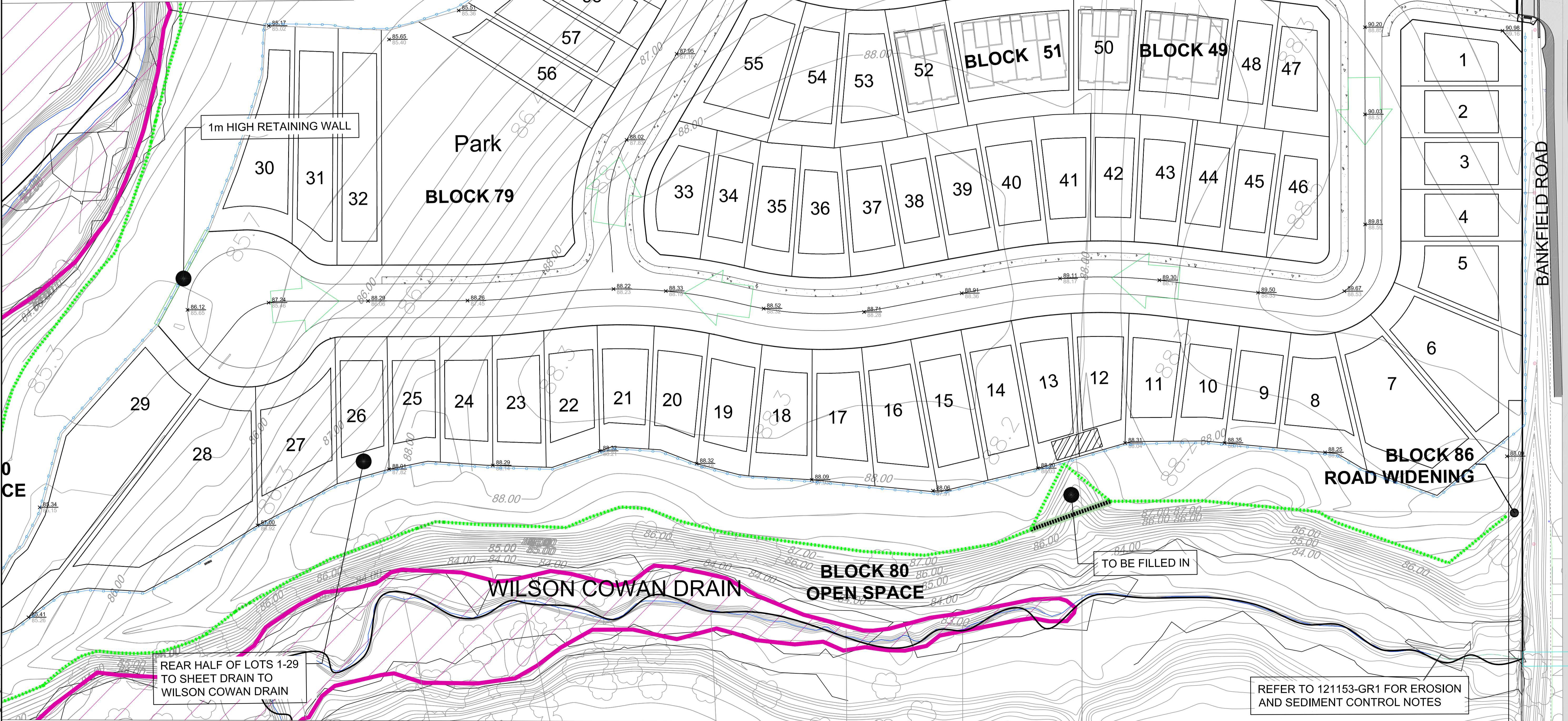
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PL-001-2110-04-1100000000



LEGEND

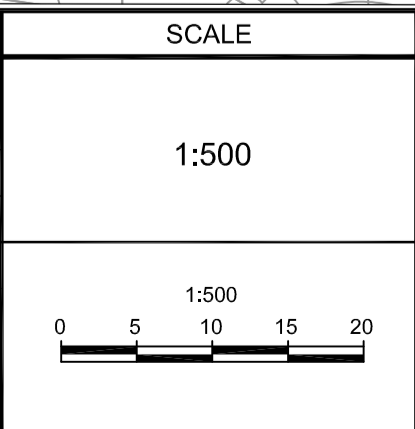
	PROPOSED ELEVATION		FLOODPLAIN
	EXISTING ELEVATION		Top of Slope (AOV, Matrix)
	EMERGENCY OVERLAND FLOW		New Top of Slope (Novatech)
	LIGHT DUTY SILT FENCE BARRIER AS PER OPSD 219.110		Existing Gravel Shoulder
	TEMPORARY LIGHT-DUTY STRAW BALE BARRIER AS PER OPSD 219.180		Existing Paved Shoulder
			Existing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



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DESIGN	SCALE
BPR	1:500
BHB	1:500
BPR	1:500
BHB	1:500
BHB	1:500
BHB	1:500



FOR REVIEW ONLY

DESIGN: BPR
CHECKED: BHB
DRAWN: BPR
CHECKED: BHB
APPROVED: BHB

LICENSED PROFESSIONAL ENGINEER
B. C. SWEET
100206031
APRIL 22, 2024
PROVINCE OF ONTARIO

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 STINSON LANDS SUBDIVISION

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

PROJECT No.: 121153-00
REV: REV # 2
DRAWING No.: 121153-GR2

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.

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 will require approximately 1075 m of pipe ranging from 250 mm to 1200 mm diameter. During detailed design, the right-of-way will be graded to contain the major system runoff from storm events exceeding the minor system capacity for all storms up to and including the 100-year design event. The site will be graded to provide an engineered overland flow route for large, infrequent storms. 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 catchbasins;
- Direct roof leaders to rear yard areas;
- Infiltration trenches underlying drainage swales in park areas;
- The use of fine sandy loam topsoil in parks and on residential lawns.

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

4.4.3 Water Quality Control

Water quality treatment will be provided using a prefabricated 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).

6 Hour Chicago Distribution:

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

12 Hour SCS Type II Distribution:

2-year Event
5-year Event
100-year Event

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

4.5.2 Downstream Boundary Conditions

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.

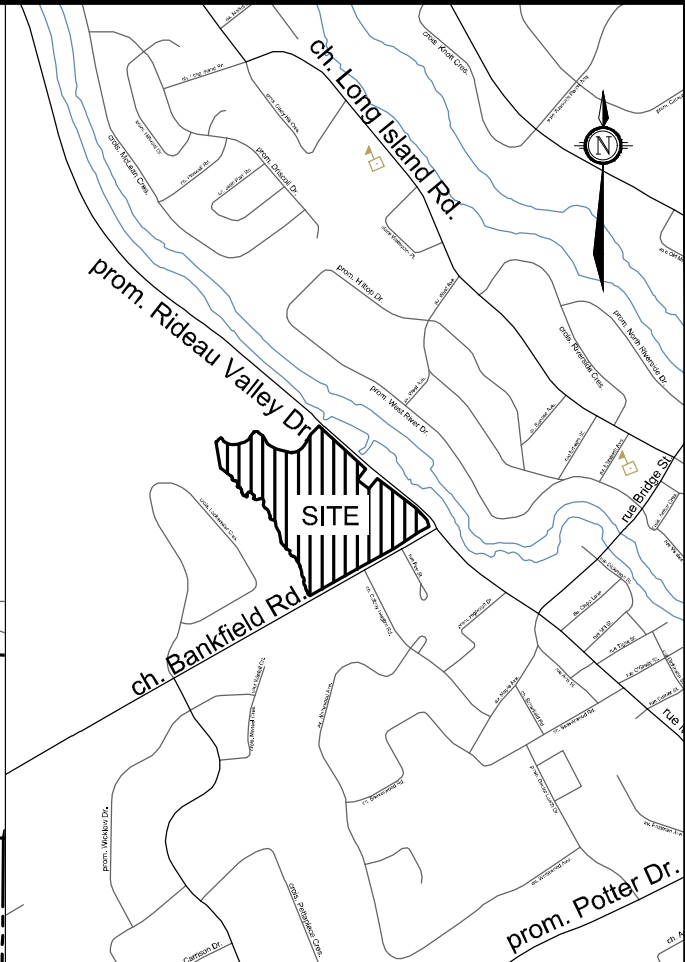
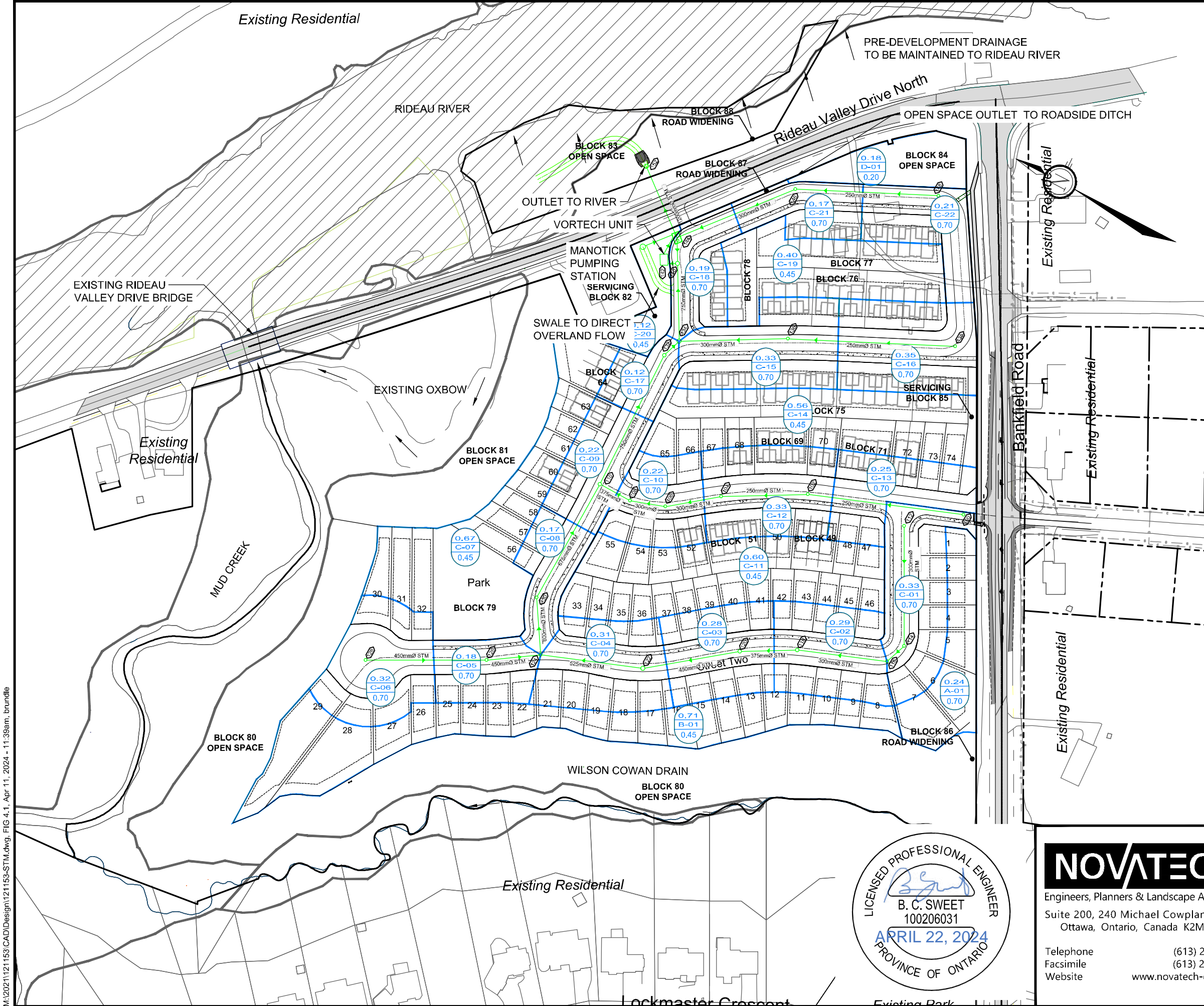
4.5.3 Storm Drainage Areas

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 the Storm Drainage Area Plan (**121153-STM**). An overview of the modeling parameters is provided in **Table 4.3**.



LEGEND

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

LICENSED PROFESSIONAL ENGINEER

 B. C. SWEET
 100206031
 APRIL 22, 2024
 PROVINCE OF ONTARIO

NOVATECH
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CITY OF OTTAWA
 STINSON LANDS
 4386 RIDEAU VALLEY DRIVE
POST-DEVELOPMENT STORM DRAINAGE AREA PLAN
 SCALE 1:2000
 DATE APR 2024 JOB 121153 FIGURE 4.1

M:\2021\121153 CAD\Design\121153-STM.dwg, FIG 4.1, Apr 11, 2024 - 11:39am, brundie

SHT11V17 DWG 270mm X 132mm

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 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.

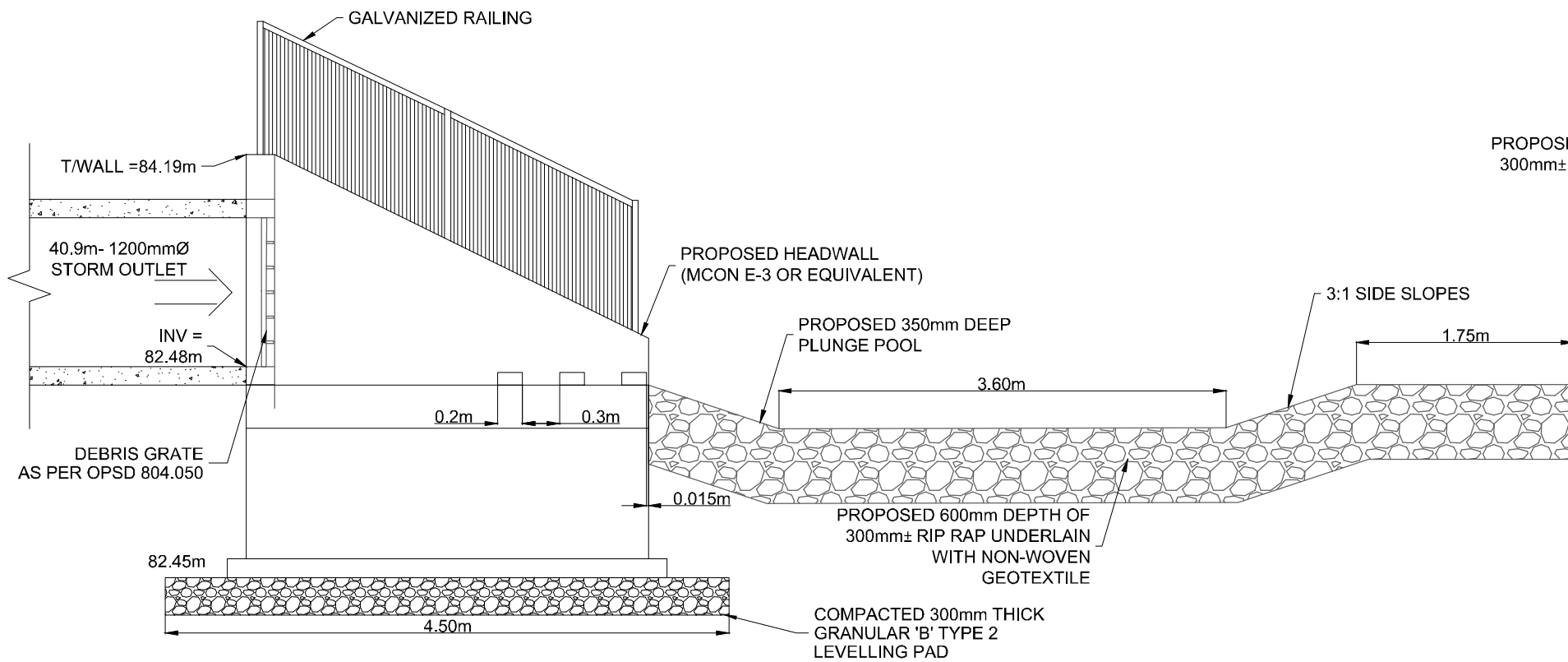
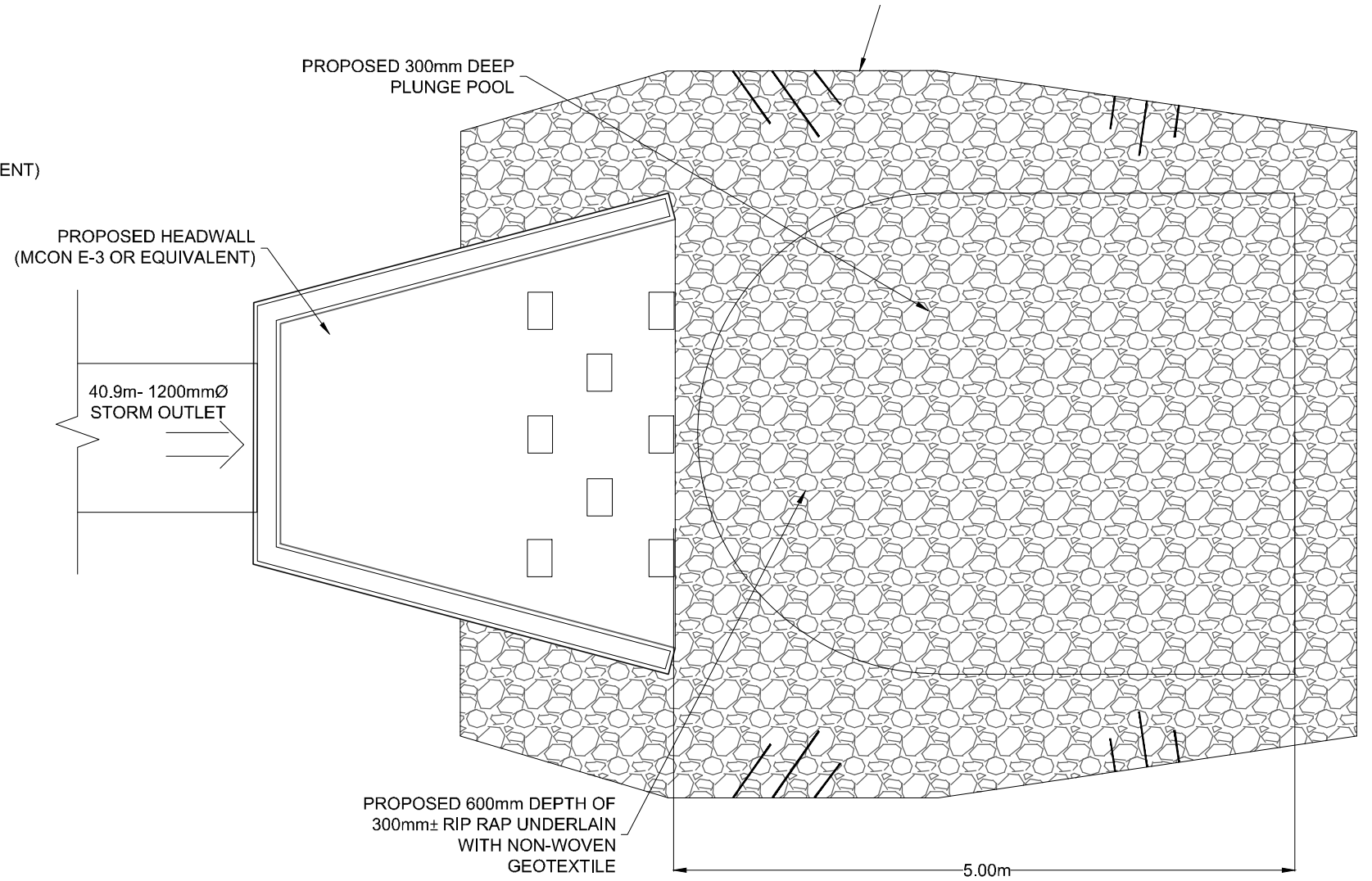
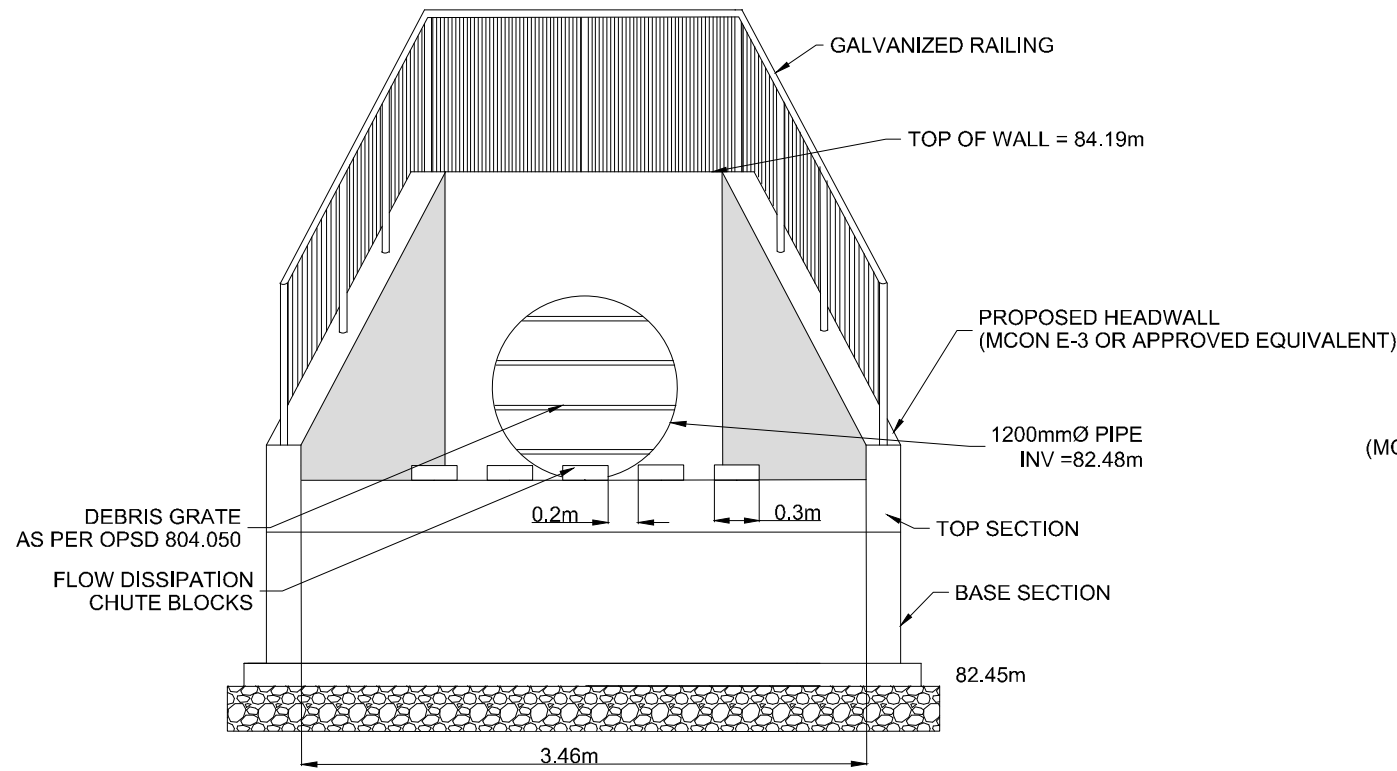
Outlets & Impact

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** 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.



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 RIDEAU VALLEY DRIVE

PROPOSED STORM OUTLET WITH PLUNGE POOL

SCALE 1:50

DATE APR 2024 JOB 121153 FIGURE 121153-OUTLET

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SHT11V17 DWG 270mm x 132mm

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 MHS A58902 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 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 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

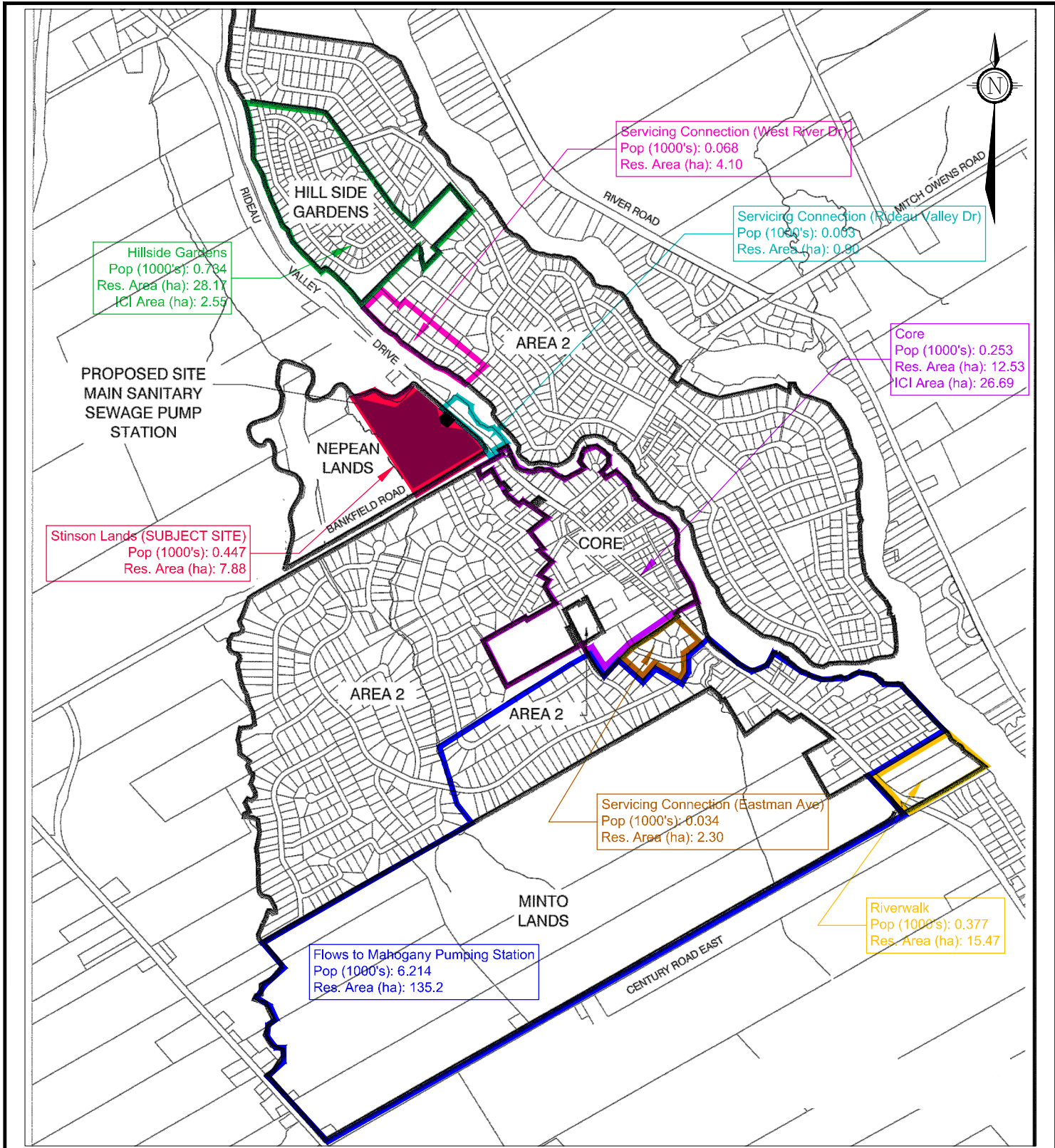
The proposed off-site works will require connecting a 25 m long, 250 mm diameter pipe to an off-site trunk sanitary sewer within the Rideau Valley Drive ROW by constructing a new manhole approximately 37 m upstream of existing MHS A58902. The proposed work will require reinstatement of the existing road to match existing conditions or better.

On-site works

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

5.4 Sanitary Demand and Design Parameters

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



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

MANOTICK PS SERVICING AREAS

STINSON LANDS SUBDIVISION

DATE	JOB	FIGURE
APR 2024	121153	FIG 5.1

M:\2021\121153\CAD\Design\SANI\Manotick PS SAN Areas.dwg, Sheet 8x11 Keyplan, Sep 14, 2023 - 9:48am, v.patel

Table 5.1: Sanitary Sewer Design Parameters

Design Component	Design Parameter
Unit Population:	
Single Detached Home	3.4 people/unit
Semi-Detached / Townhomes	2.7 people/unit
2-BR Apartments	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)

¹A 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.

5.5 Hydraulic Grade Line (HGL)

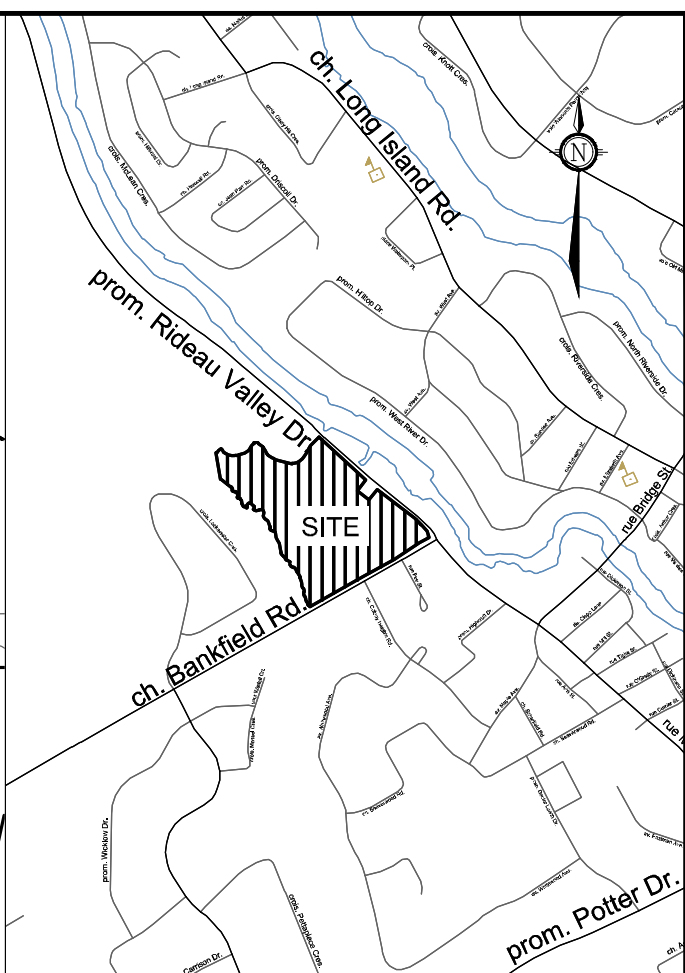
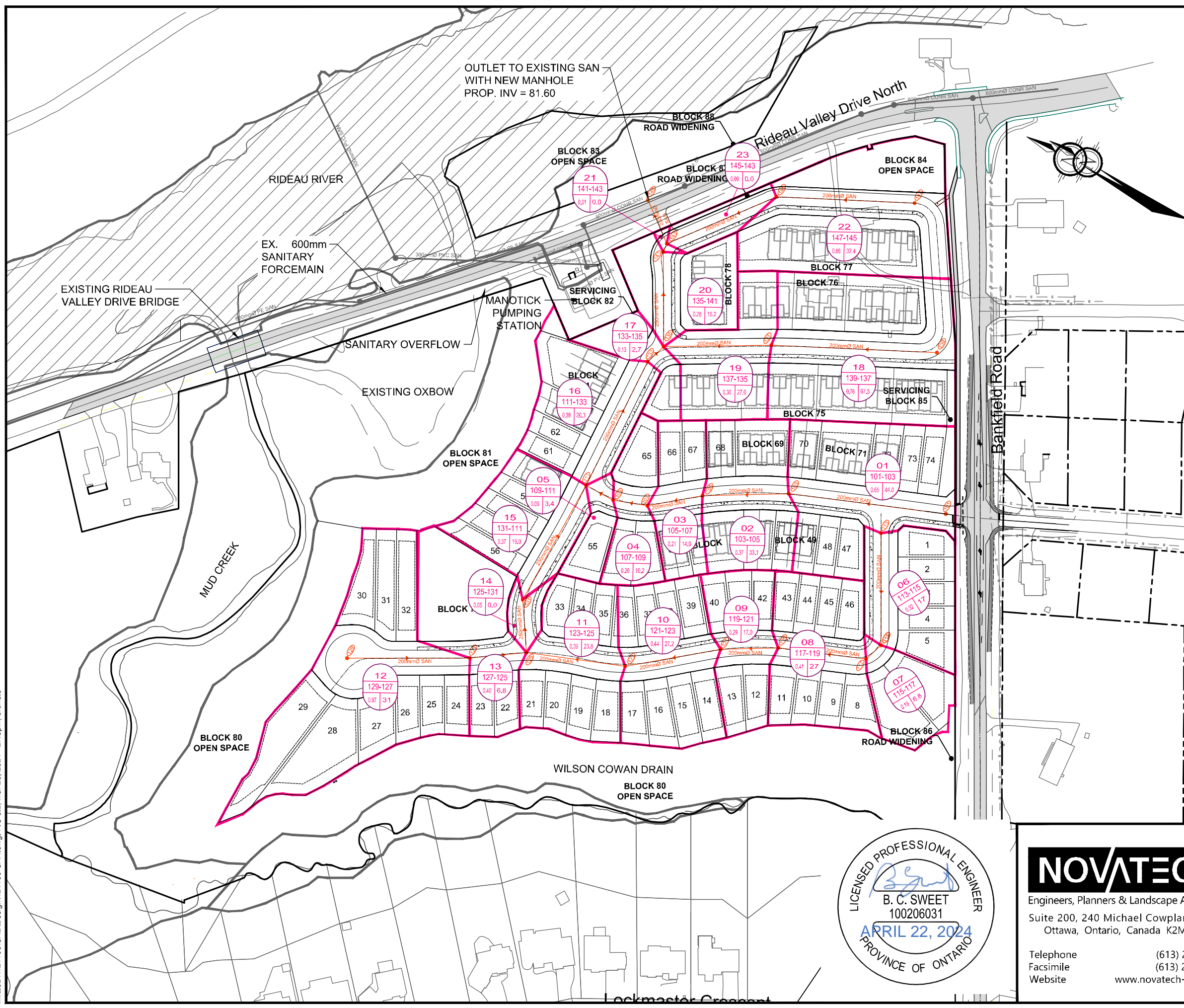
The emergency overflow elevation at the Manotick Pumping Station is located at the by-pass maintenance hole (MHSA58901) within the station's compound which is directed to the Oxbow Ditch. The elevation of the overflow is 83.57m, based on GeoOttawa Mapping, which is set above the 100-year water level 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.

OUTLET TO EXISTING SAN WITH NEW MANHOLE
PROP. INV = 81.60



LEGEND

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

M:\2021\121153\CAD\Design\121153-SAN.dwg, FIG 5.2, Mar 26, 2024 - 2:48pm, 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 RIDEAU VALLEY DRIVE		
SANITARY DRAINAGE AREA PLAN		
SCALE	1:2000	
DATE	APR 2024	FIGURE 5.2
JOB	121153	

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 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 will require approximately 1130 m of 200 mm and 250 mm on-site watermain. Proposed hydrant locations have been provided. An additional fire hydrant has been provided along Street Two's dead-end portion to ensure the required fire flow is available for the furthest lot (lot 29). 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 m² of building area; and
 - b. Min separation of 10 m between the backs of adjacent units.

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

6.4 System Pressure Modeling and Results

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

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

Domestic Demand

The water demand summary for the complete build out of the Subject Site for the average daily and peak hour demands has been provided in **Table 6.2** below. For detailed results refer to the tables provided in **Appendix E**.

The detailed results are also demonstrated in **Figure 6.3** – Maximum Pressures During AVDY Condition and **Figure 6.4** – Minimum Pressures During PKHR Condition.

Table 6.2: 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.

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

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.

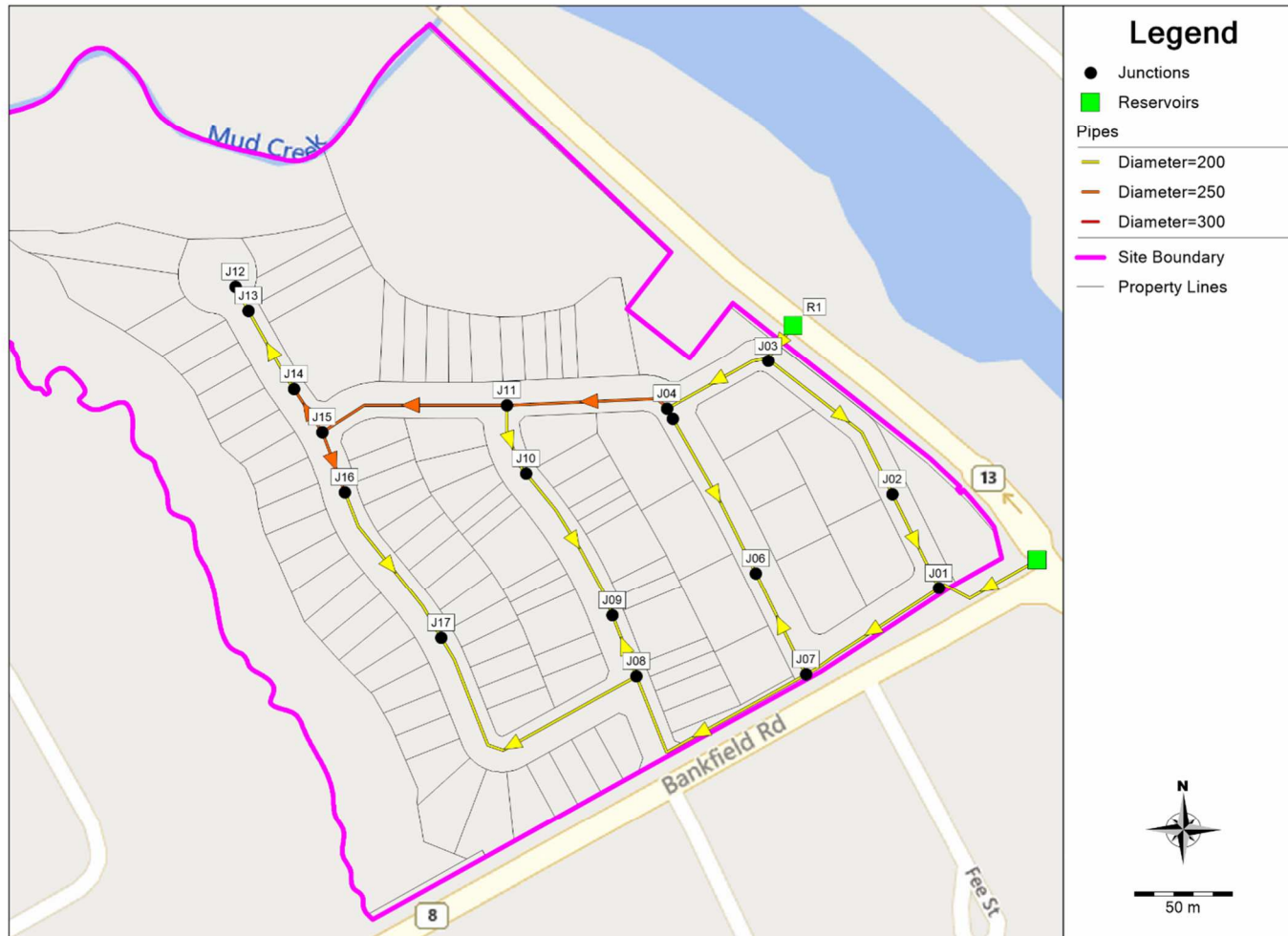


Figure 6.1
Proposed Watermain Sizing, Layout and Junction IDs

Date: 2024/04/08

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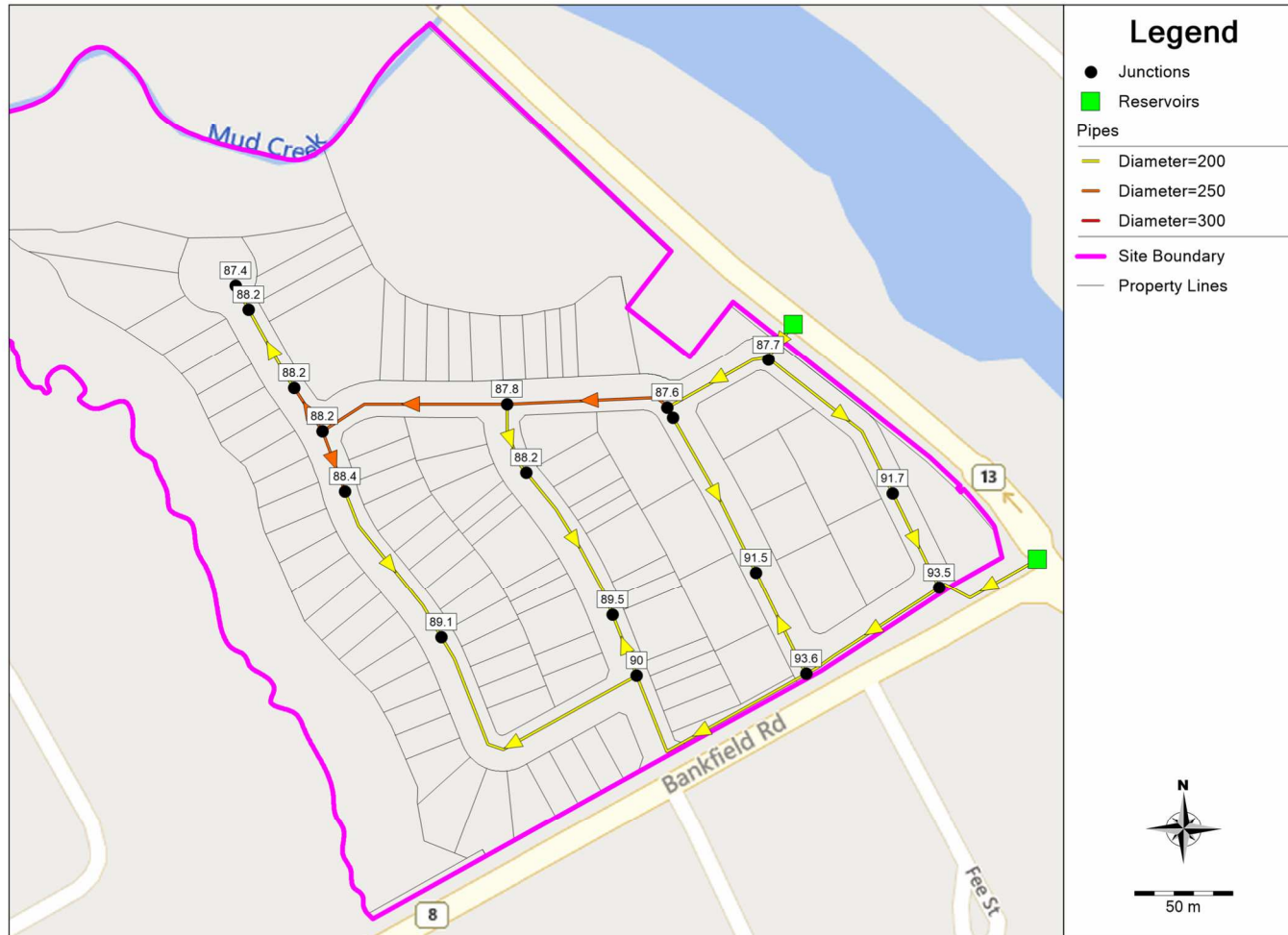


Figure 6.2
Ground Elevations (m)

Date: 2024/04/08

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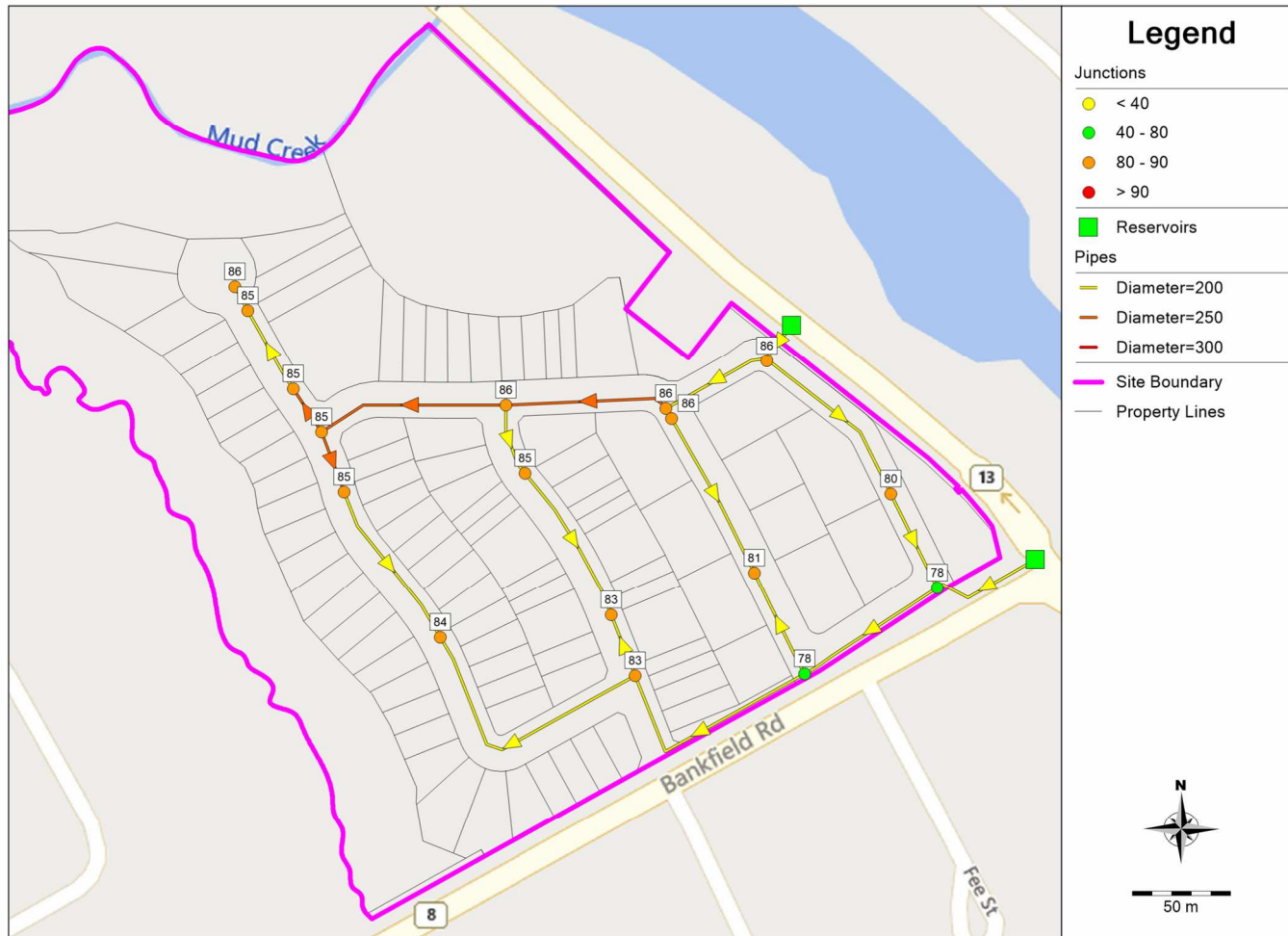


Figure 6.3(i)
Maximum Pressure During AVDY Conditions – Future

Date: 2024/04/08

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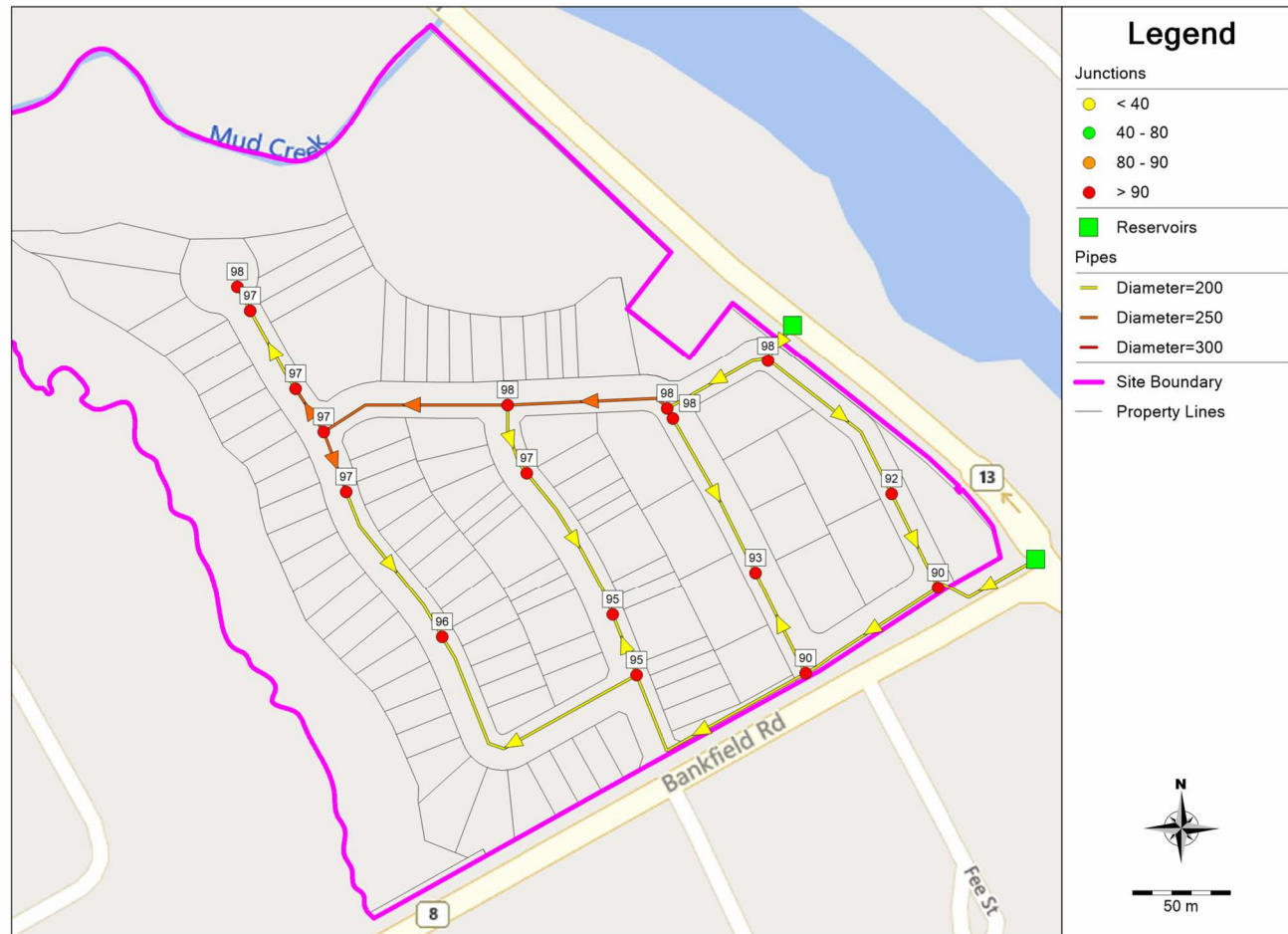


Figure 6.3(ii)
Maximum Pressure During AVDY Conditions – Existing

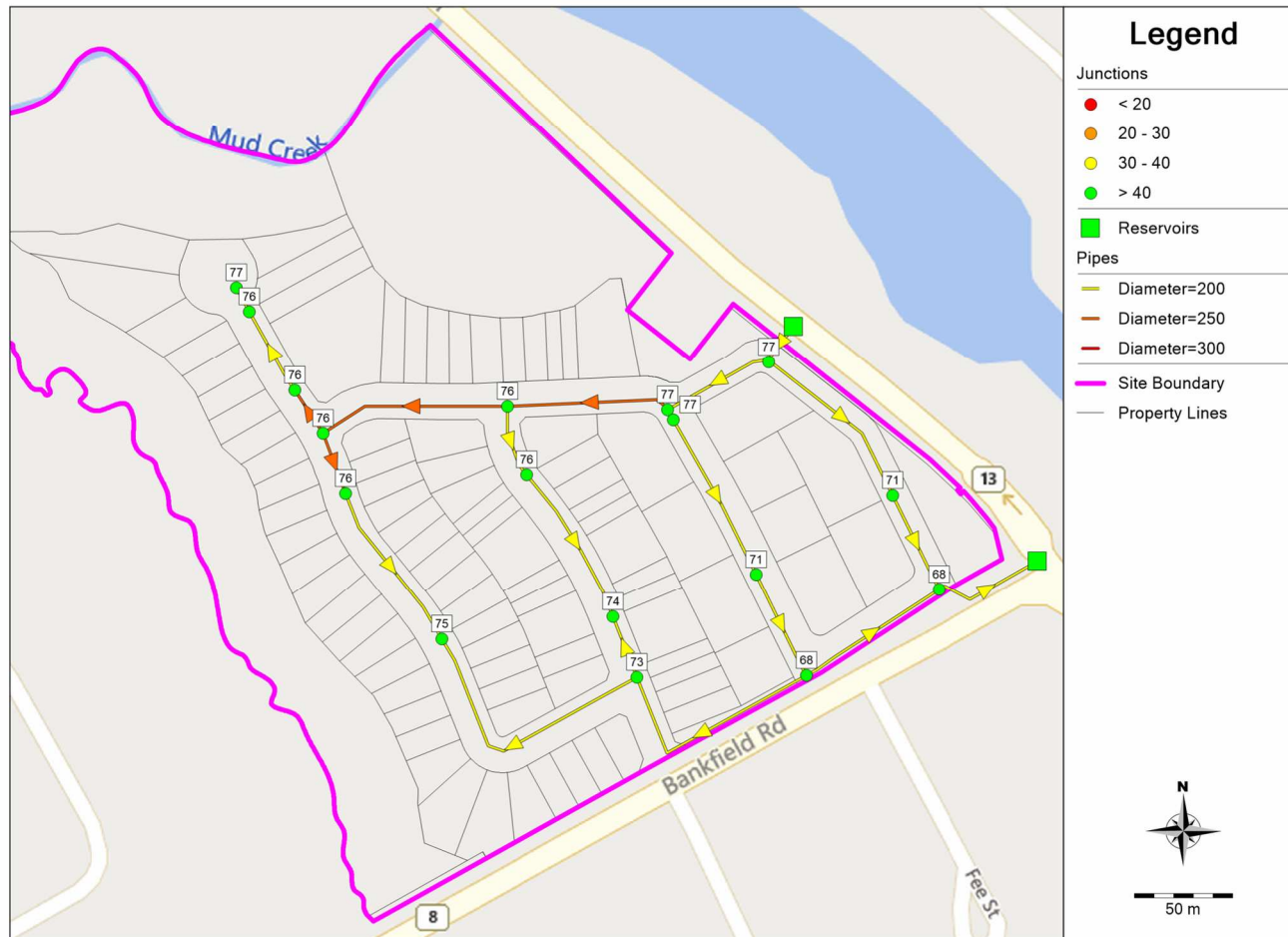


Figure 6.4(i)
Minimum Pressure During PKHR Conditions – Future

**Stinson Lands (121153)
PCSWMM Model Schematic**

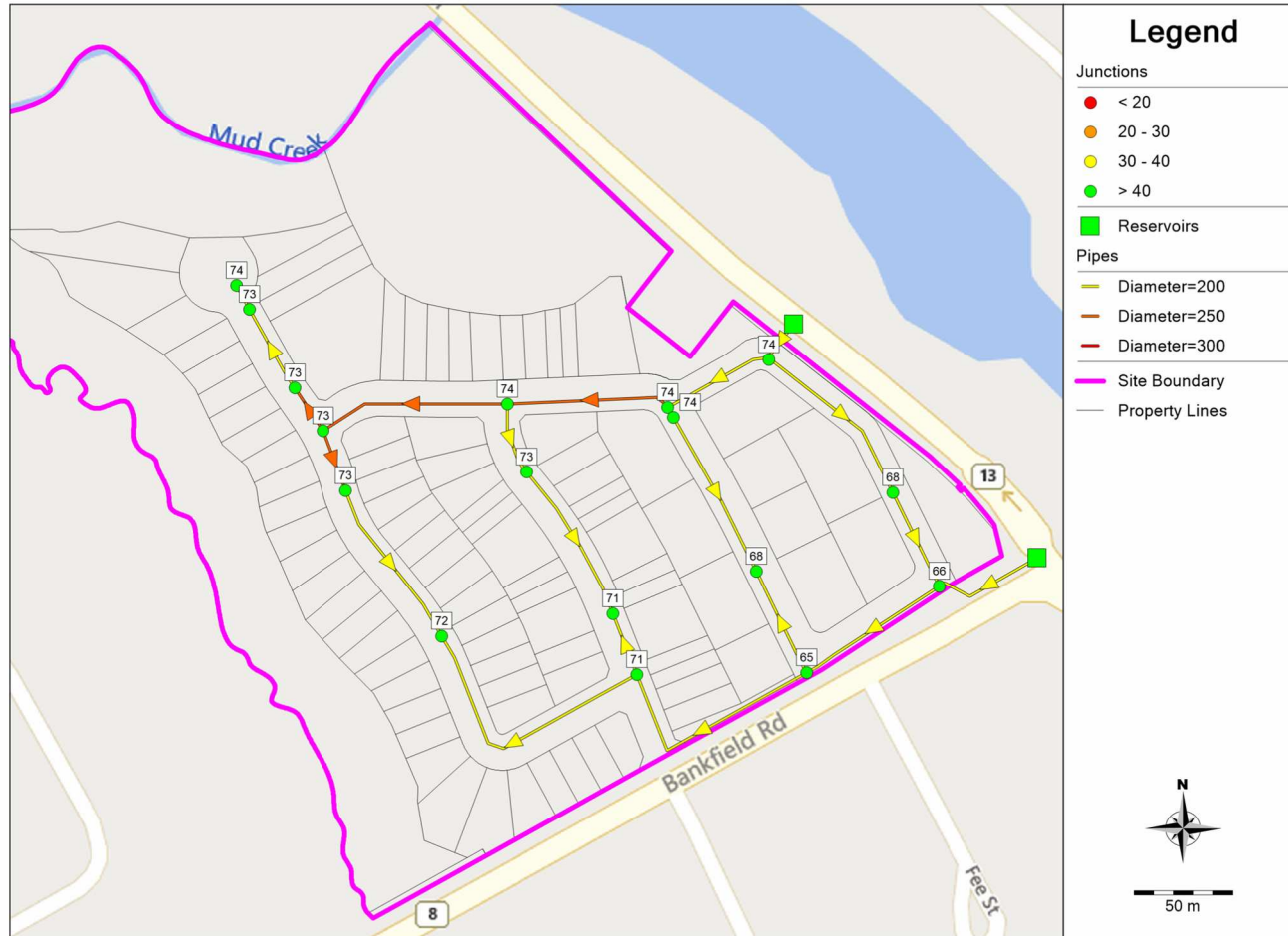


Figure 6.4(ii)
Minimum Pressure During PKHR Conditions – Existing

**Stinson Lands (121153)
PCSWMM Model Schematic**

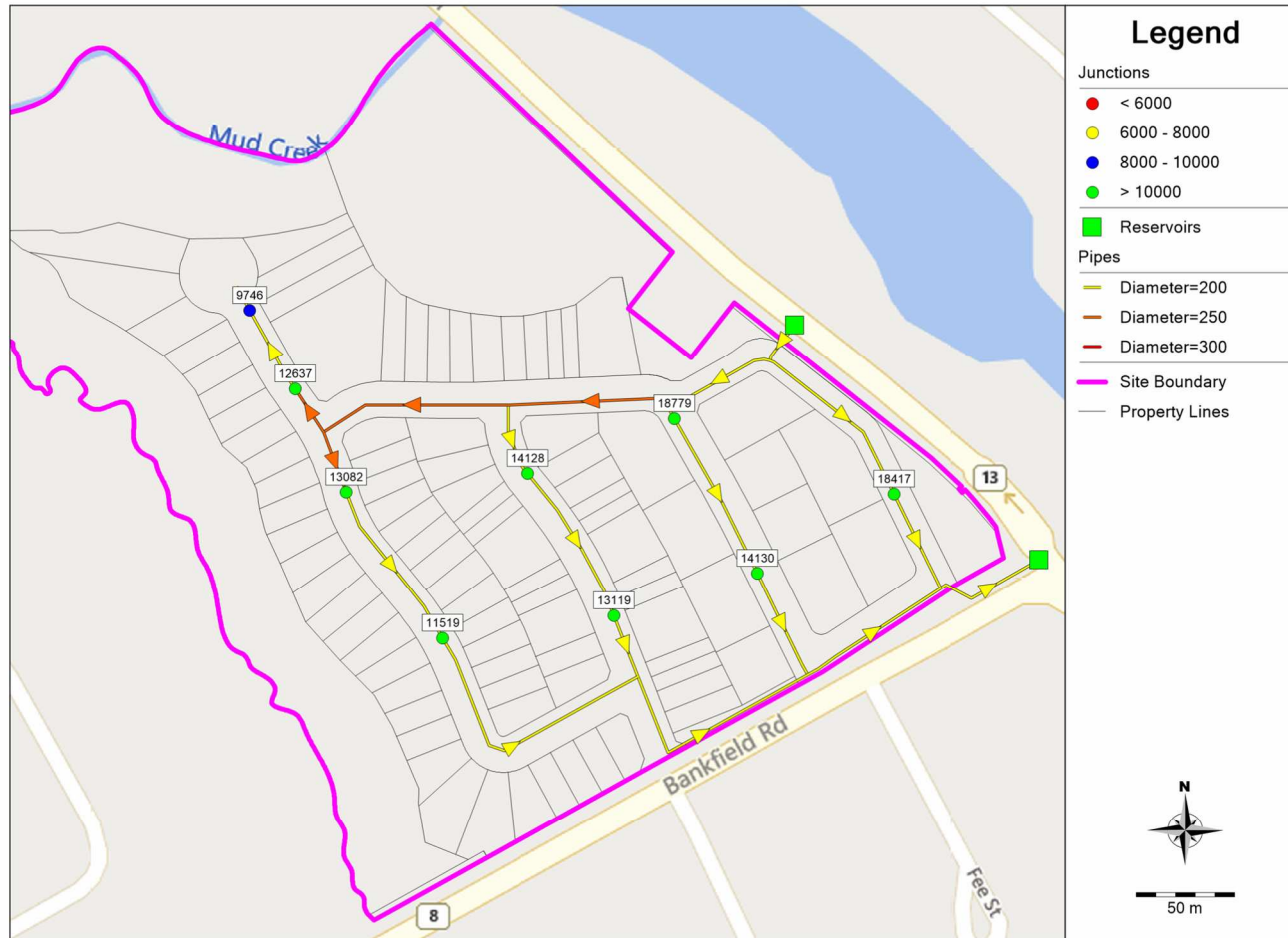


Figure 6.5(i)
Available Flow at 20psi During MXDY+FF Conditions – Future

**Stinson Lands (121153)
PCSWMM Model Schematic**

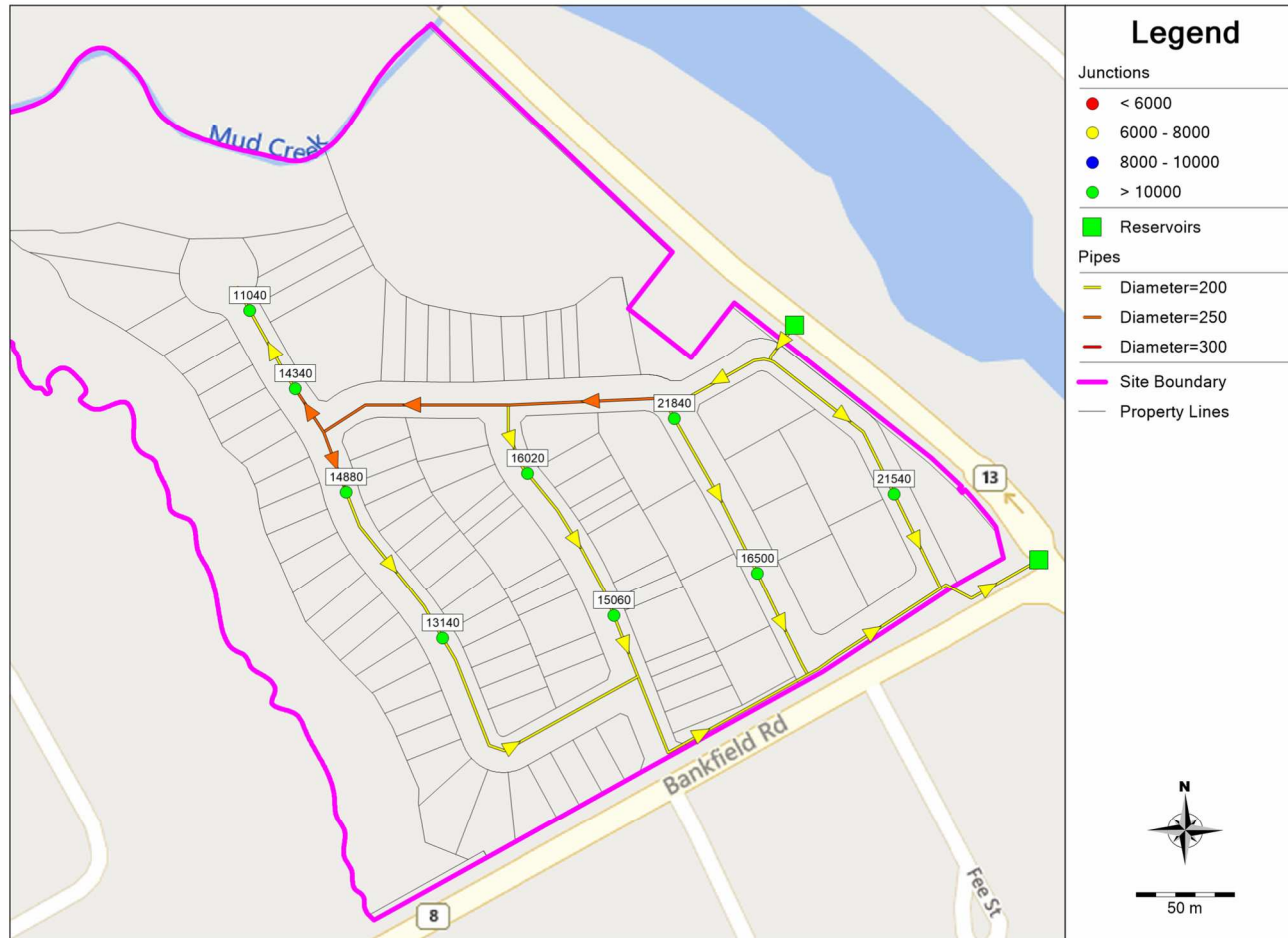


Figure 6.5(ii)
Available Flow at 20psi During MXDY+FF Conditions – Existing

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 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 Subject Site will be serviced with approximately 1075 m of on-site storm sewers ranging from 250 mm to 1200 mm in diameter. The on-site storm sewers will outlet to the 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 82, 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 will require approximately 1000 m of on-site sanitary sewer to collect and direct wastewater flows to the outlet pipe located in the north-east corner of the Subject Site.

Water Supply System

- There will be two connections made to the 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 works will require approximately 1130m of on-site watermain. The location of hydrants will be confirmed during detailed design.

Erosion and Sediment Control and Dewatering Measures

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

Next Steps, Coordination, and Approvals

- MECP PTTW / 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:



Brendan Rundle, B.Eng.
EIT | Land Development

Reviewed by:



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



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



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

Appendix A Correspondence

MEETING NOTES

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

Attendance:

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

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

Post meeting notes are indicated with blue italic text

Action Items are indicated with bold italic text

Description of Discussion	Action
<p>SB provided a summary of the proposed development and stormwater management strategy:</p> <ul style="list-style-type: none"> • SWM Outlet: <ul style="list-style-type: none"> ○ 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 <ul style="list-style-type: none"> ○ 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 <ul style="list-style-type: none"> ○ 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
<ul style="list-style-type: none"> ○ 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 <p>SB requested questions/comments on the proposed SWM Strategy from the other meeting attendees:</p> <p><u>Municipal Drains</u></p> <ul style="list-style-type: none"> ● EH commented on the watershed boundary and hydraulic design: <ul style="list-style-type: none"> ○ There may be an opportunity to incorporate the change to the watershed boundary for Wilson Cowan Drain through an existing report that is being completed for another development. The Mud Creek Municipal Drain is very old and doesn't feel that there is a current need to update its watershed boundary. ○ No major changes to the existing channel design are proposed for either drain; if there are no physical changes needed, EH has no further comments on the hydraulic design. ● AR commented on the culvert extension noting that it needs to meet the level of service for Wilson Cowan Drain and added that he will need to review as part of his report. If changes to the culvert are needed, they could be incorporated under an existing report being prepared, if timing permits. ● EH commented that the proposed Open Space Block would provide adequate space for access to the Wilson Cowan Drain ● AR noted that the existing outlet to Wilson Cowan Drain near lot 5/6 of sketch will need to be filled and that the City will require a relatively flat area to access do maintenance works ● GW confirmed that there is approximately 15m from the top of the slope to the proposed development limit ● AR commented that 15m is relatively narrow for maintenance works ● GW pointed out that there is also access to Wilson Cowan Drain from the other side via the abutting Lockmaster Crescent subdivision ● AR stated that a change in land use triggers a requirement that they produce a Section 65 report; for Wilson Cowan Drain, they may be able to update it as part of an existing report. ● SB stated that Novatech will confirm that the City has a flat enough access to safely operate an excavator for maintenance works ● AR noted that a 5% slope seems reasonable for access ● AR commented on the oxbow outlet stating that rip rap protection should be provided wherever it's tied in to avoid erosion along confluence with Mud Creek ● SB asked whether a Draft Plan submission in late July/early August would work for the engineer's report and schedule of assessments ● EH responded that if the submission is in early enough, it can be updated as part of the existing Section 78 report with Wilson Cowan Drain. ● AR added that the sooner the better, but that it's not a critical timeframe; the present schedule for updating existing reports would occur before one year out and that it's dependent on the drainage information that's received from upstream developments. <p><u>Environment</u></p> <ul style="list-style-type: none"> ● 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. <p><u>Fluvial Geomorphology</u></p> <ul style="list-style-type: none"> ● 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. 	<p>Novatech</p>

Description of Discussion	Action
<ul style="list-style-type: none"> 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. <p><i>**DW announced that he had to leave the meeting at this point **</i></p> <ul style="list-style-type: none"> 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 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 <i>JO scheduled a meeting on June 29th to continue the Fluvial Geomorphological submission requirements</i> 	<p>JO</p>
<p>SB asked if there are any other items to discuss:</p> <p><u>ROW Widths</u></p> <ul style="list-style-type: none"> 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. 	<p>Novatech</p>
<p>Meeting concluded, but Eric Lalande (EL) stayed on with Novatech to get caught up on the above-noted discussion:</p> <ul style="list-style-type: none"> SB provided a brief overview of proposed drainage a development limits EL provided the following comments: <ul style="list-style-type: none"> 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 	<p>EL</p>

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

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.

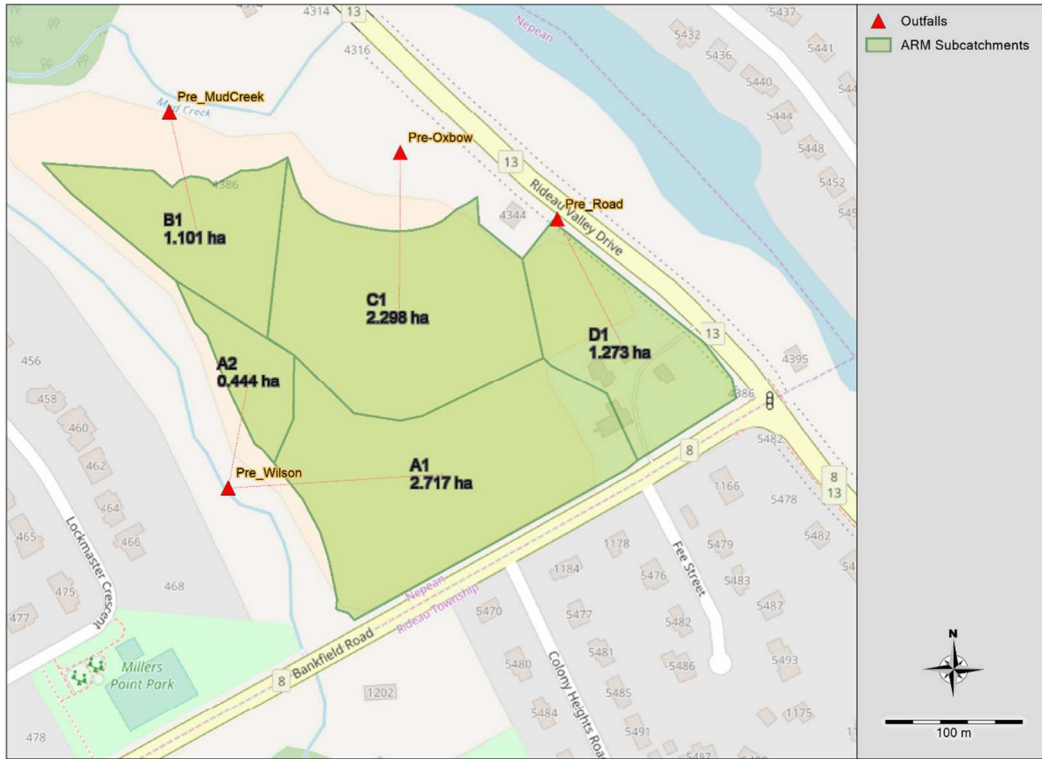


Figure 1: PCSWMM Model Schematic – Existing conditions



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, Vortech, 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.

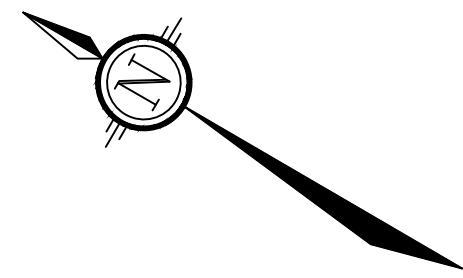
An Enhanced level of water quality treatment will be provided using a combination of lot level and conveyance BMPs, in conjunction with a hydrodynamic separator at the outlet to the Oxbow Ditch. No engineered water quality treatment measures will be required for rear yards and park areas draining directly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

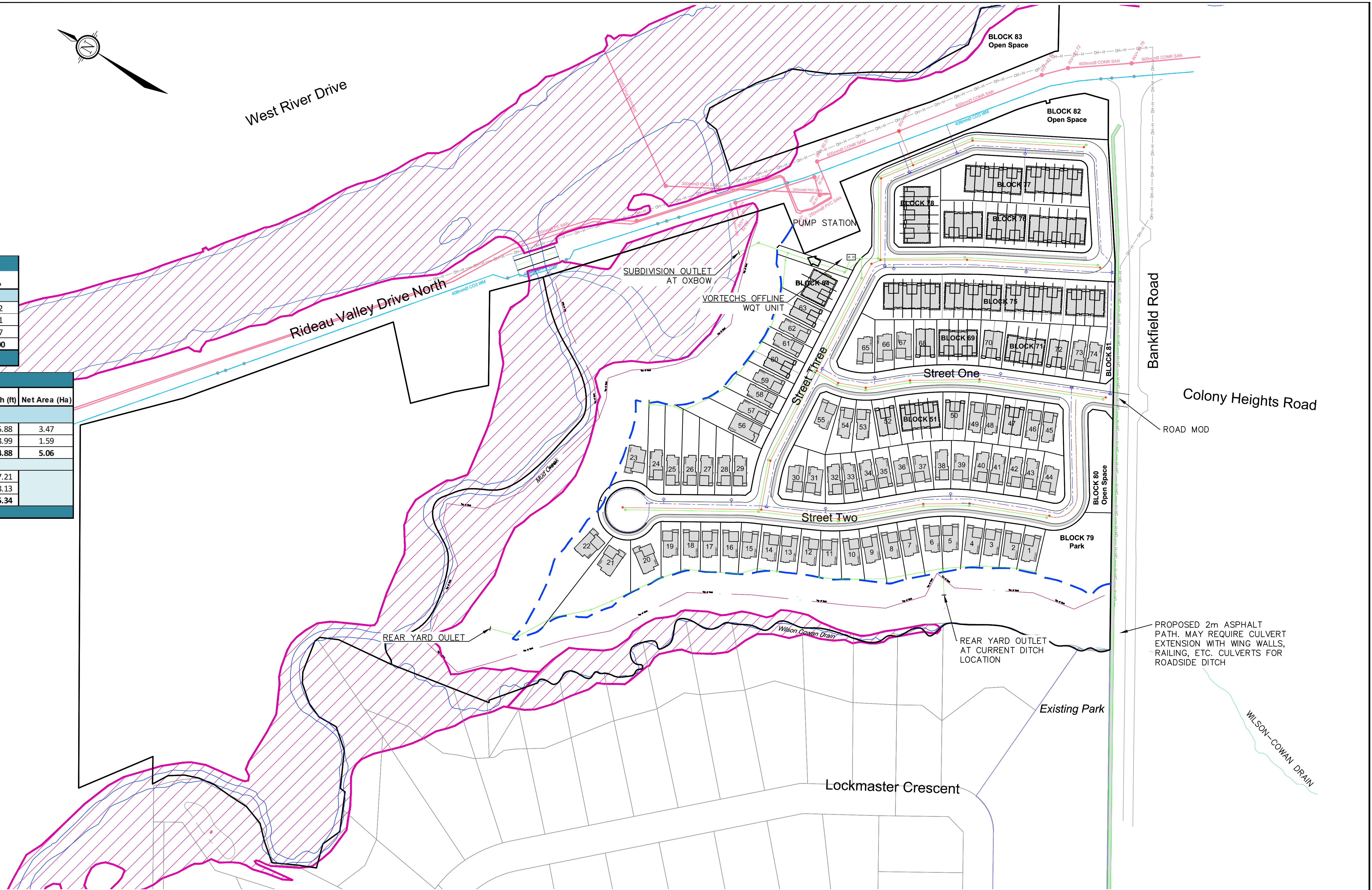
LEGEND

- 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	



PROPOSED 2m ASPHALT PATH. MAY REQUIRE CULVERT EXTENSION WITH WING WALLS, RAILING, ETC. CULVERTS FOR ROADSIDE DITCH

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMANS, 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	FOR REVIEW ONLY
CHECKED: XXXX	
DRAWN: XXX	
CHECKED: wls	
APPROVED: XXXX	
	CHECKED: XXX

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
121153-STINTSON LANDS

DRAWING NAME: **MACRO SERVICING PLAN**

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

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
<p>This meeting was scheduled as a continuation of the geomorphology discussion from the Stormwater Management Strategy meeting that was held on June 22, 2022.</p> <p>The two key items for discussion at this meeting were (1) quantity control and (2) the requirement for a fluvial geomorphology study.</p> <p><u>Quantity Control</u></p> <ul style="list-style-type: none"> 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
<p><u>Quality Control</u></p> <ul style="list-style-type: none"> • 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. <p><u>Fluvial Geomorphological Study Requirements</u></p> <ul style="list-style-type: none"> • 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 Geomorph 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. <p><u>Other Items</u></p> <ul style="list-style-type: none"> • Impact Assessment of adjacent Municipal Depot (4244 Rideau Valley Drive): <ul style="list-style-type: none"> ○ 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: <ul style="list-style-type: none"> ○ 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 <i>"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."</i> 	<p>Novatech</p>

Description of Discussion	Action
<ul style="list-style-type: none"> ○ 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

MEMORANDUM

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 ~7700m³ 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.

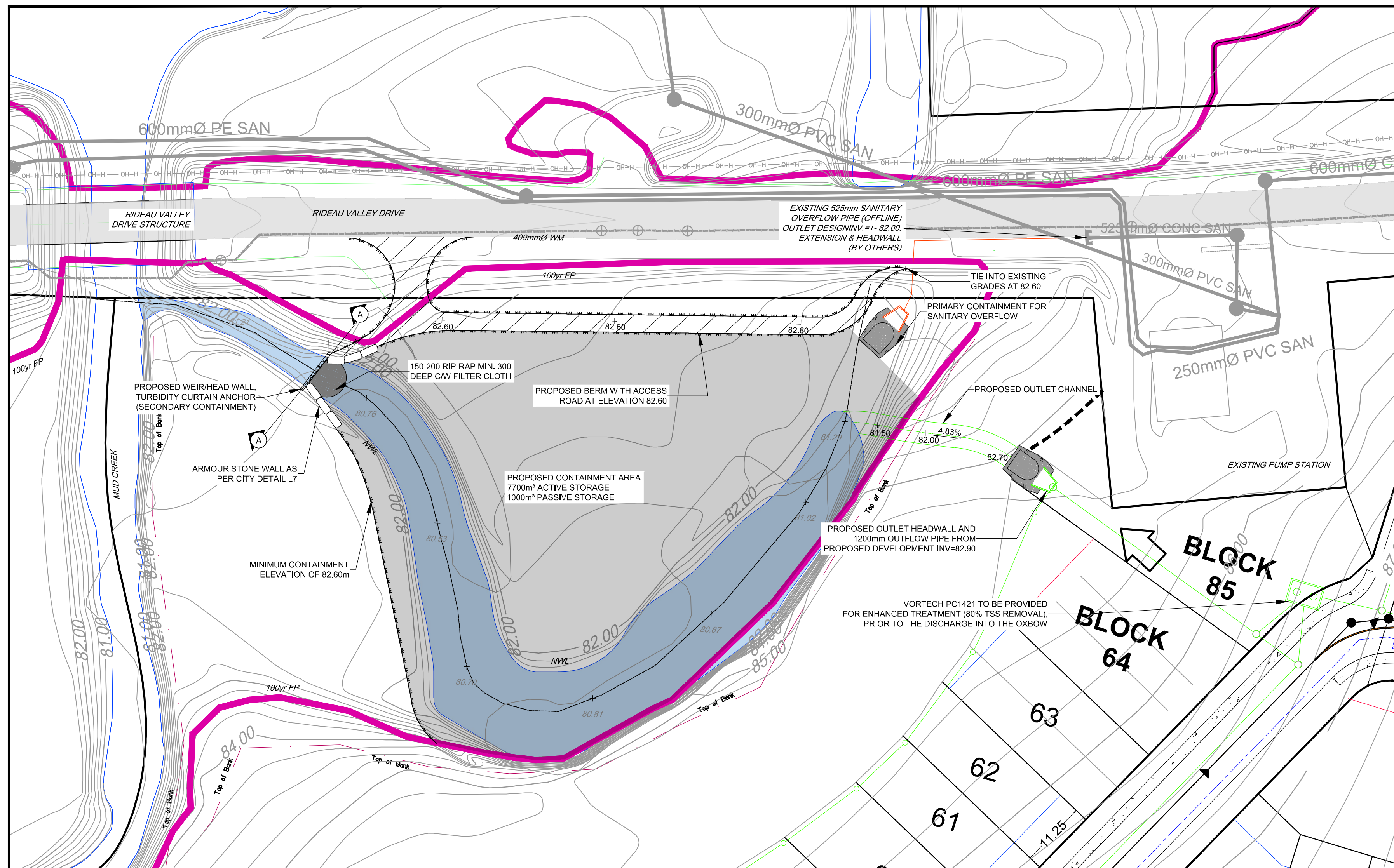
Next Steps and Conclusion

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:

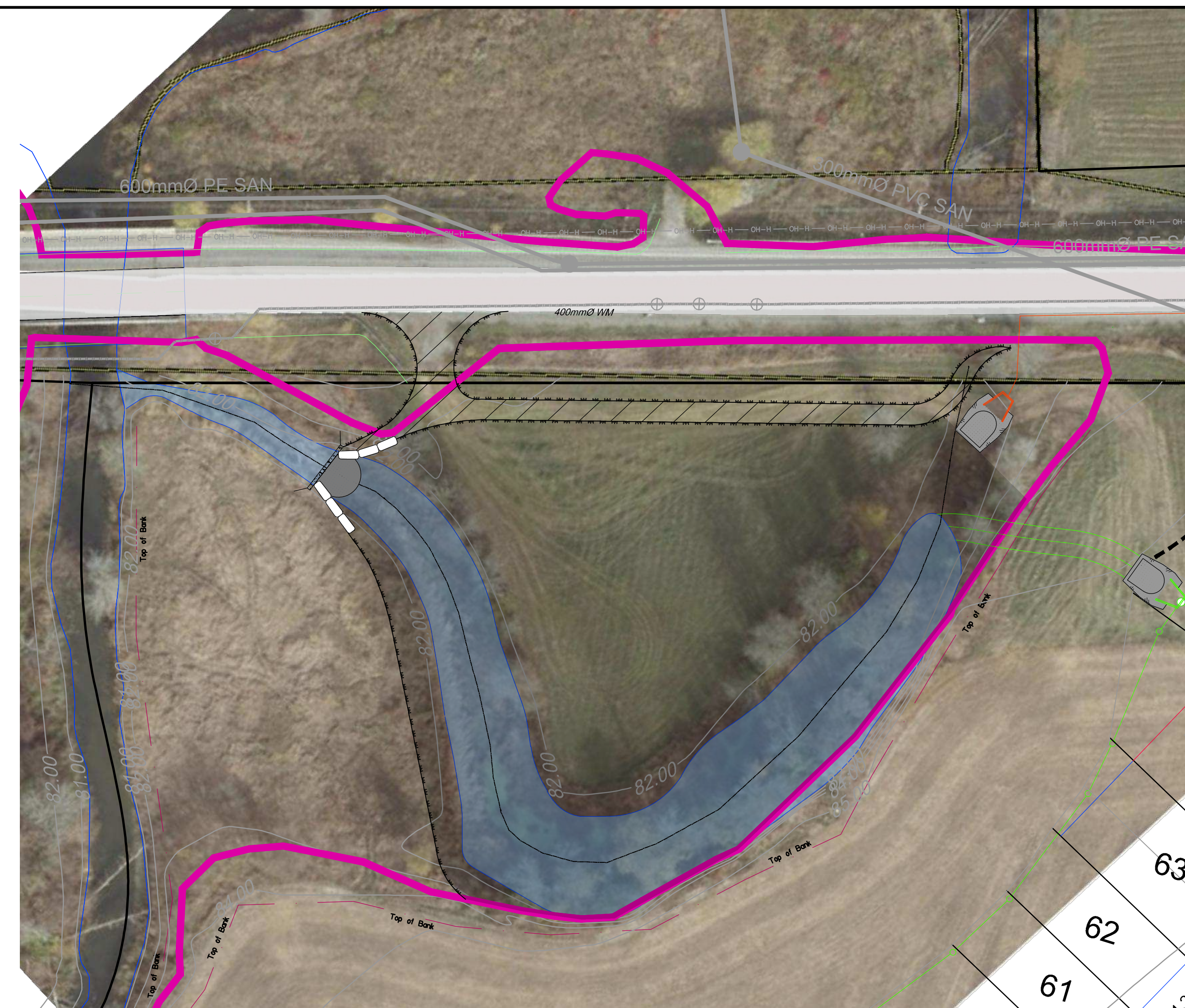
- Agreement in principle of the above approach (after buy-in from MECP and Operations)
- Draft Plan Approval, so we can begin detailed design on behalf of the Proponent.
- Coordinate the detailed design of the Oxbow Headwall between the City and Proponent's Engineers
- Design approvals and permits
- 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.

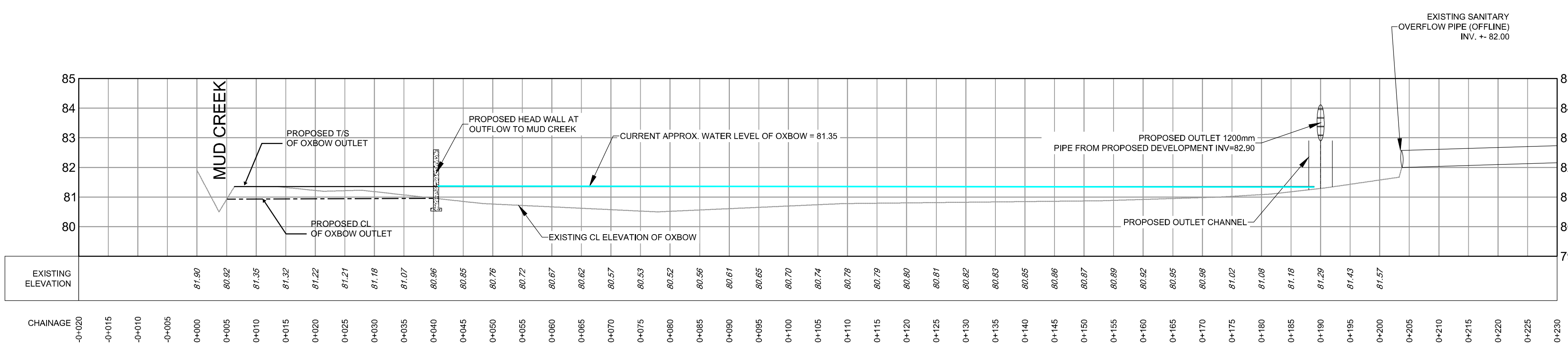
Attach (121153-Oxbow Preliminary Design)



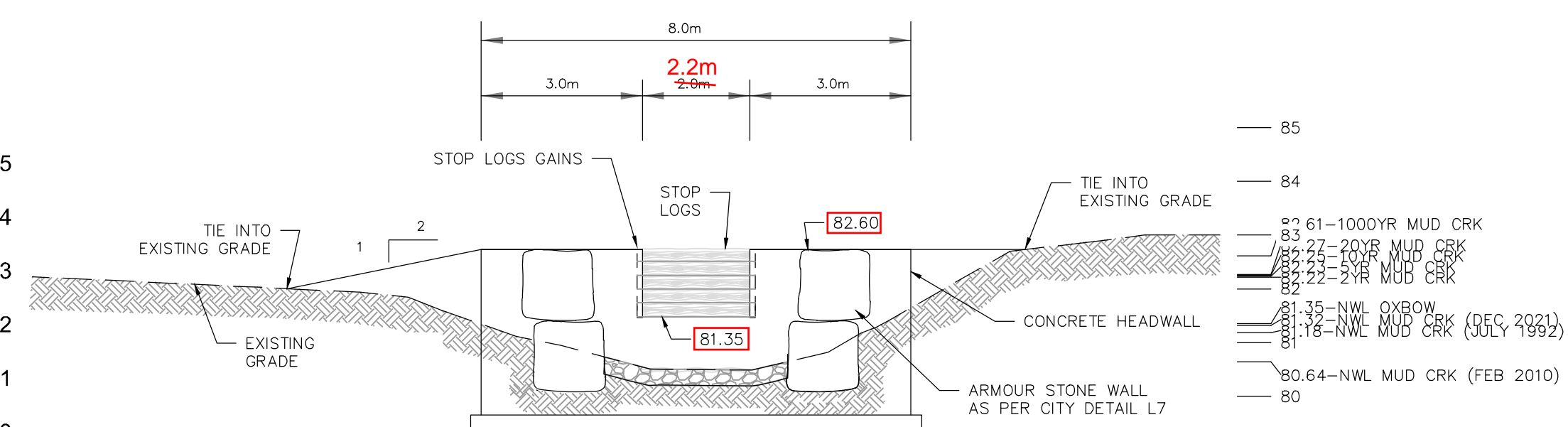
OXBOW - PLAN VIEW
PLAN VIEW - SCALE: 1:500



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



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



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

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

SCALE		DESIGN	
AS SHOWN		CHECKED	BPR
		DRAWN	BHB
		CHECKED	BPR
		APPROVED	BHB
			BHB

No.	REVISION	DATE	BY
1.	PRELIMINARY DESIGN	JUN 09/23	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

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.	121153-0XBOW



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.

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.

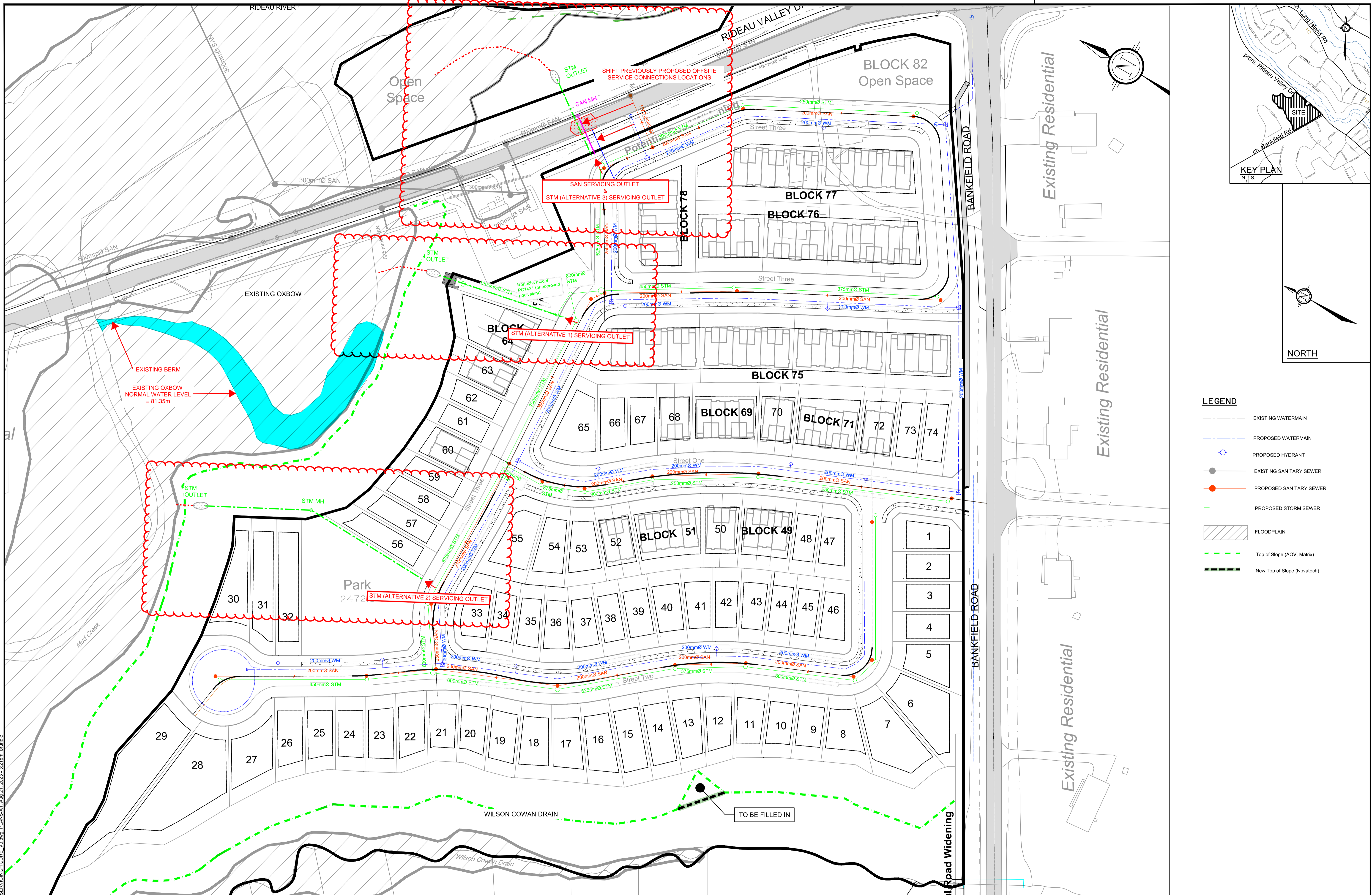
Uniform is prepared to move forward with Alternative 3 despite the cost premium to continue to advance the file, if City Staff can provide buy-in. Alternative 3 would also allow Uniform to carve out the Oxbow lands in advance of subdivision registration pending further discussions/agreement about timing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



NOTE:
 THE POSITION OF ALL POLE LINES, CONDUITS,
 WATERMANS, 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
	BHB
	BPR
	BHB
	BHB
	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
 STINSON LANDS SUBDIVISION

DRAWING NAME
GENERAL PLAN OF SERVICES

PROJECT No.
 121153-00

REV
 REV # 2

DRAWING No.
 121153-GP

M:\2021\121153\CAD\Design\Figures\121153-SERVINGFIGURE_V3.dwg PLANS-A1, Aug 21, 2023 - 3:22 pm, brouillette



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

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.	NA		
Summary of Pre-consultation Meetings with City and other approval agencies.	Y	1	
Reference and confirm conformance to higher level studies and reports (Master Servicing Studies, Environmental Assessments, Community Design Plans), or in the case where it is not in conformance, the proponent must provide justification and develop a defensible 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 feeder mains, 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 - 6.8	

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		

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 Project #: 121153
 Project Name: Stinson Lands
 Date Prepared: 9/6/2022
 Date Revised: 3/25/2024
 Input By: Brendan Rundle
 Reviewed By: Ben Sweet/Sam Bahia
 Drawing Reference: 121153-GPO AND 121153-STM

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

LOCATION			DEMAND													CAPACITY											
STREET	FROM MH	TO MH	AREA						TOTAL AREA (ha)	WEIGHTED RUNOFF COEFFICIENT	INDIVI 2.78 AR	ACCUM 2.78 AR	TIME OF CONC (min.)	RAIN INTENSITY (mm/hr)			TOTAL UNCONTROLLED PEAK FLOW (QDesign) (L/s)	TOTAL RESTRICTED PEAK FLOW (Q) (L/s)	PROPOSED SEWER PIPE SIZING / DESIGN					FULL FLOW CAPACITY (L/s)	FULL FLOW VELOCITY (m/s)	TIME OF FLOW (min.)	QPEAK DESIGN / QFULL (%)
			AREA ID	HIGH DENSITY	ROAD	REAR YARD 1	REAR YARD 2	PARK						2yr	5yr	100yr			LENGTH (m)	SIZE / MATERIAL (mm / type)	ID ACTUAL (m)	ROUGHNESS	DESIGN GRADE (%)				
Street 1	100	102	C13	0.85	0.70	0.50	0.45	0.20	0.25	0.70	0.49	0.49	10.00	76.81		37.37	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	0.49	10.99	73.21		35.62	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	0.70	0.64	1.13	11.53	71.38		80.56	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	1.13	11.83	70.43		79.49	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	0.70	0.43	1.56	11.99	69.93		108.86	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	0.70	0.64	0.64	10.00	76.81		49.32	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	0.64	10.88	73.58		47.25	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	0.64	11.05	72.99		46.87	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	0.70	0.56	1.21	10.88	73.58		88.78	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	0.70	0.54	1.75	11.25	72.33		126.68	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	0.70	0.60	2.35	11.74	70.70		166.48	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	0.70	0.62	0.62	10.00	76.81		47.83	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	0.70	0.35	0.97	11.17	72.57		70.61	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	3.33	12.22	69.21		230.32	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	0.54	1.16	4.49	12.68	67.85		304.79	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	0.56	1.21	7.26	13.64	65.17		472.88	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	0.70	0.23	7.49	15.00	61.77		462.63	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	0.70	0.41	0.41	10.00	76.81		31.39	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	0.56	0.89	1.30	10.46	75.08		97.27	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	1.30	10.93	73.41		95.10	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	0.70	0.68	0.68	10.00	76.81		52.31	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	0.70	0.64	1.32	10.50	74.93		99.15	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	0.70	0.37	9.18	15.13	61.46		564.38	564.4	41.1	825 CONC	0.838	0.013	1.00	1497.5	2.71	0.25	37.7%	
									0.00		0.00	10.48	15.38	60.88		637.90											

STORM SEWER DESIGN SHEET

LOCATION			DEMAND													CAPACITY										
STREET	FROM MH	TO MH	AREA						TOTAL AREA (ha)	WEIGHTED RUNOFF COEFFICIENT	INDIVI 2.78 AR	ACCUM 2.78 AR	TIME OF CONC (min.)	RAIN INTENSITY (mm/hr)			TOTAL UNCONTROLLED PEAK FLOW (QDesign) (L/s)	TOTAL RESTRICTED PEAK FLOW (Q) (L/s)	PROPOSED SEWER PIPE SIZING / DESIGN							
			AREA ID	HIGH DENSITY	ROAD	REAR YARD 1	REAR YARD 2	PARK						2yr	5yr	100yr			PEAK FLOW (L/s)	LENGTH (m)	SIZE / MATERIAL (mm / type)	ID ACTUAL (m)	ROUGHNESS	DESIGN GRADE (%)	FULL FLOW CAPACITY (L/s)	FULL FLOW VELOCITY (m/s)
Easement Block	140	142	-	0.85	0.70	0.50	0.45	0.20	0.00	0.00	0.00	15.38			0.00	637.9		5.1	1050 CONC	1.067	0.013	1.00	2848.8	3.19	0.03	22.4%
	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%
	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	15.46			0.00											

DEMAND EQUATION
 $Q = 2.78 \text{ AIR}$
 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
 $Q_{full} = (1/n) A R^{2/3} S_o^{1/2}$
 Where : Q full = Capacity (L/s)
 n = Manning coefficient of roughness (0.013)
 A = Flow area (m²)
 R = Wetted perimeter (m)
 S_o = Pipe Slope/gradient

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

Time to Peak Calculations

(Uplands Overland Flow Method)

Existing Conditions

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

TOTAL: 7.83

Weighted Curve Number Calculations

Soil type Silty Clay = D

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

Weighted IA Calculations

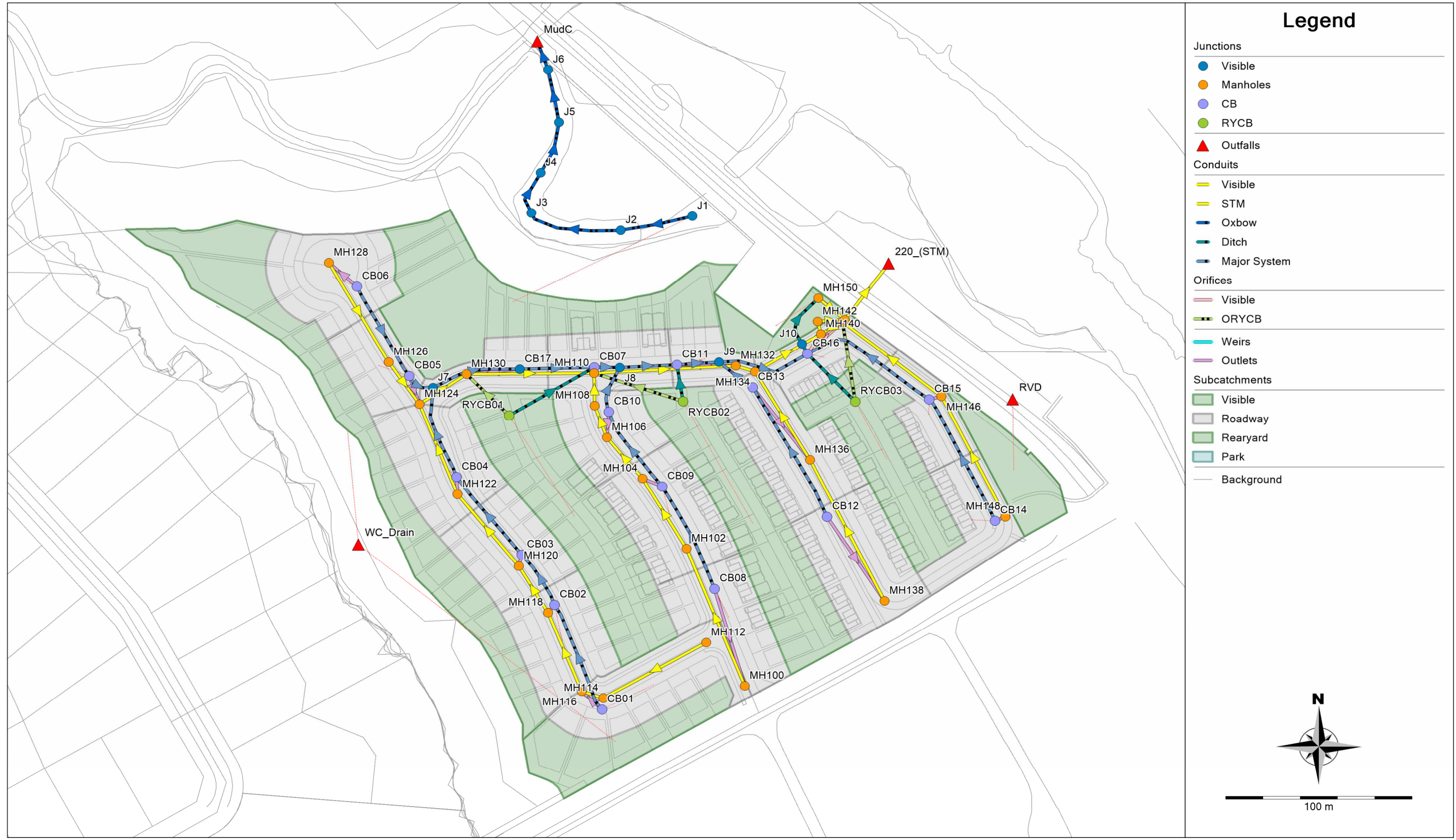
Area ID	Land Use 1	Area	S	IA	Land Use 2	Area	S	IA	Land Use 3	Area	S	IA	Weighted IA
A1	Building & Roads	4%	41.35	6.20	Tree Farm	1%	55.76	8.36	Row Crops	95%	31.39	6.28	6.32
A2	Building & Roads	0%	41.35	6.20	Tree Farm	0%	55.76	8.36	Row Crops	100%	31.39	6.28	6.28
B1	Building & Roads	0%	41.35	6.20	Tree Farm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



Stinson Lands
Post-Development Model Parameters

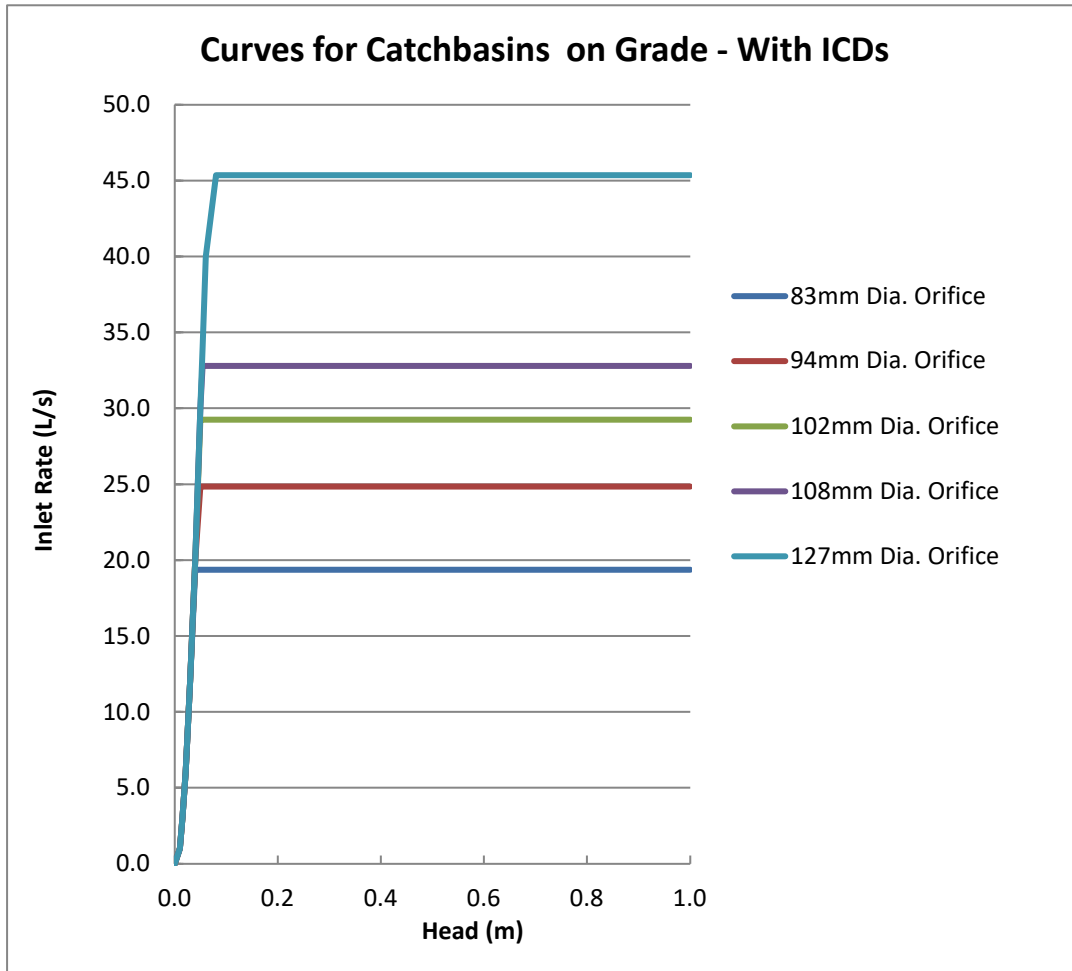
Area ID	Catchment Area (ha)	Runoff Coefficient (C)	Percent Impervious (%)	No Depression (%)	Flow Path Length (m)	Equivalent Width (m)	Average Slope (%)
A-01	0.240	0.45	36%	100%	25.02	97.54	1.0%
B-01	0.710	0.45	36%	100%	21.31	334.06	1.0%
C-01	0.330	0.70	71%	45%	20.51	161.84	1.0%
C-02	0.290	0.70	71%	45%	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



Legend

Junctions	
● Visible	Blue circle
● Manholes	Orange circle
● CB	Purple circle
● RYCB	Green circle
▲ Outfalls	Red triangle
Conduits	
— Visible	Yellow line
— STM	Orange line
— Oxbow	Blue line
— Ditch	Green line
— Major System	Dark blue line
Orifices	
— Visible	Red dashed line
— ORYCB	Green dashed line
— Weirs	Cyan line
— Outlets	Purple line
Subcatchments	
Visible	Green fill
Roadway	Grey fill
Rearyard	Light green fill
Park	Light blue fill
Background	White fill



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_g) 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_g) 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_a) 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_g) 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_a vs. Q_c) varies significantly with road grade, flow depth vs. captured flow (D vs. Q_c) does not.

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

Inlet Control Devices

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

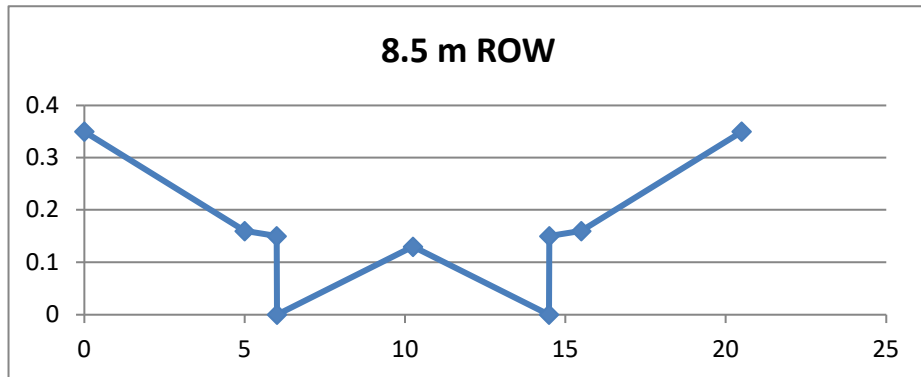
Stinson Lands
HGL Elevations

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

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

Stinson Lands
Design Storm Time Series Data
SCS Design Storms



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

Stinson Lands
Design Storm Time Series Data
SCS Design Storms



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

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

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full 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

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

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

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

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

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
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Stinson Lands

100-year 6-hr Chicago Storm

Model Output

	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	0.8127	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
*****
    
```

Peak Runoff	Runoff Coeff	Total Precip	Total Runon	Total Evap	Total Infil	Imperv Runoff	Perv Runoff	Total Runoff	Total Runoff
Subcatchment		mm	mm	mm	mm	mm	mm	mm	10^6 ltr
A-01	0.581	82.32	0.00	0.00	34.54	29.68	18.17	47.84	0.12
B-01	0.583	82.32	0.00	0.00	34.36	29.68	18.36	48.03	0.34
C-01	0.808	82.32	0.00	0.00	15.28	57.90	8.62	66.52	0.22
C-02	0.807	82.32	0.00	0.00	15.32	57.90	8.57	66.47	0.19
C-03	0.808	82.32	0.00	0.00	15.31	57.90	8.58	66.48	0.18
C-04	0.808	82.32	0.00	0.00	15.31	57.90	8.58	66.48	0.21
C-05	0.808	82.32	0.00	0.00	15.31	57.90	8.59	66.49	0.12
C-06	0.806	82.32	0.00	0.00	15.43	57.89	8.46	66.35	0.21
C-07	0.560	82.32	0.00	0.00	36.22	29.66	16.47	46.14	0.32
C-08	0.808	82.32	0.00	0.00	15.30	57.90	8.59	66.49	0.11
C-09	0.808	82.32	0.00	0.00	15.30	57.90	8.60	66.50	0.14
C-10	0.808	82.32	0.00	0.00	15.30	57.90	8.59	66.49	0.15
C-11	0.585	82.32	0.00	0.00	34.24	29.68	18.47	48.15	0.29
C-12	0.808	82.32	0.00	0.00	15.27	57.90	8.63	66.53	0.22
C-13	0.808	82.32	0.00	0.00	15.31	57.90	8.58	66.48	0.17
C-14	0.588	82.32	0.00	0.00	33.98	29.68	18.74	48.41	0.27
C-15	0.808	82.32	0.00	0.00	15.30	57.90	8.60	66.50	0.22

C-16	0.808	82.32	0.00	0.00	15.29	57.90	8.60	66.50	0.23
C-17	0.808	82.32	0.00	0.00	15.30	57.90	8.59	66.49	0.08
C-18	0.808	82.32	0.00	0.00	15.29	57.90	8.60	66.50	0.12
C-19	0.588	82.32	0.00	0.00	33.96	29.68	18.76	48.43	0.19
C-20	0.576	82.32	0.00	0.00	34.40	29.11	18.31	47.43	0.06
C-21	0.808	82.32	0.00	0.00	15.26	57.90	8.63	66.54	0.11
C-22	0.808	82.32	0.00	0.00	15.26	57.90	8.63	66.54	0.14
D-01	0.338	82.32	0.00	0.00	54.51	0.00	27.84	27.84	0.05

Node Depth Summary

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth 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

Node	Type	Maximum Lateral Inflow LPS	Maximum Total Inflow LPS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error 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 Surcharging Summary

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim 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

Outfall Node	Flow Freq Pcnt	Avg Flow LPS	Max Flow LPS	Total Volume 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

Link	Type	Maximum Flow LPS	Time of Max Occurrence days hr:min	Maximum Veloc m/sec	Max/ Full Flow	Max/ Full 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

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			

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

Conduit	Adjusted /Actual Length	Fraction of Time in Flow Class								
		Up Dry	Down Dry	Sub Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet 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

Conduit	Hours Full			Hours	Hours
	Both Ends	Upstream	Dnstream	Above Full Normal Flow	Capacity 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

**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 Ratio¹ =
$$\frac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(14.3 \text{ m}^2)} = 0.79$$

Rainfall Intensity mm/hr	Operating Rate² % of capacity	Flow Treated (l/s)	% Total Rainfall Volume ³	Rmvl. Effic⁴ (%)	Rel. Effic⁵ (%)
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 Adjustment⁵ = 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/m².

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

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

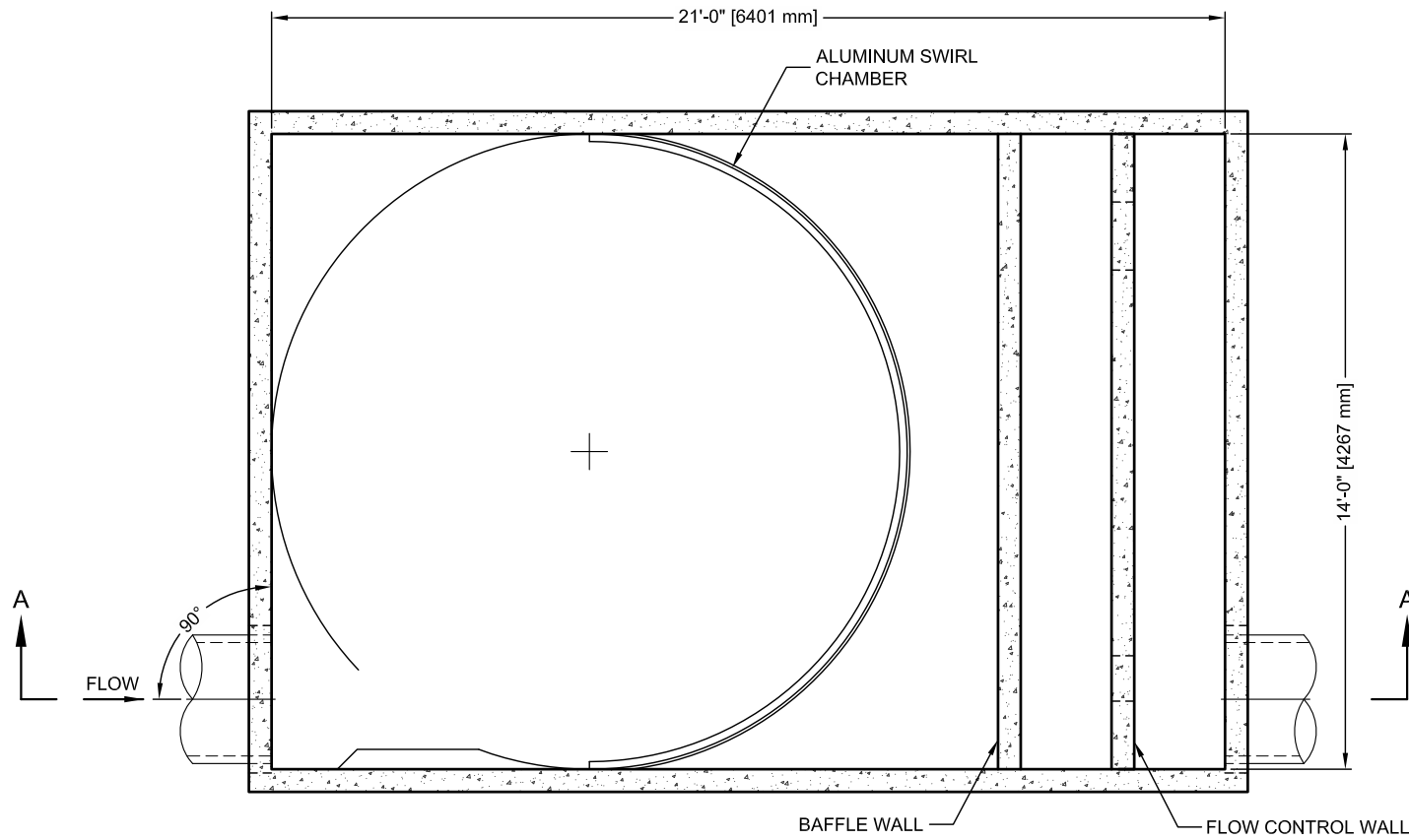
5- Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.

Calculated by: JAK 7/26 | Checked by:

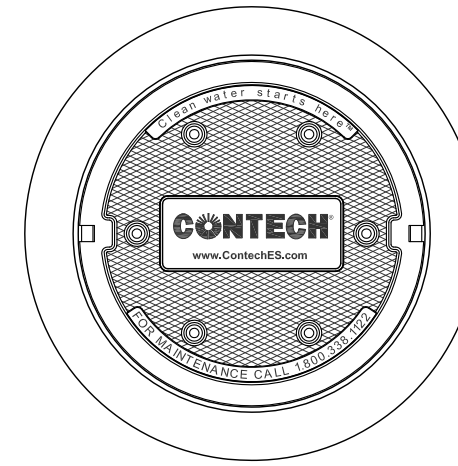
VORTECHS PC1421 DESIGN NOTES

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

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

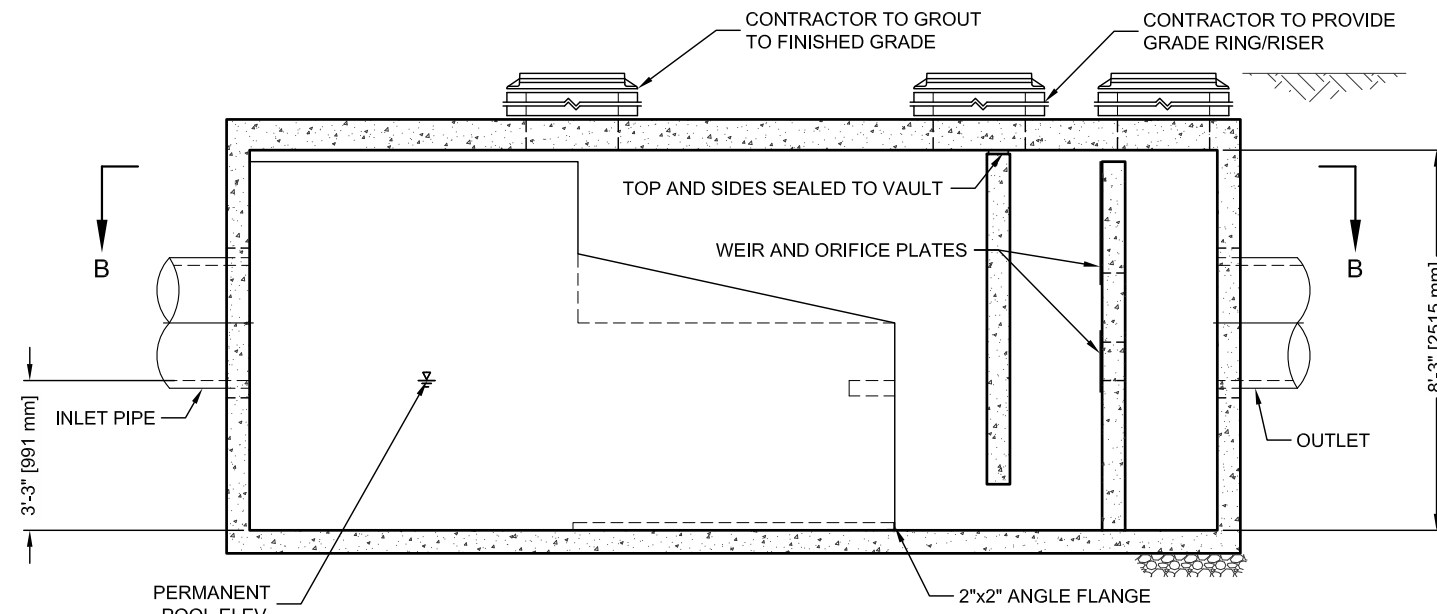


SECTION B-B



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			



SECTION A-A

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 PERPENDICULAR 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 ACCOMMODATE 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.

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



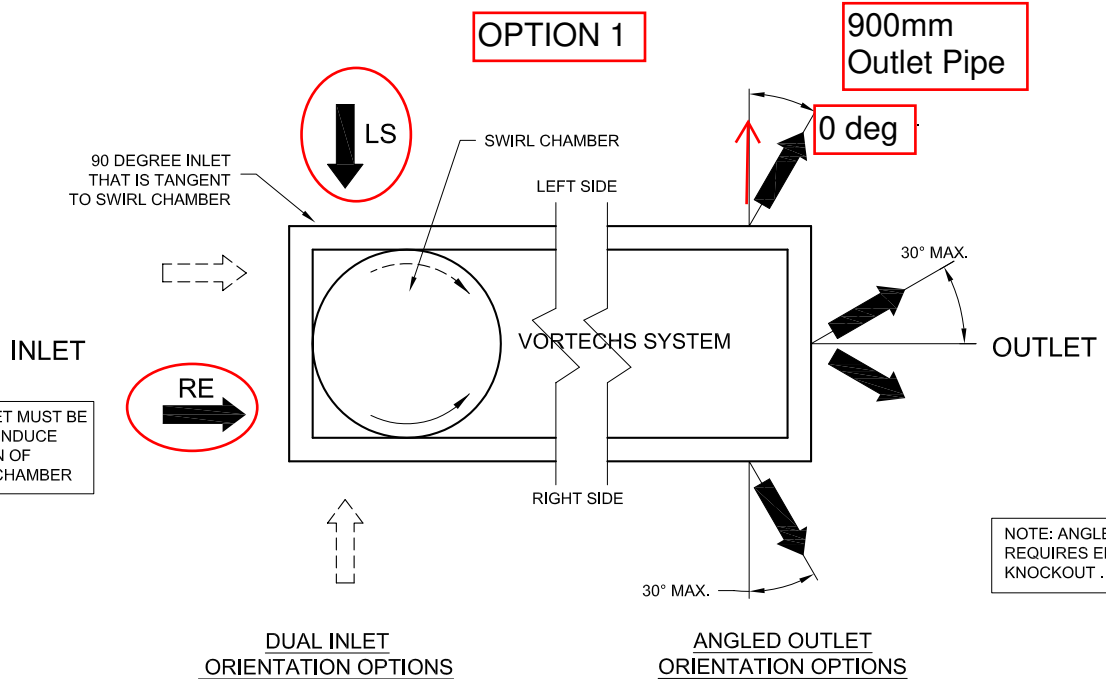
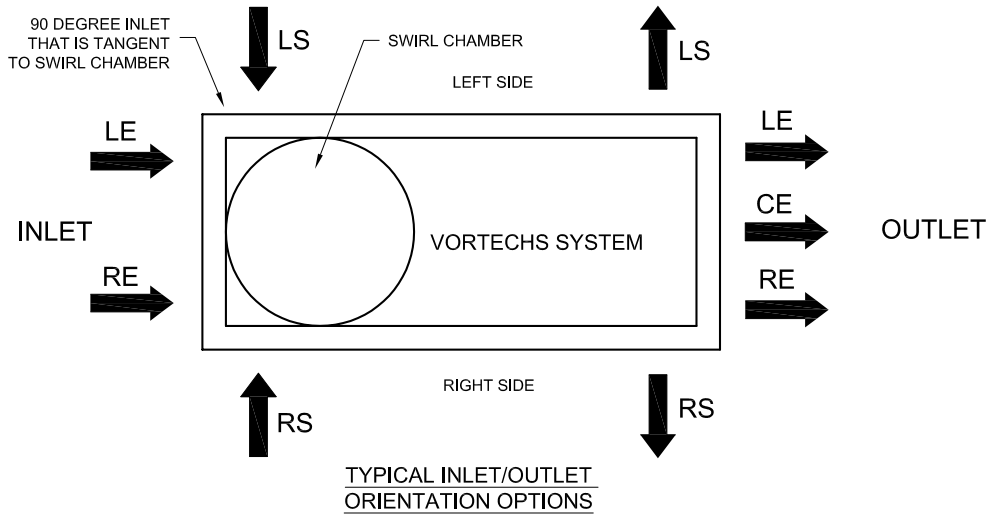
THIS PRODUCT MAY BE PROTECTED BY THE FOLLOWING
U.S. PATENT: 5,759,415; RELATED FOREIGN PATENTS.



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

VORTECHS PC1421
STANDARD DETAIL

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



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

ORIENTATION KEY

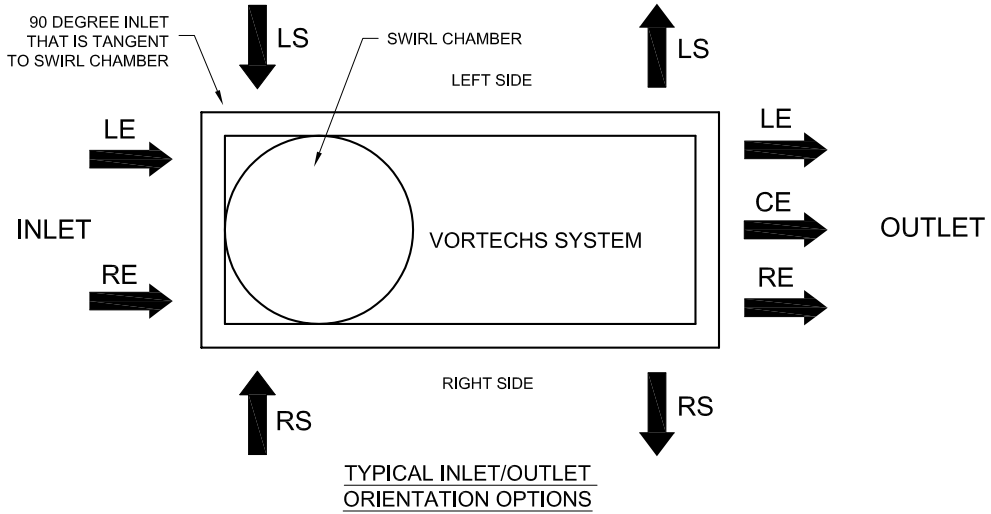
LE = LEFT END
 RE = RIGHT END
 LS = LEFT SIDE
 RS = RIGHT SIDE
 CE = CENTER END

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

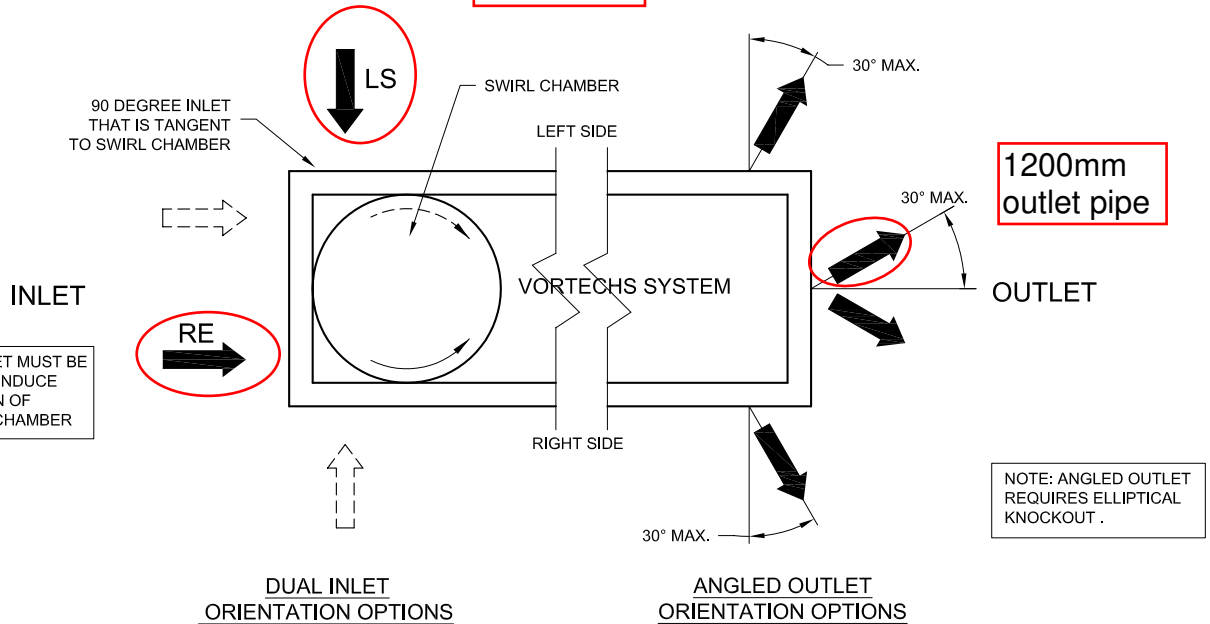


TYPICAL VORTECHS® SYSTEM ORIENTATIONS

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



OPTION 2



ORIENTATION KEY

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

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



TYPICAL VORTECHS® SYSTEM ORIENTATIONS

DATE: 4/7/06

SCALE: NONE

FILE NAME: TYPVX ORIENTATION

DRAWN: GMC

CHECKED: NDG

**VORTECHS SYSTEM[®] ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
 BASED ON AN AVERAGE PARTICLE SIZE OF 80 MICRONS
 Stinson Subdivision (4386 Rideau Valley Drive)**



**Ottawa, ON
 Model 1522CIP In-line**

Design Ratio¹ = $\frac{(6.12 \text{ hectares}) \times (0.67) \times (2.775)}{(16.4 \text{ m}^2)} = 0.69$

<u>Rainfall Intensity</u> mm/hr	<u>Operating Rate²</u> % of capacity	<u>Flow Treated</u> (l/s)	<u>% Total Rainfall</u> Volume ³	<u>Rmvl. Effcy⁴</u> (%)	<u>Rel. Effcy</u> (%)
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 Adjustment⁵ = 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/m².

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, L_s , is $10h_s$, but no less than $3W_0$; the length of the apron, L_A , is $5h_s$, but no less than W_0 . The overall length of the basin (pool plus apron), L_B , is $15h_s$, but no less than $4W_0$.
 - For the energy dissipating pool:
 - $10h_s = 10 * 0.60m = 6.0m$, or $3W_0 = 3 * 1.2m = 3.6m$ minimum
 - Designed L_s is 5.7m, which is $> 3W_0$ and just 0.3m shy of $10h_s$.
 - Length of the apron:
 - $L_A = 5h_s = 5 * 0.60m = 1.75m$, which is $> W_0$
 - Overall length of the basin:
 - $15h_s = 15 * 0.35m = 5.25m$, which is $> 4W_0$
 - 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.

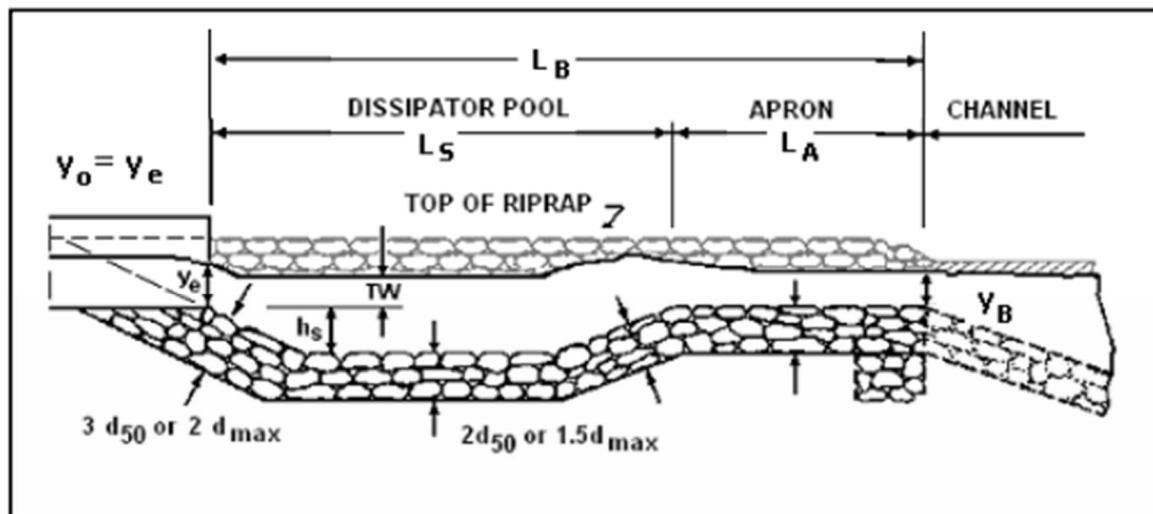


Figure 10.1. Profile of Riprap Basin

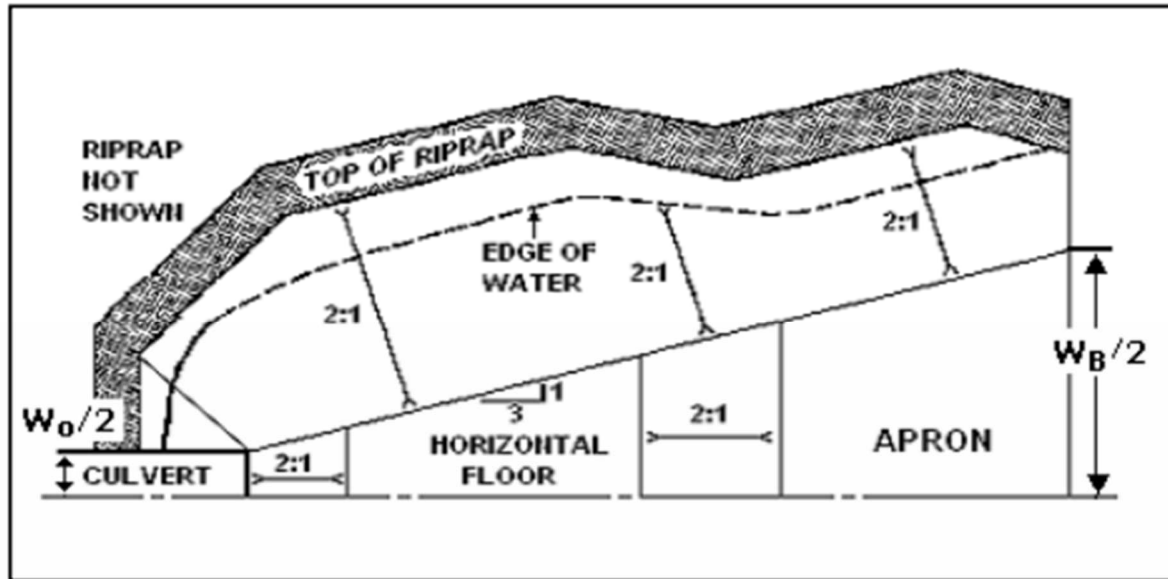


Figure 10.2. Half Plan of Riprap Basin

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 $3W_0$; the length of the apron, L_A , is $5h_s$, but no less than W_0 . The overall length of the basin (pool plus apron), L_B , is $15h_s$, but no less than $4W_0$.
- A riprap cutoff wall or sloping apron can be constructed if downstream channel degradation is anticipated as shown in Figure 10.1.

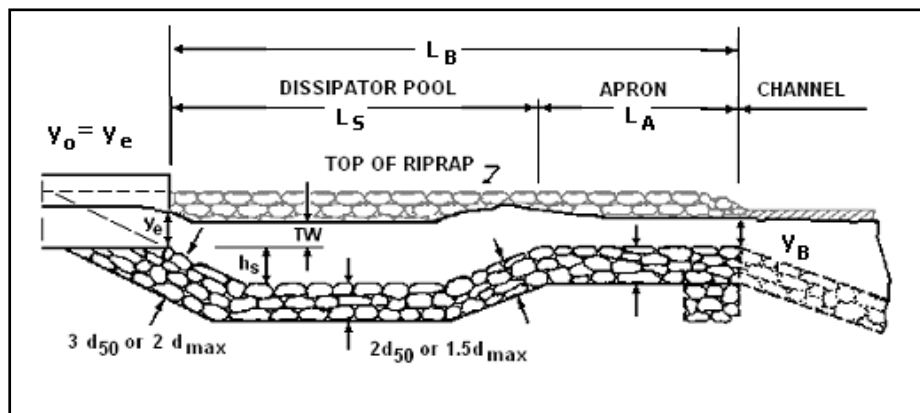


Figure 10.1. Profile of Riprap Basin

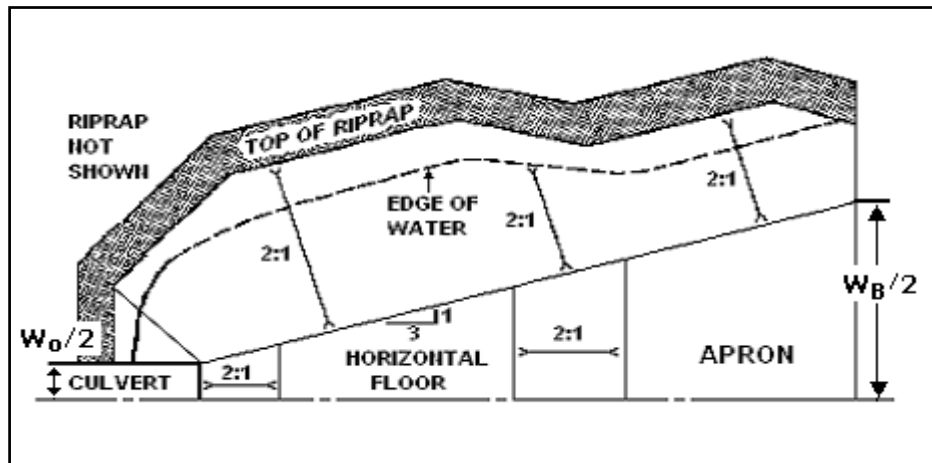


Figure 10.2. Half Plan of Riprap Basin

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 m³/s (0.1 to 100 ft³/s). Both angular and rounded rock with an average size, D_{50} , ranging from 6 mm (1.4 in) to 177 mm (7 in) and gradation coefficients ranging from 1.05 to 2.66 were tested. Two pipe slopes were considered, 0 and 3.75%. In all, 459 model basins were studied. The following conclusions were drawn from an analysis of the experimental data and observed operating characteristics:

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

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

Anticipating that standard or modified end sections would not likely be used when a riprap basin is located at a culvert outlet, the data with these configurations were not used to develop the design relationships. This assumption reduced the number of applicable runs to 346. A total of 128 runs had a D_{50}/y_e of less than 0.1. These data did not exhibit relationships that appeared

useful for design and were eliminated. An additional 69 runs where $h_s/D_{50} < 2$ were also eliminated by the authors of this edition of HEC 14. These runs were not considered reliable for design, especially those with $h_s = 0$. Therefore, the final design development used 149 runs from the study. Of these, 106 were for pipe culverts and 43 were for box culverts. Based on these data, two design relationships are presented here: an envelope design and a best fit design.

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

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

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:

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

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.

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

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

10.1.2 Basin Length

Frequency tables for both box culvert data and pipe culvert data of relative length of scour hole ($L_s/h_s < 6$, $6 < L_s/h_s < 7$, $7 < L_s/h_s < 8$. . . $25 < L_s/h_s < 30$), with relative tailwater depth TW/y_e in increments of 0.03 m (0.1 ft) as a third variable, were constructed using data from 346

experimental runs. For box culvert runs L_s/h_s was less than 10 for 78% of the data and L_s/h_s was less than 15 for 98% of the data. For pipe culverts, L_s/h_s was less than 10 for 91% of the data and, L_s/h_s was less than 15 for all data. A 3:1 flare angle is recommended for the basins walls. This angle will provide a sufficiently wide energy dissipating pool for good basin operation.

10.1.3 High Tailwater

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

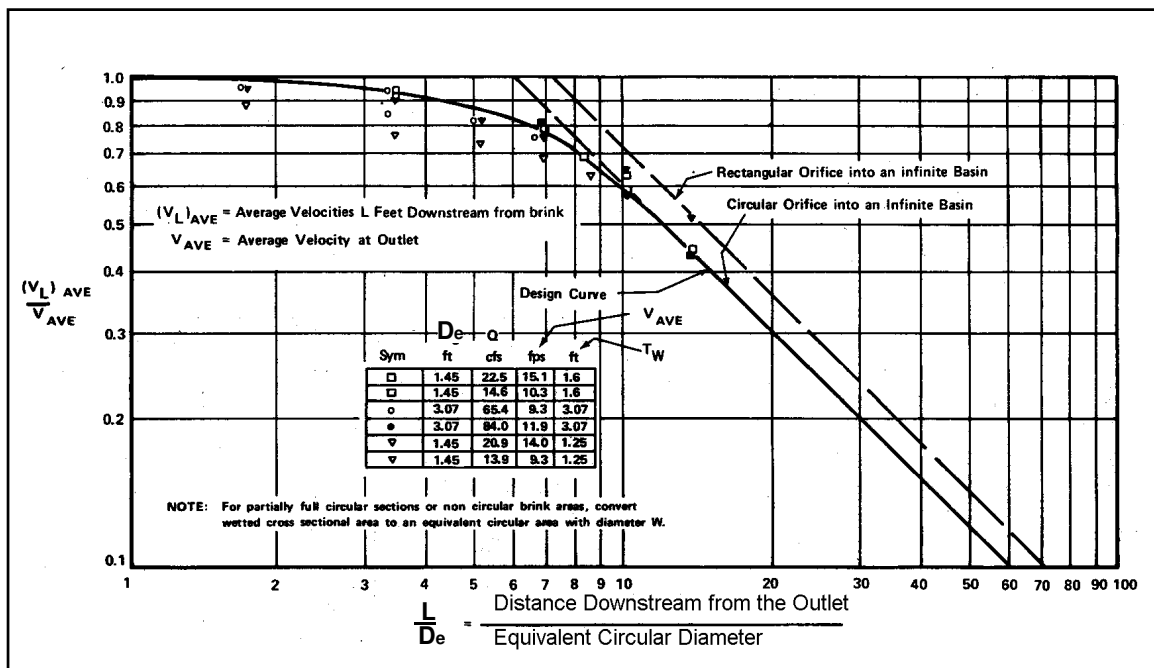


Figure 10.3. Distribution of Centerline Velocity for Flow from Submerged Outlets

10.1.4 Riprap Details

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

10.1.5 Design Procedure

The design procedure for a riprap basin is as follows:

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

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

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

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

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

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

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

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

$$V_c = Q/A_c$$

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:

$$Q = 22.7 \text{ m}^3/\text{s}$$

$$y_o = 1.22 \text{ m (normal flow depth) = brink depth}$$

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

$$y_o = y_e = 1.22 \text{ m}$$

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

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

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 $D_{50}/y_e \geq 0.1$.

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

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.

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

From Equation 10.1,

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

$$h_s = (h_s / y_e) y_e = 1.55 (1.22) = 1.89 \text{ m}$$

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

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

$$L_S = 10h_s = 10(1.89) = 18.9 \text{ m}$$

$$L_S \text{ min} = 3W_o = 3(2.44) = 7.3 \text{ m, use } L_S = 18.9 \text{ m}$$

$$L_B = 15h_s = 15(1.89) = 28.4 \text{ m}$$

$$L_B \text{ min} = 4W_o = 4(2.44) = 9.8 \text{ m, use } L_B = 28.4 \text{ m}$$

$$W_B = W_o + 2(L_B/3) = 2.44 + 2(28.4/3) = 21.4 \text{ m}$$

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, $A_c = 10.7$ m²

$$V_B = V_c = Q/A_c = 22.7/10.7 = 2.1 \text{ m/s (acceptable)}$$

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,

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

$$h_s = (h_s / y_e)y_e = 1.09 (1.22) = 1.34 \text{ m}$$

$h_s/D_{50} = 1.34/0.75 = 1.8$ 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,

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

$$h_s = (h_s / y_e)y_e = 1.16 (1.22) = 1.42 \text{ m}$$

$h_s/D_{50} = 1.42/0.71 = 2.0$ 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.

$$L_S = 10h_s = 10(1.42) = 14.2 \text{ m}$$

$$L_S \text{ min} = 3W_o = 3(2.44) = 7.3 \text{ m, use } L_S = 14.2 \text{ m}$$

$$L_B = 15h_s = 15(1.42) = 21.3 \text{ m}$$

$$L_B \text{ min} = 4W_o = 4(2.44) = 9.8 \text{ m, use } L_B = 21.3 \text{ m}$$

$$W_B = W_o + 2(L_B/3) = 2.44 + 2(21.3/3) = 16.6 \text{ m}$$

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, $A_c = 9.9$ m²

$V_B = V_c = Q/A_c = 22.7/9.9 = 2.3$ 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:

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

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

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 and 10.2) 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:

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

$$y_o = 4 \text{ ft (normal flow depth) = brink depth}$$

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 .

$$y_o = y_e = 4 \text{ ft}$$

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

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

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 $D_{50}/y_e \geq 0.1$.

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

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

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

From Equation 10.1,

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

$$h_s = (h_s/y_e)y_e = 1.50 (4) = 6.0 \text{ ft}$$

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

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

$$L_S = 10h_s = 10(6.0) = 60 \text{ ft}$$

$$L_S \text{ min} = 3W_o = 3(8) = 24 \text{ ft, use } L_S = 60 \text{ ft}$$

$$L_B = 15h_s = 15(6.0) = 90 \text{ ft}$$

$$L_B \text{ min} = 4W_o = 4(8) = 32 \text{ ft, use } L_B = 90 \text{ ft}$$

$$W_B = W_o + 2(L_B/3) = 8 + 2(90/3) = 68 \text{ ft}$$

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

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

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

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

$$V_B = V_c = Q/A_c = 800/113.9 = 7.0 \text{ ft/s (acceptable)}$$

The initial trial of riprap ($D_{50} = 1.83 \text{ ft}$) 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.

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

From Equation 10.1,

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

$$h_s = (h_s / y_e)y_e = 1.04 (4) = 4.2 \text{ ft}$$

$h_s/D_{50} = 4.2/2.5 = 1.7$ 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.

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

From Equation 10.1,

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

$$h_s = (h_s / y_e)y_e = 1.15 (4) = 4.6 \text{ ft}$$

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

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

$$L_s = 10h_s = 10(4.6) = 46 \text{ ft}$$

$$L_s \text{ min} = 3W_o = 3(8) = 24 \text{ ft, use } L_s = 46 \text{ ft}$$

$$L_B = 15h_s = 15(4.6) = 69 \text{ ft}$$

$$L_B \text{ min} = 4W_o = 4(8) = 32 \text{ ft, use } L_B = 69 \text{ ft}$$

$$W_B = W_o + 2(L_B/3) = 8 + 2(69/3) = 54 \text{ ft}$$

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, $V_B = V_c$.

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

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

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

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

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

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

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

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

Compute equivalent circular diameter, D_e , for brink area:

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

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

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

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

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

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

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

- D = 1.83 m CMP with Manning's $n = 0.024$
- $S_o = 0.004$ m/m
- Q = 3.82 m³/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, $y_o/D = 0.45$

$$y_o = (y_o/D)D = 0.45(1.83) = 0.823 \text{ m (brink depth)}$$

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

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

$$V_o = Q/A = 3.82/1.15 = 3.32 \text{ m/s}$$

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

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

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 $D_{50}/y_e \geq 0.1$.

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

$$TW/y_e = 0.61/0.76 = 0.80. \text{ Therefore, from Equation 10.2,}$$

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

From Equation 10.1,

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

$$h_s = (h_s/y_e)y_e = 0.933(0.76) = 0.71 \text{ m}$$

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

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

$$L_S = 10h_s = 10(0.71) = 7.1 \text{ m}$$

$$L_S \text{ min} = 3W_o = 3(1.83) = 5.5 \text{ m, use } L_S = 7.1 \text{ m}$$

$$L_B = 15h_s = 15(0.71) = 10.7 \text{ m}$$

$$L_B \text{ min} = 4W_o = 4(1.83) = 7.3 \text{ m, use } L_B = 10.7 \text{ m}$$

$$W_B = W_o + 2(L_B/3) = 1.83 + 2(10.7/3) = 9.0 \text{ m}$$

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

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

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

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

$$V_c = Q/A_c = 3.82/2.48 = 1.5 \text{ m/s (acceptable)}$$

The initial trial of riprap ($D_{50} = 0.15 \text{ m}$) 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.

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

From Equation 10.1,

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

$$h_s = (h_s / y_e) y_e = 0.320 (0.76) = 0.24 \text{ m}$$

$h_s/D_{50} = 0.24/0.25 = 0.96$ 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.

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

From Equation 10.1,

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

$$h_s = (h_s / y_e) y_e = 0.545 (0.76) = 0.41 \text{ m}$$

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

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

$$L_S = 10h_s = 10(0.41) = 4.1 \text{ m}$$

$$L_S \text{ min} = 3W_o = 3(1.83) = 5.5 \text{ m, use } L_S = 5.5 \text{ m}$$

$$L_B = 15h_s = 15(0.41) = 6.2 \text{ m}$$

$$L_B \text{ min} = 4W_o = 4(1.83) = 7.3 \text{ m, use } L_B = 7.3 \text{ m}$$

$$W_B = W_o + 2(L_B/3) = 1.83 + 2(7.3/3) = 6.7 \text{ m}$$

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 0.25 \text{ m}$) 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, $V_B = V_c$.

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

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

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

$$V_c = Q/A_c = 3.82/2.28 = 1.7 \text{ m/s (acceptable)}$$

Two feasible options have been identified. First, a 0.71 m deep, 7.1 m long pool, with an 3.6 m apron using $D_{50} = 0.15 \text{ m}$. Second, a 0.41 m deep, 5.5 m long pool, with a 1.8 m apron using $D_{50} = 0.25 \text{ m}$. 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 $TW/y_o \leq 0.75$

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

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:

$$\begin{aligned}D &= 6 \text{ ft CMP with Manning's } n = 0.024 \\S_o &= 0.004 \text{ ft/ft} \\Q &= 135 \text{ ft}^3/\text{s} \\y_n &= 4.5 \text{ ft (normal flow depth in the pipe)} \\V_n &= 5.9 \text{ ft/s (normal velocity in the pipe)} \\TW &= 2.0 \text{ ft (tailwater depth)}\end{aligned}$$

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 .

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

$$TW/D = 2.0/6 = 0.33$$

From Figure 3.4, $y_o/D = 0.45$

$$y_o = (y_o/D)D = 0.45(6) = 2.7 \text{ ft (brink depth)}$$

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

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

$$V_o = Q/A = 135/12.35 = 10.9 \text{ ft/s}$$

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

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

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 $D_{50}/y_e \geq 0.1$.

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

$$TW/y_e = 2.0/2.48 = 0.806. \text{ Therefore, from Equation 10.2,}$$

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

From Equation 10.1,

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

$$h_s = (h_s/y_e)y_e = 0.923(2.48) = 2.3 \text{ ft}$$

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

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

$$L_S = 10h_S = 10(2.3) = 23 \text{ ft}$$

$$L_S \text{ min} = 3W_o = 3(6) = 18 \text{ ft, use } L_S = 23 \text{ ft}$$

$$L_B = 15h_S = 15(2.3) = 34.5 \text{ ft}$$

$$L_B \text{ min} = 4W_o = 4(6) = 24 \text{ ft, use } L_B = 34.5 \text{ ft}$$

$$W_B = W_o + 2(L_B/3) = 6 + 2(34.5/3) = 29 \text{ ft}$$

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

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

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

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

$$V_c = Q/A_c = 135/26.4 = 5.1 \text{ ft/s (acceptable)}$$

The initial trial of riprap ($D_{50} = 0.5 \text{ ft}$) 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.

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

From Equation 10.1,

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

$$h_s = (h_s/y_e)y_e = 0.311(2.48) = 0.8 \text{ ft}$$

$h_s/D_{50} = 0.8/0.83 = 0.96$ 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.

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

From Equation 10.1,

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

$$h_s = (h_s/y_e)y_e = 0.581(2.48) = 1.4 \text{ ft}$$

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

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

$$L_S = 10h_S = 10(1.4) = 14 \text{ ft}$$

$$L_S \text{ min} = 3W_o = 3(6) = 18 \text{ ft, use } L_S = 18 \text{ ft}$$

$$L_B = 15h_S = 15(1.4) = 21 \text{ ft}$$

$$L_B \text{ min} = 4W_o = 4(6) = 24 \text{ ft, use } L_B = 24 \text{ ft}$$

$$W_B = W_o + 2(L_B/3) = 6 + 2(24/3) = 22 \text{ ft}$$

However, since the trial D_{50} is not available, the next larger riprap size ($D_{50} = 0.83 \text{ ft}$) 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, $y_c = 1.02 \text{ ft}$, $T_c = 26.1 \text{ ft}$, $A_c = 24.5 \text{ ft}^2$

$$V_c = Q/A_c = 135/24.5 = 5.5 \text{ ft/s (acceptable)}$$

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

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

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

10.2 RIPRAP APRON

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

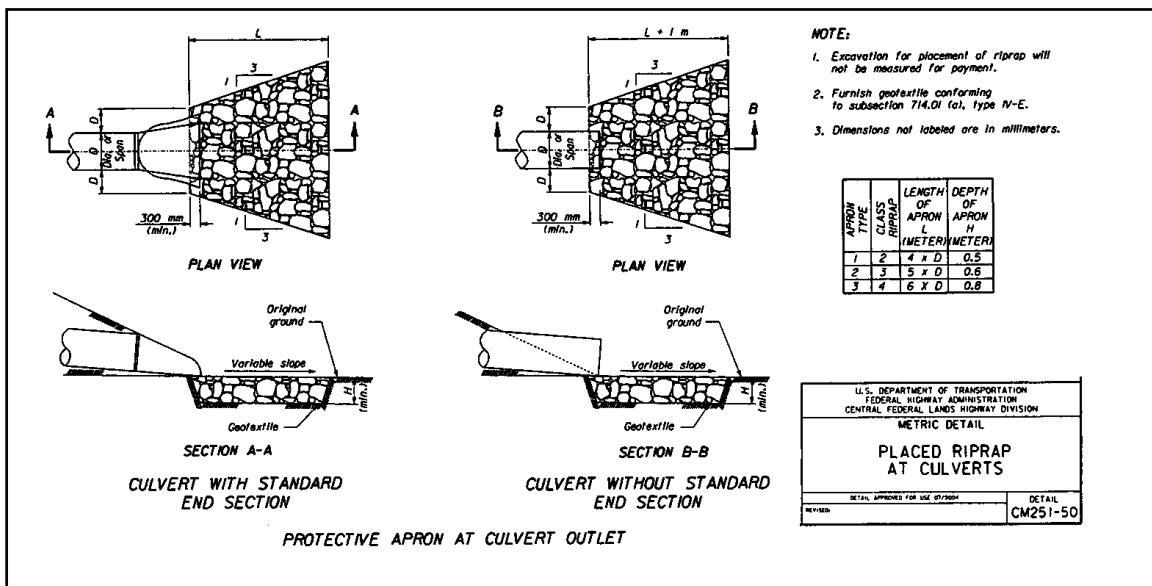


Figure 10.4. Placed Riprap at Culverts (Central Federal Lands Highway Division)

They are constructed of riprap or grouted riprap at a zero grade for a distance that is often related to the outlet pipe diameter. These aprons do not dissipate significant energy except

through increased roughness for a short distance. However, they do serve to spread the flow helping to transition to the natural drainage way or to sheet flow where no natural drainage way exists. However, if they are too short, or otherwise ineffective, they simply move the location of potential erosion downstream. The key design elements of the riprap apron are the riprap size as well as the length, width, and depth of the apron.

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

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

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:

$$D' = \frac{D + y_n}{2} \quad (10.5)$$

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

¹D 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 3.5D₅₀ for the smallest riprap to a limit of 2.0D₅₀ 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:

$$\begin{aligned} Q &= 2.33 \text{ m}^3/\text{s} \\ D &= 1.5 \text{ m} \\ TW &= 0.5 \text{ m} \end{aligned}$$

Solution

Step 1. Calculate D₅₀ 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

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

Step 2. Determine riprap class. From Table 10.1, riprap class 2 (D₅₀ = 0.15 m) is required.

Step 3. Estimate apron dimensions.

From Table 10.1 for riprap class 2,

Length, $L = 4D = 4(1.5) = 6 \text{ m}$

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

Width (at apron end) $= 3D + (2/3)L = 3(1.5) + (2/3)(6) = 8.5 \text{ m}$

Design Example: Riprap Apron (CU)

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

$Q = 85 \text{ ft}^3/\text{s}$

$D = 5.0 \text{ ft}$

$TW = 1.6 \text{ ft}$

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 $TW = 0.4D = 0.4(5) = 2.0 \text{ ft}$

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

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

Step 3. Estimate apron dimensions.

From Table 10.1 for riprap class 2,

Length, $L = 4D = 4(5) = 20 \text{ ft}$

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

Width (at apron end) $= 3D + (2/3)L = 3(5) + (2/3)(20) = 28.3 \text{ ft}$

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.

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

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).

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 1000m³ 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 m³/yr to 3,600 m³/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 m³ 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 m³ 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 m³).

Infiltration (Loss)

The Groundwater Impact Assessment prepared by Paterson provides a soil hydraulic conductivity of 1×10^{-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

¹Post-development runoff volume to the oxbow.

²Volume of water infiltrated/evaporated from the oxbow.

³Volume 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 m³) is greater than the retention volume in the oxbow (1,000 m³) 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

RIDEAU RIVER

RIDEAU VALLEY DRIVE

MUD CREEK

BERM

NORMAL WATER LEVELS

OXBOW

BLO
64

63

62

61

60

STINSON SUBDIVISION
(SUBJECT SITE)

59

58

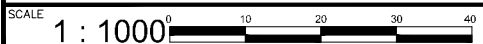


Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

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

CITY of OTTAWA
STINSON SUBDIVISION
4386 RIDEAU VALLEY DRIVE

OXBOW
EXISTING CONDITIONS



DATE APR 2024 JOB 121153 FIGURE 1

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Figure 2: Pre-Development Drainage Area

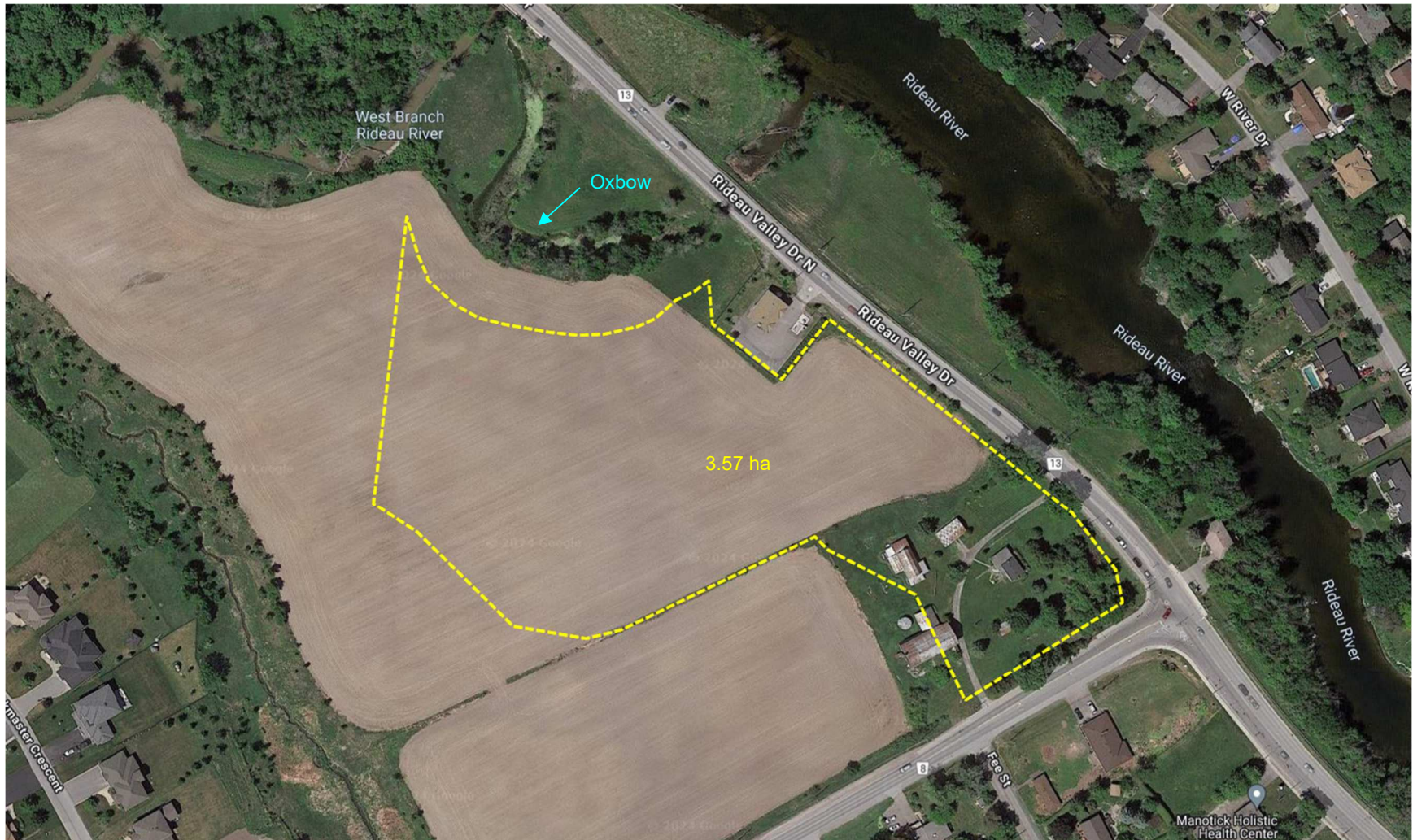
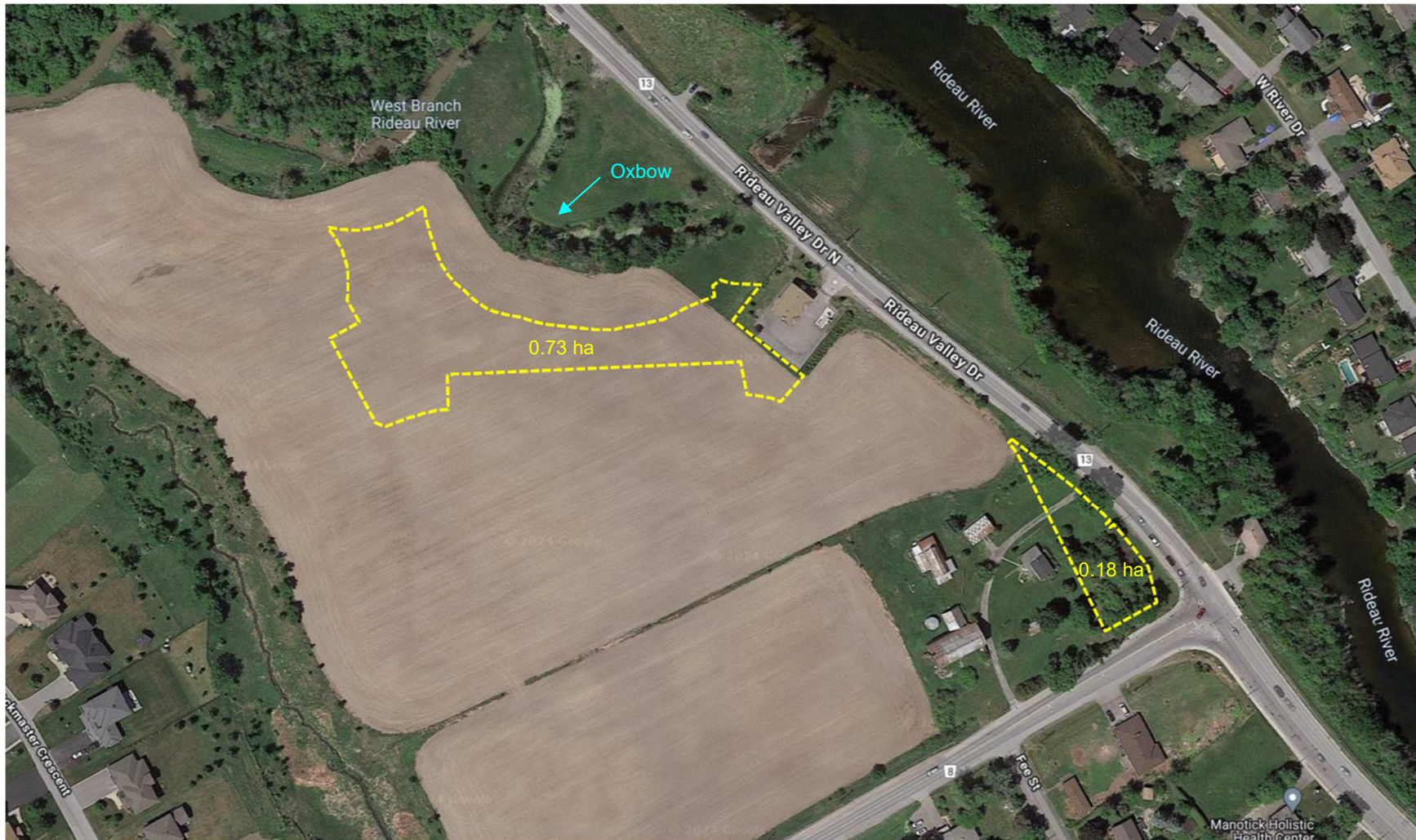
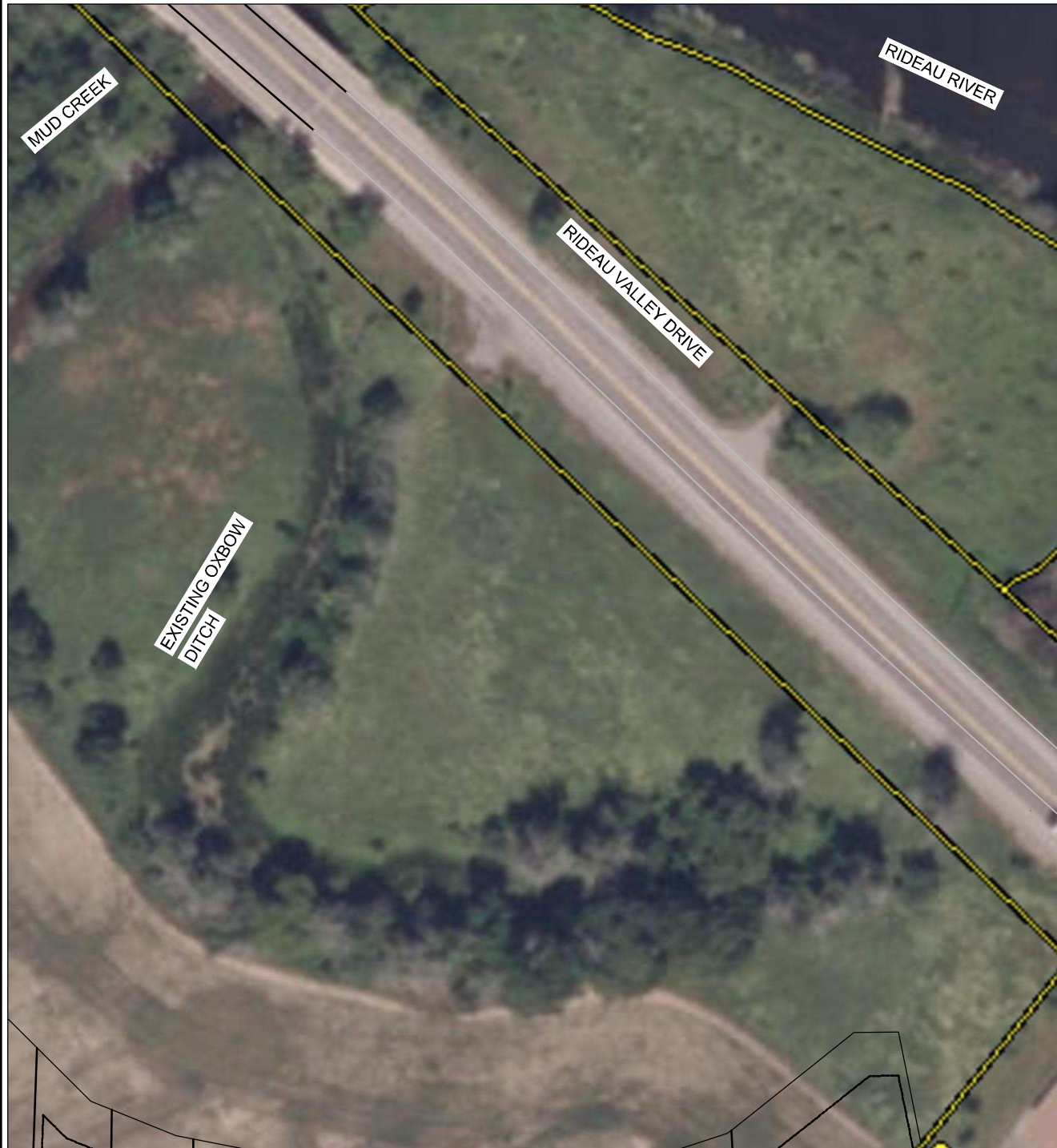


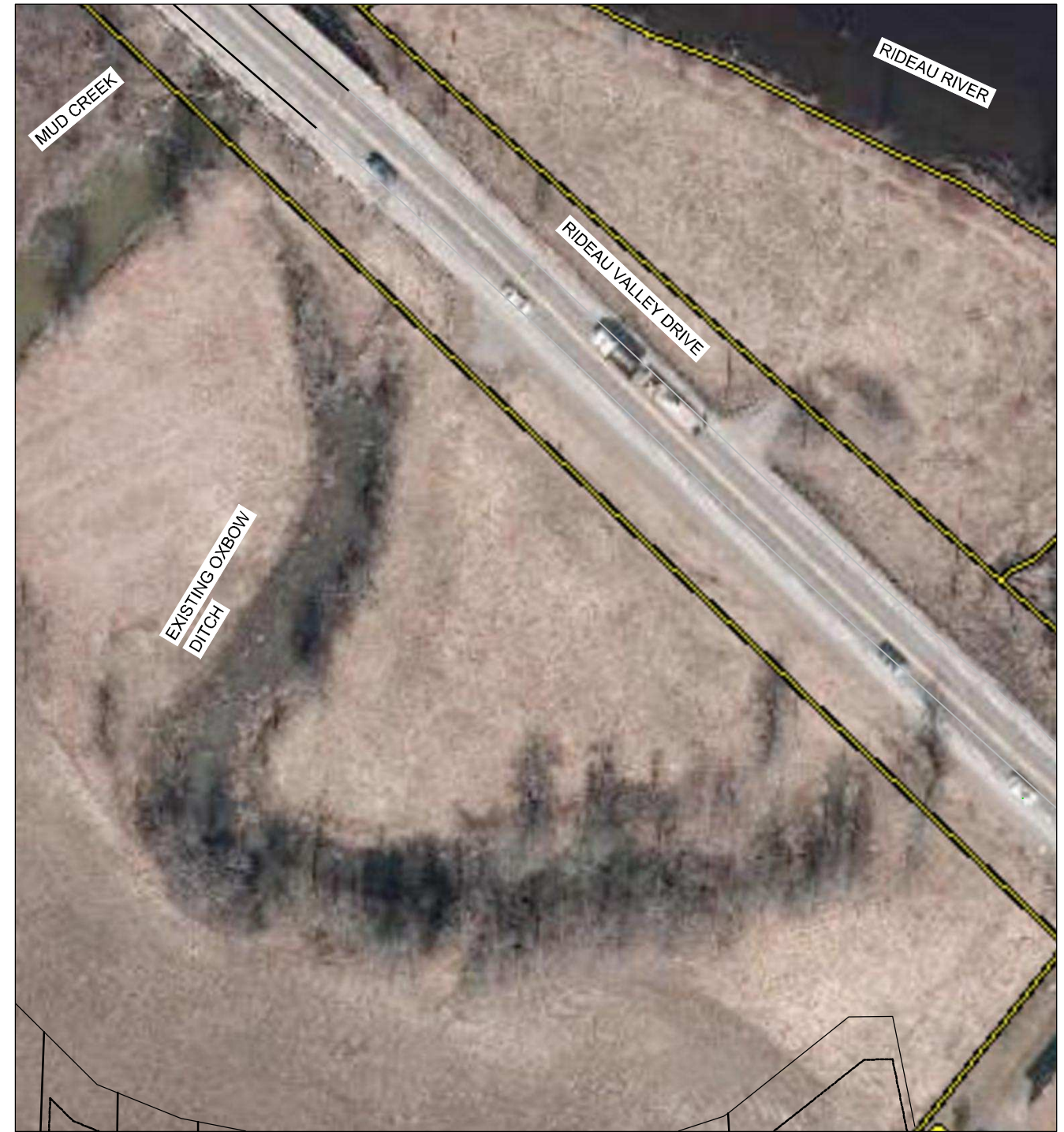
Figure 3: Post-Development Drainage Areas



JULY/AUGUST 2022 AERIAL



MARCH/APRIL 2021 AERIAL



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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 RIDEAU VALLEY DRIVE		
EXISTING OXBOW DITCH AERIAL		
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AUGUST/SEPTEMBER 2019 AERIAL



APRIL/MAY 2017 AERIAL



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

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

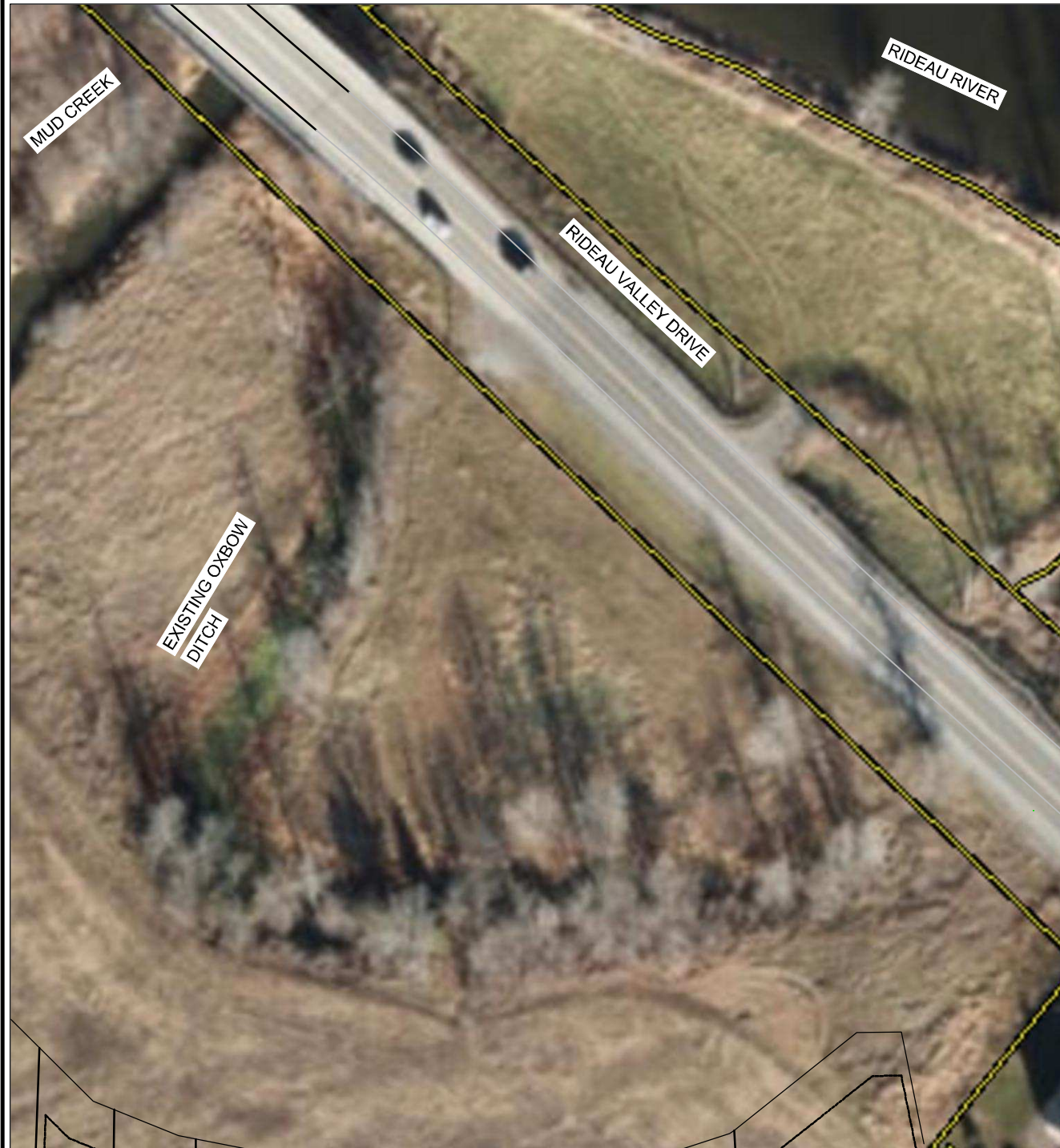
CITY OF OTTAWA
STINSON LANDS
4386 RIDEAU VALLEY DRIVE

**EXISTING OXBOW DITCH
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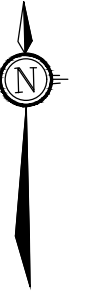
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JULY/AUGUST 2015 AERIAL



APRIL/MAY 2014 AERIAL



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 Ottawa, Ontario, Canada K2M 1P6

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

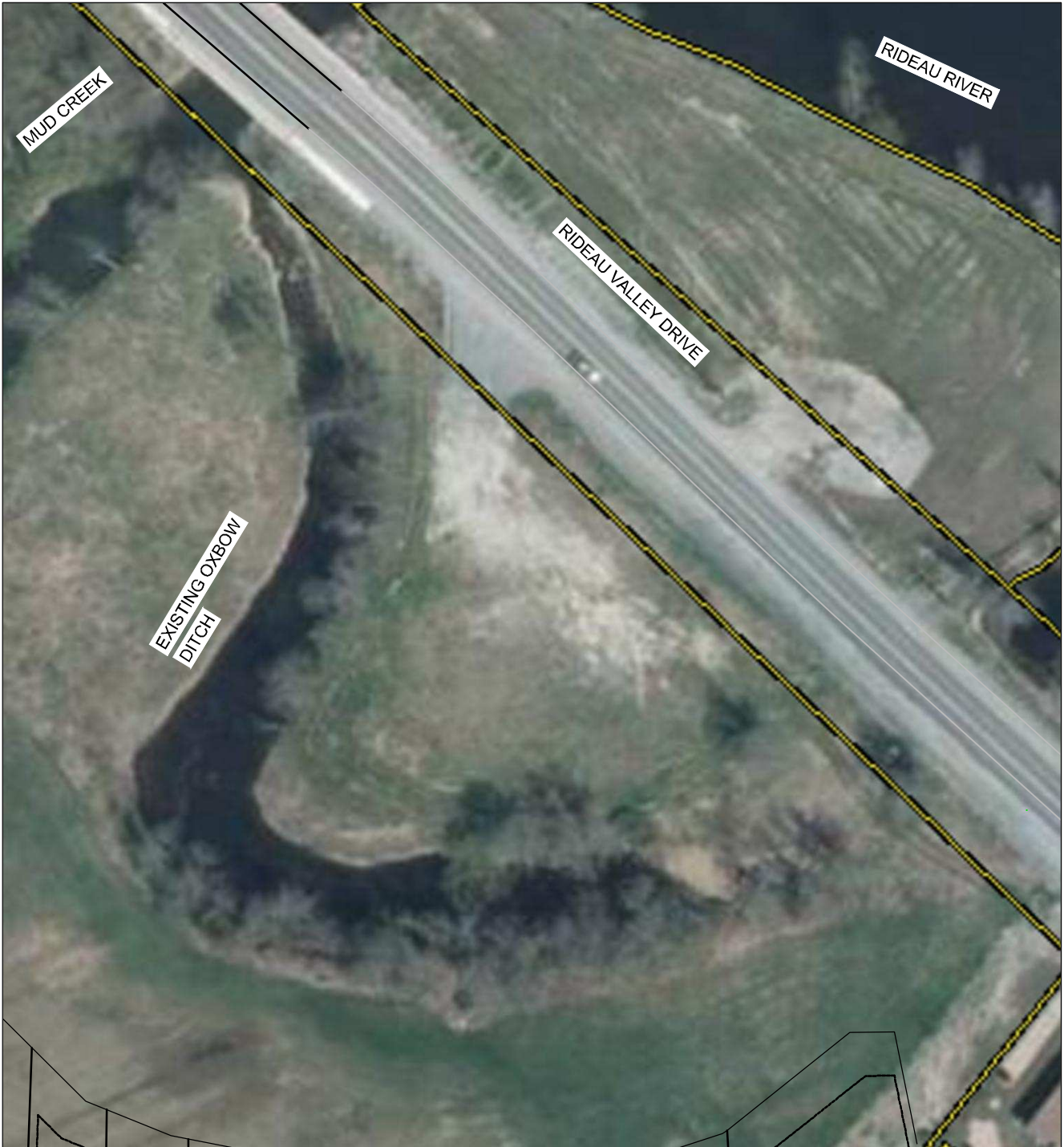
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 4386 RIDEAU VALLEY DRIVE

**EXISTING OXBOW DITCH
 AERIAL**


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DATE	JOB	FIGURE
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2011 AERIAL



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 <p>Engineers, Planners & Landscape Architects Suite 200, 240 Michael Cowpland Drive Ottawa, Ontario, Canada K2M 1P6</p> <p>Telephone (613) 254-9643 Facsimile (613) 254-5867 Website www.novatech-eng.com</p>	CITY OF OTTAWA STINSON LANDS 4386 RIDEAU VALLEY DRIVE		
	EXISTING OXBOW DITCH AERIAL		
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DATE APR 2024	JOB 121153	FIGURE 7	

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)*.

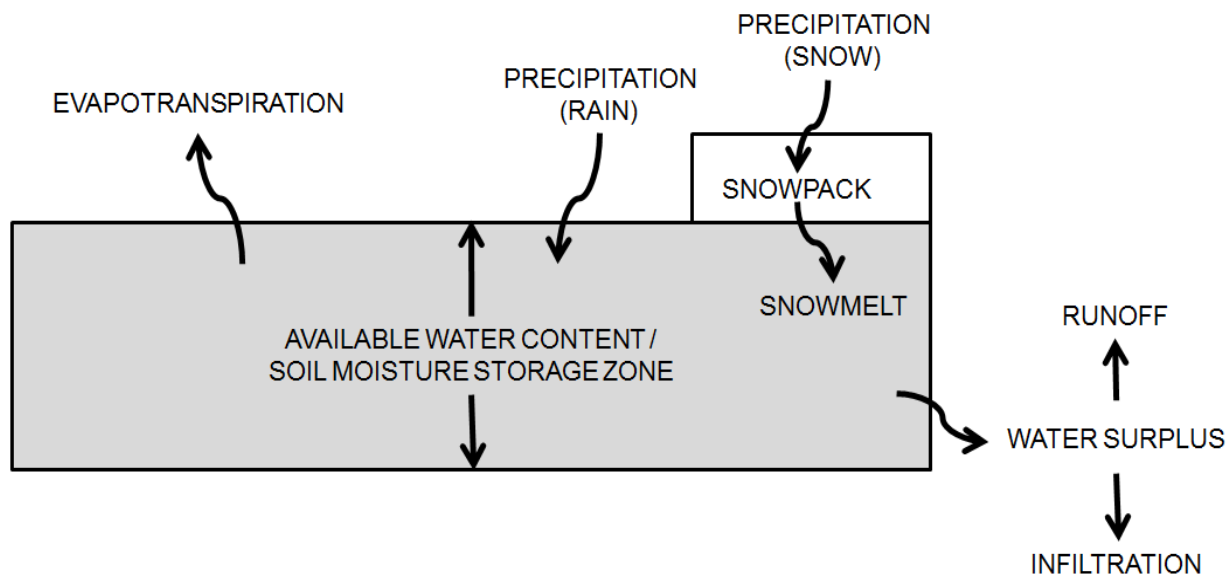


Figure 1: Conceptual Water Balance Model

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):

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

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

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

Therefore, the snowmelt equation is:

- $MELT: \text{If } (MAX\ TEMP > 0, \text{ IF}(SNOWPACK > 0, \text{ MIN}((0.45\text{cm}/^{\circ}\text{C}\text{-day} * MAX\ TEMP * [MAX\ TEMP / (MAX\ TEMP - MIN\ TEMP)] * 10\text{mm/cm}), SNOWPACK), 0), 0)$

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:

$$SNOWPACK_N = SNOWPACK_{N-1} + SNOW - MELT$$

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.

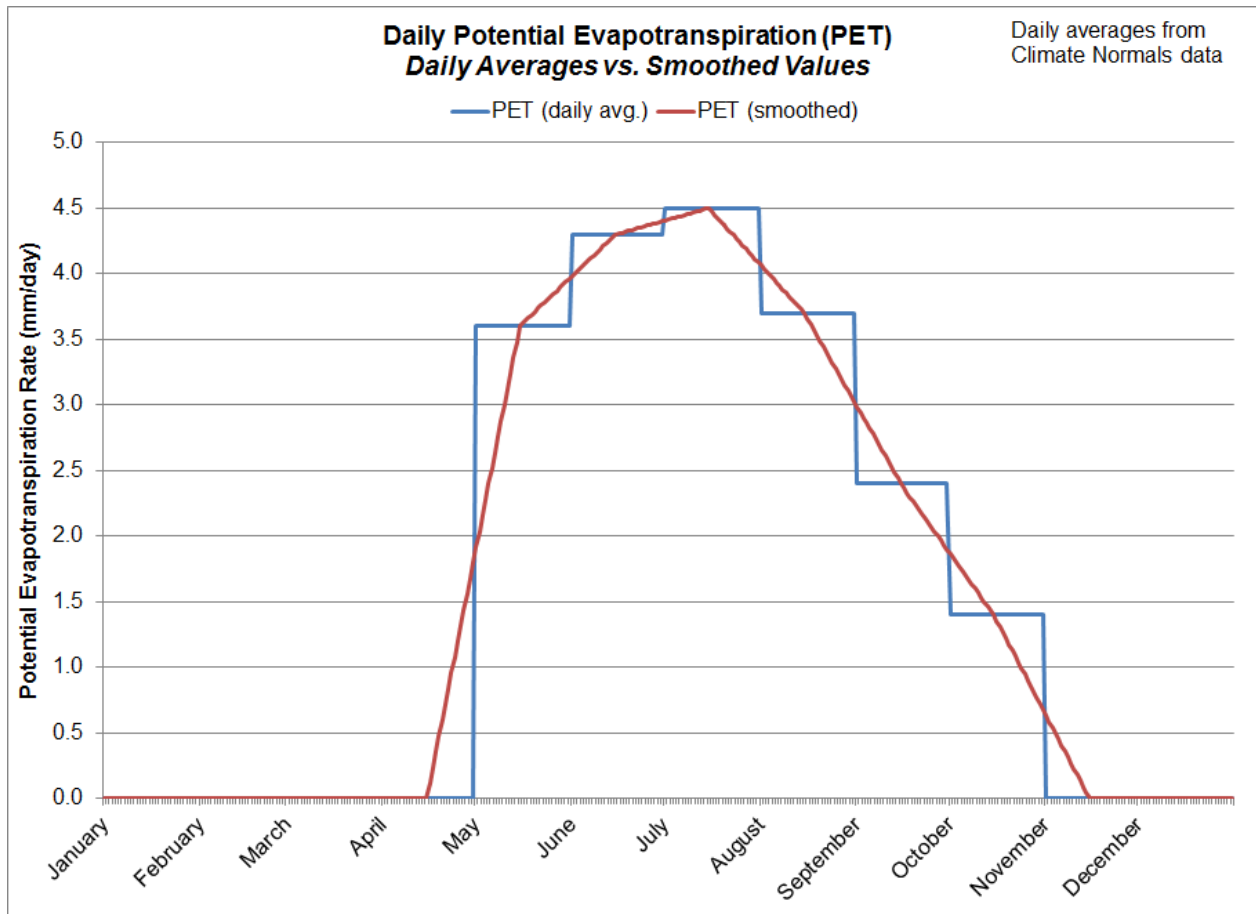


Figure 2: Daily Potential Evapotranspiration Rates (Daily Averages vs. Smoothed Values)

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:

$$PET = PE \times \text{Crop Cover Coefficient}$$

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.

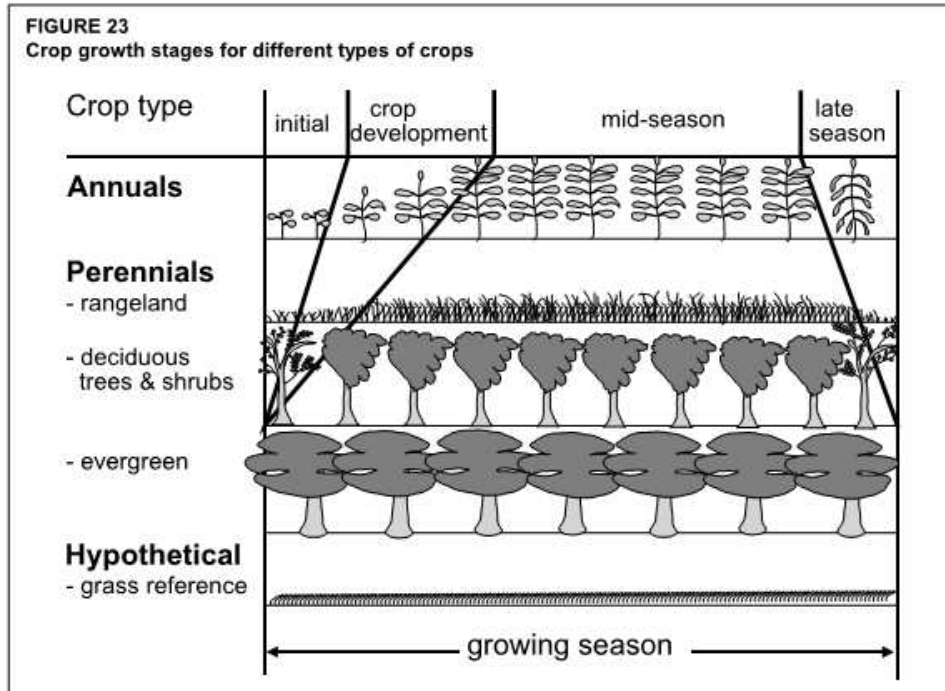


Figure 3: Crop Growth Stages for Different Types of Crops

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.

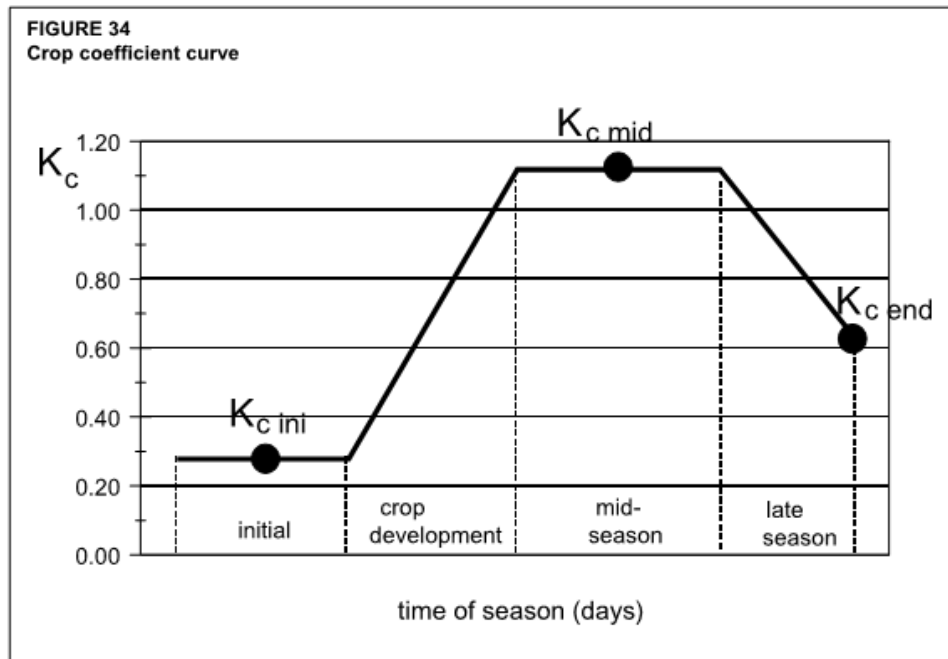


Figure 4: Crop Coefficient Curve

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):

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

$$WHERE: \Delta SOIL \text{ WATER} = SOIL \text{ WATER}_{N-1} - SOIL \text{ WATER}_N$$

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

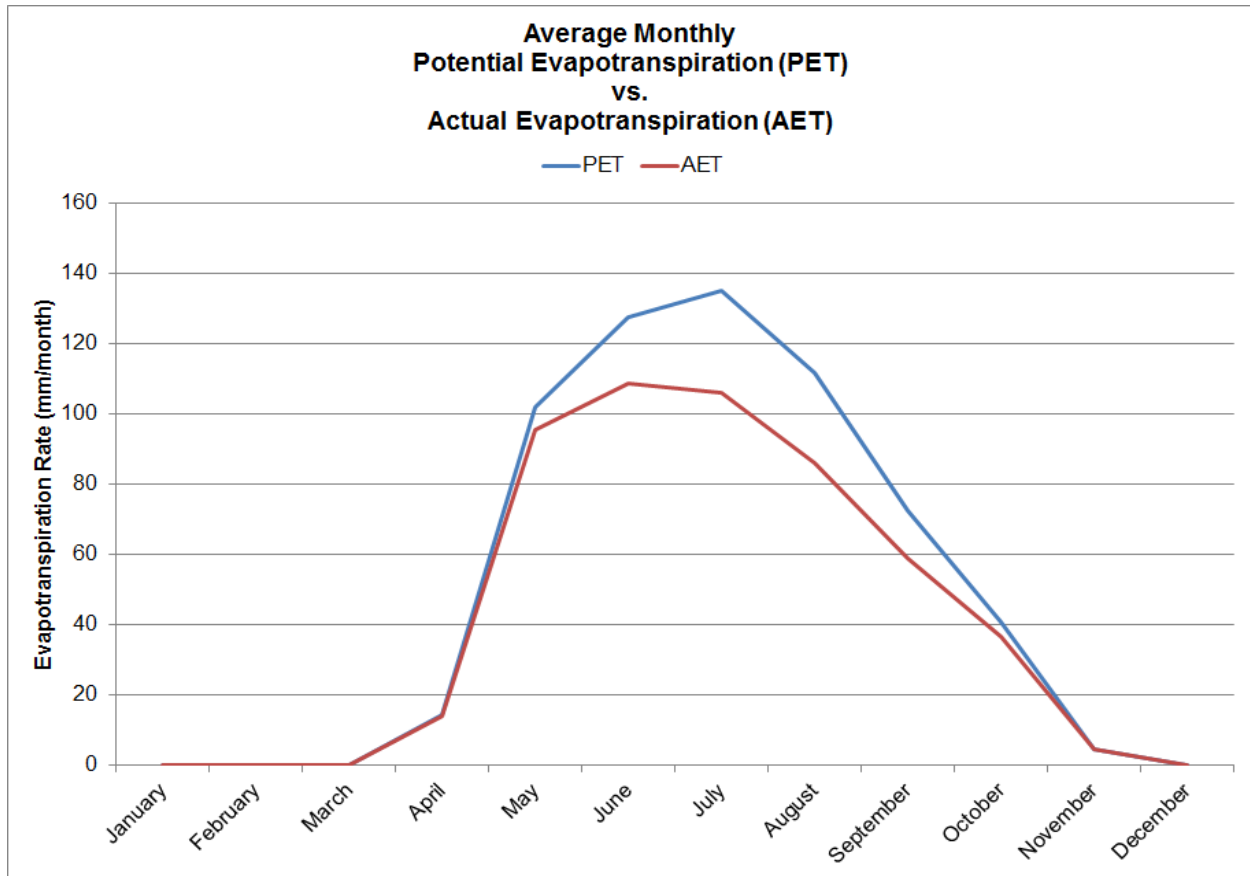


Figure 5: Average Monthly Potential Evapotranspiration vs. Actual Evapotranspiration

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 ($\Delta SOIL \text{ WATER}$) is based on the following relationship (Thornthwaite, 1948), where AWC represents the available water content:

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

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
<i>Topography</i>	
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
<i>Surficial Soils</i>	
Tight impervious clay	0.1
Medium combination of clay and loam	0.2
Open sandy loam	0.4
<i>Land Cover</i>	
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).

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	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	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

¹Available 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)

²Crop 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

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

Area ID	Area (ha)	Runoff (mm/yr)	Runoff (m ³ /yr)
1	0.20	216	432
2	2.80	267	8,021
3	0.49	303	1,493
4	0.08	711	569
TOTAL	3.57	294	10,514

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

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	88.7	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	238.9	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

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

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 ³	IF (soils)	IF (cover)	IF (topo)	IF (Total)	Dormant Season	Initial Growing Season	Middle of Growing Season	End of Growing Season	Potential Evaporation Rates (AVG. mm/d)											
		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	3.6	4.3	4.4	3.7	2.4	1.4	0.0	0.0
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

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

Novatech Project #:
Project Name:
Date Prepared:
Date Revised:
Input By:
Reviewed By:
Drawing Reference:

121153
Simons Lands Subdivision
1/11/2023
3/25/2024
Brendan Rundle
Sam Baha
121153-GP

Legend:

PROJECT SPECIFIC INFO
USER DESIGN INPUT
CUMULATIVE CELL
CALCULATED DESIGN CELL OUTPUT
CALCULATED ANNUAL CELL OUTPUT
CALCULATED RARE CELL OUTPUT
USER AS-BUILT INPUT

LOCATION				RESIDENTIAL FLOW														DEMAND										DESIGN CAPACITY										
STREET	AREA	FROM MH	TO MH	SINGLES	SEMS/TOWNS	APARTS	PARK AREA (ha)	POPULATION (in 1000's)	CUMULATIVE POPULATION (in 1000's)	PEAK FACTOR M	AVG POPULATION FLOW Q(c) (L/s)	PEAKED DESIGN POP 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.)	COMMERCIAL / INSTITUTIONAL AREA (ha.)	CUMULATIVE COMMERCIAL / INSTITUTIONAL AREA (ha.)	AVG DESIGN COMMERCIAL / INSTITUTIONAL FLOW Q (c) (L/s)	COMMERCIAL / INSTITUTIONAL PEAK FACTOR	CUMULATIVE ICI DRAINAGE AREA (ha.)	PEAKED DESIGN ICI FLOW Q (c) (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)	PROPOSED SEWER PIPE SIZING / DESIGN							
																															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	64.5	200 PVC	0.203	0.013	1.30	39.0	1.20	1.8%
Street 1	2	103	105	12	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			0.018	0.068	3.61	0.28	1.63	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	0.66	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	1.82	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	1.89	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	0.899	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	0.096	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	1.658	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	0.46	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	0.650	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	6.6%
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	0.690	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	6.2%
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	0.00	7.880	2.60	2.36	4.33	7.52	5.26	7.228							

DEMAND EQUATION		CAPACITY EQUATION	
Design Parameters:	Q(D) = Peak Design Flow (L/sec)	Q(A) = Peak Annual Flow (L/sec)	Q full (1/m) A R ^{1/2} / (2/3) So ^{1/12}
1. Q(D), Q(A), Q(R) =	Q(p) + Q(f) + Q(ici) + Q(e)	Q(R) = Peak Rare Flow (L/sec)	Where: Q full = Capacity (L/s)
2. Q(p) =	(P x q x M x K / 86,400)		n = Manning coefficient of roughness (0.013)
3. q Avg capita flow (L/per/day) =	280 L/per/day (design)		A = Flow area (m ²)
4. M = Harmon Formula (maximum of 4.0)	200 L/per/day (annual and rare)		R = Wetted perimeter (m)
5. K =	0.8 (design)		So = Pipe Slope/gradient
	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)		
	Q(f) = Foundation Flow (L/sec)		
	Q(ici) = Industrial / Commercial / Institutional		
	Design = 35000	Industrial	Commercial / Institutional
	Annual / Rare = 10000	Design = 17000	L/gha/d
	ICI Peak =	1.0	Light/d
	Annual / Rare =	1.0	* ICI Peak = 1.0 Default, 1.5 if ICI in contributing area is >20% (design only)

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



LOCATION			DEMAND										
AREA	FROM MH	TO MH	RESIDENTIAL FLOW				INDUSTRIAL / COMMERCIAL / INSTITUTIONAL FLOW				EXTRANEOUS FLOW	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.)	COMMERCIAL / INSTITUTIONAL AREA (ha.)	AVG DESIGN COMMERCIAL / INSTITUTIONAL FLOW Q (ci) (L/s)	COMMERCIAL / 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) Core	64236	59270	0.034	3.68	0.11	0.41	2.300	0.000	0.00	1.00	0.00	0.76	1.16
Servicing Connection (Rideau Valley Dr)	58922	69314	0.253	3.49	0.82	2.86	12.530	26.690	8.65	1.50	12.97	12.94	28.78
			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

Design Parameters:			Definitions:		
1. Q(D), Q(A), Q(R) =	Q(p) + Q(fd) + Q(ici) + Q(e)		Q(D) = Peak Design Flow (L/sec)	Q(A) = Peak Annual Flow (L/sec)	
2. Q(p) =	(P x q x M x K / 86,400)		Q(e) = Extraneous Flow (L/sec)	Q(R) = Peak Rare Flow (L/sec)	
3. q Avg capita flow (L/per/day)=	280 L/per/day (design) 200 L/per/day (annual and rare)		Q(p) = Population Flow (L/sec)		
4. M = Harmon Formula (maximum of 4.0)			K = Harmon Correction Factor		<u>Singles</u>
5. K =	0.8 (design) 0.6 (annual and rare)		Typ Service Diameter (mm)		3.4
6. Park flow is considered equivalent to a single unit / ha			Typ Service Length (m)		135
7. Foundation Drains	0.45 L/s/unit	Single Unit Equivalent / Park ha	l/l Pipe Rate (L/mm dia/m/hr) =		15
8. Q(ici) =	ICI Area x ICI Flow x ICI Peak		Q(fd) = Foundation Flow (L/sec)		0.007
9. Q(e) =	0.33 L/sec/ha (design) 0.30 L/sec/ha (annual) 0.55 L/sec/ha (rare)		Q(ici) = Industrial / Commercial / Institutional Flow (L/sec)		
			<u>Institutional / Commercial / Industrial</u>		
			Design =		
			Annual / Rare =		
			<u>ICI Peak *</u>		
			Design =		
			Annual / Rare =		

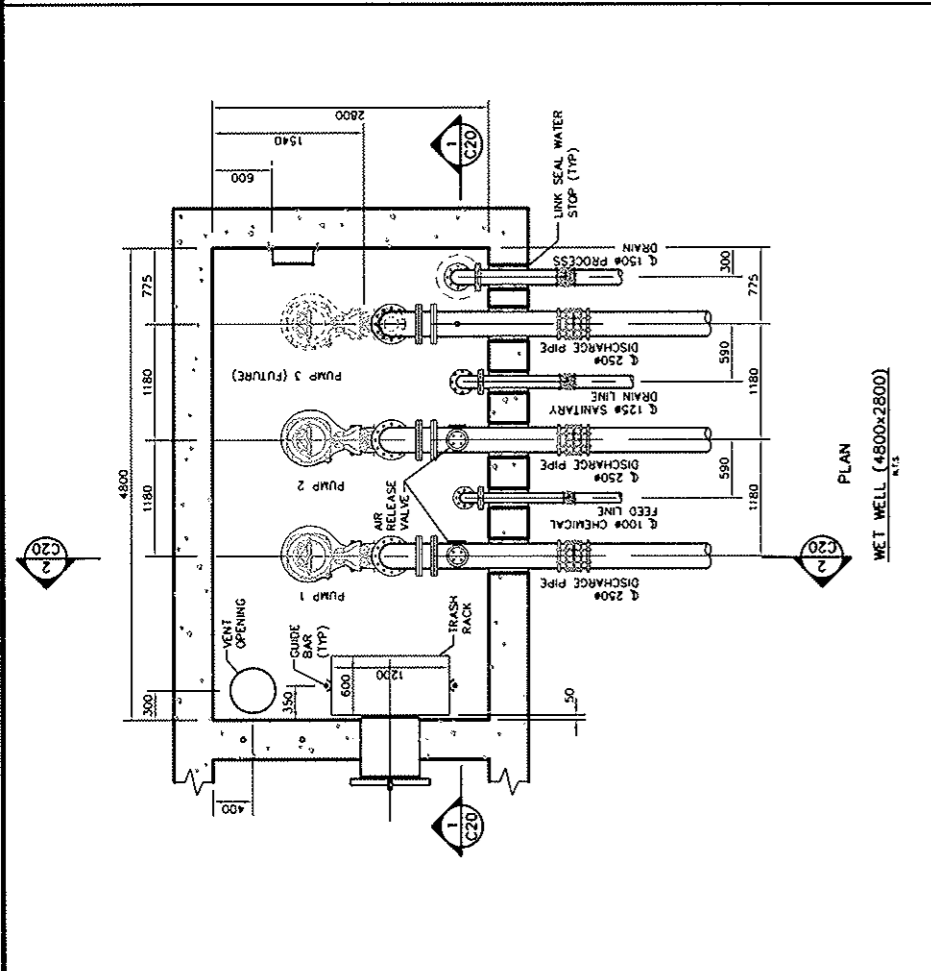
DESIGN BRIEF
VILLAGE OF MANOTICK
MUNICIPAL SERVICING
MAIN SANITARY SEWAGE PUMP STATION
CITY OF OTTAWA

11931

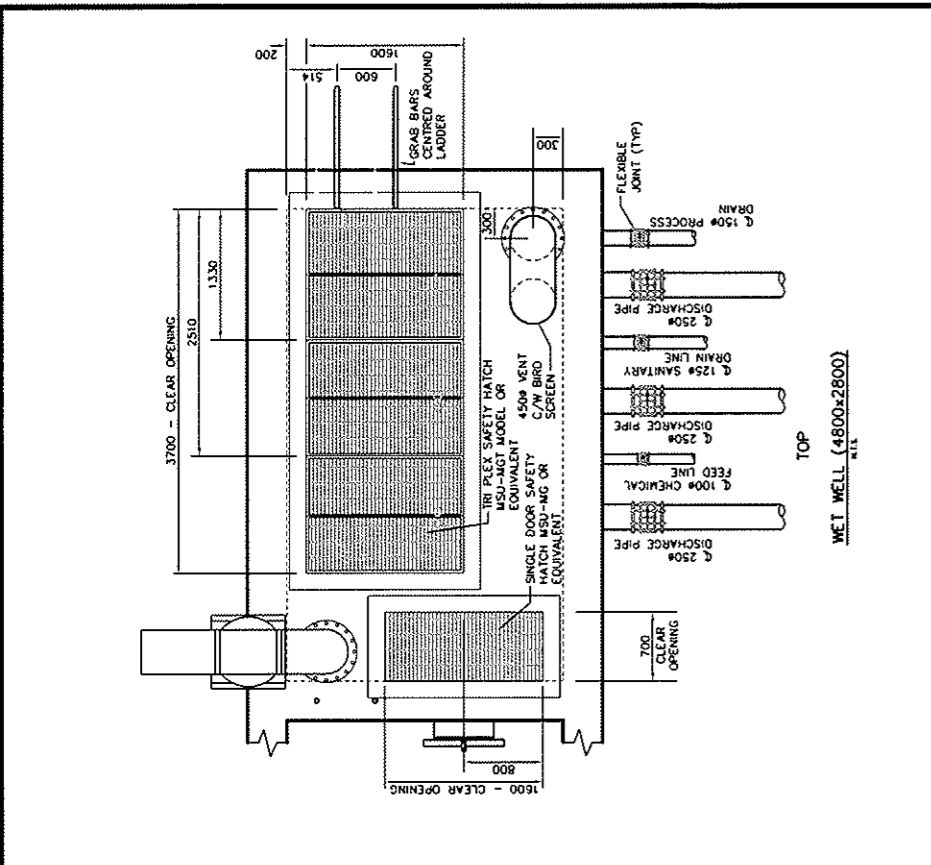
SEPTEMBER 2008



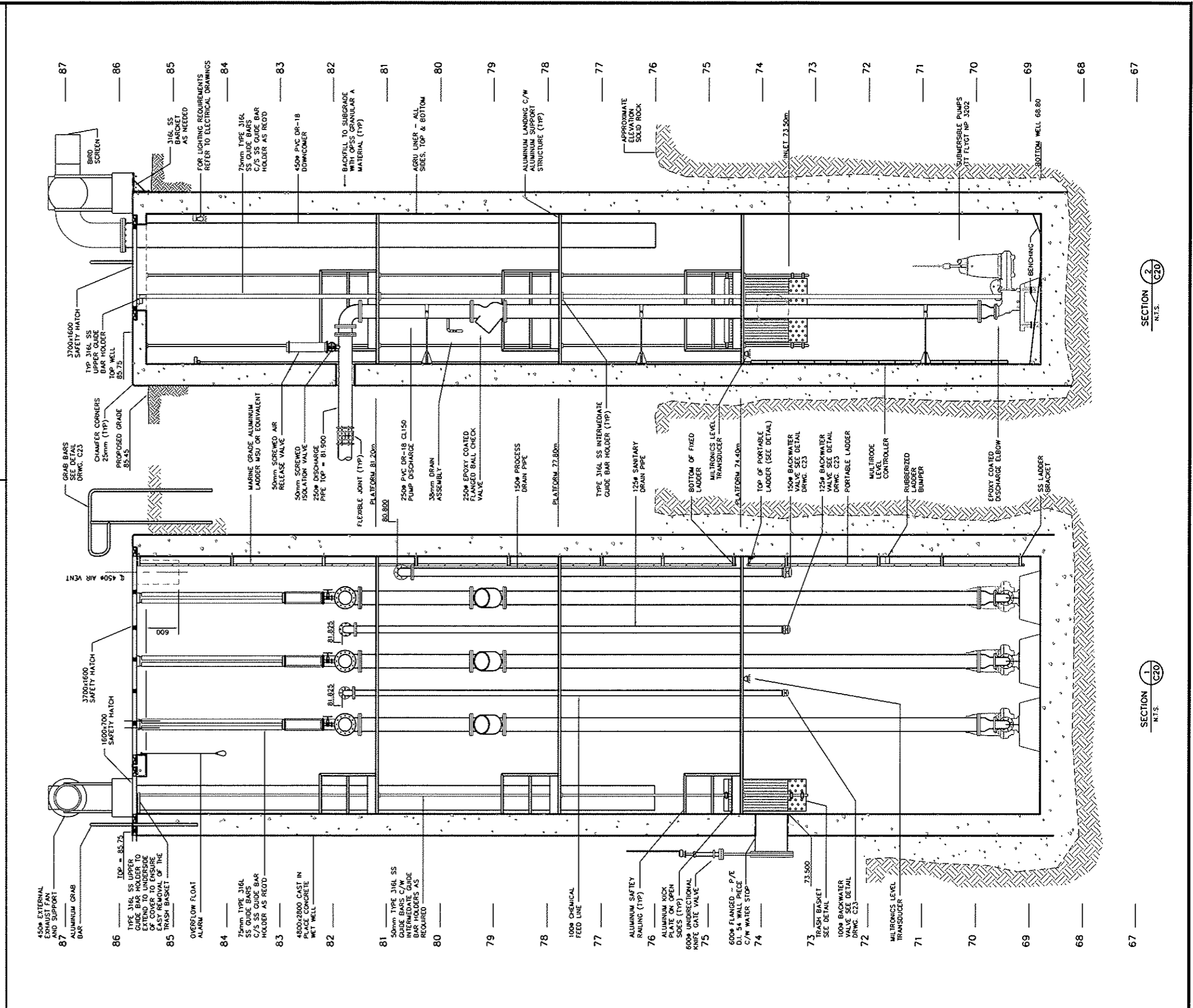
J:\11931_Manotick\9 Drawings\99civil\current\Design Brief-Sept. 2008\Fig 7 - 8 Control Wet Well.dwg Sheet Set: ###
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PLAN
 WET WELL (4800x2800)
 N.T.S.



TOP
 WET WELL (4800x2800)
 N.T.S.



SECTION 2
 N.T.S.

SECTION 1
 N.T.S.



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	FIGURE 7	

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.

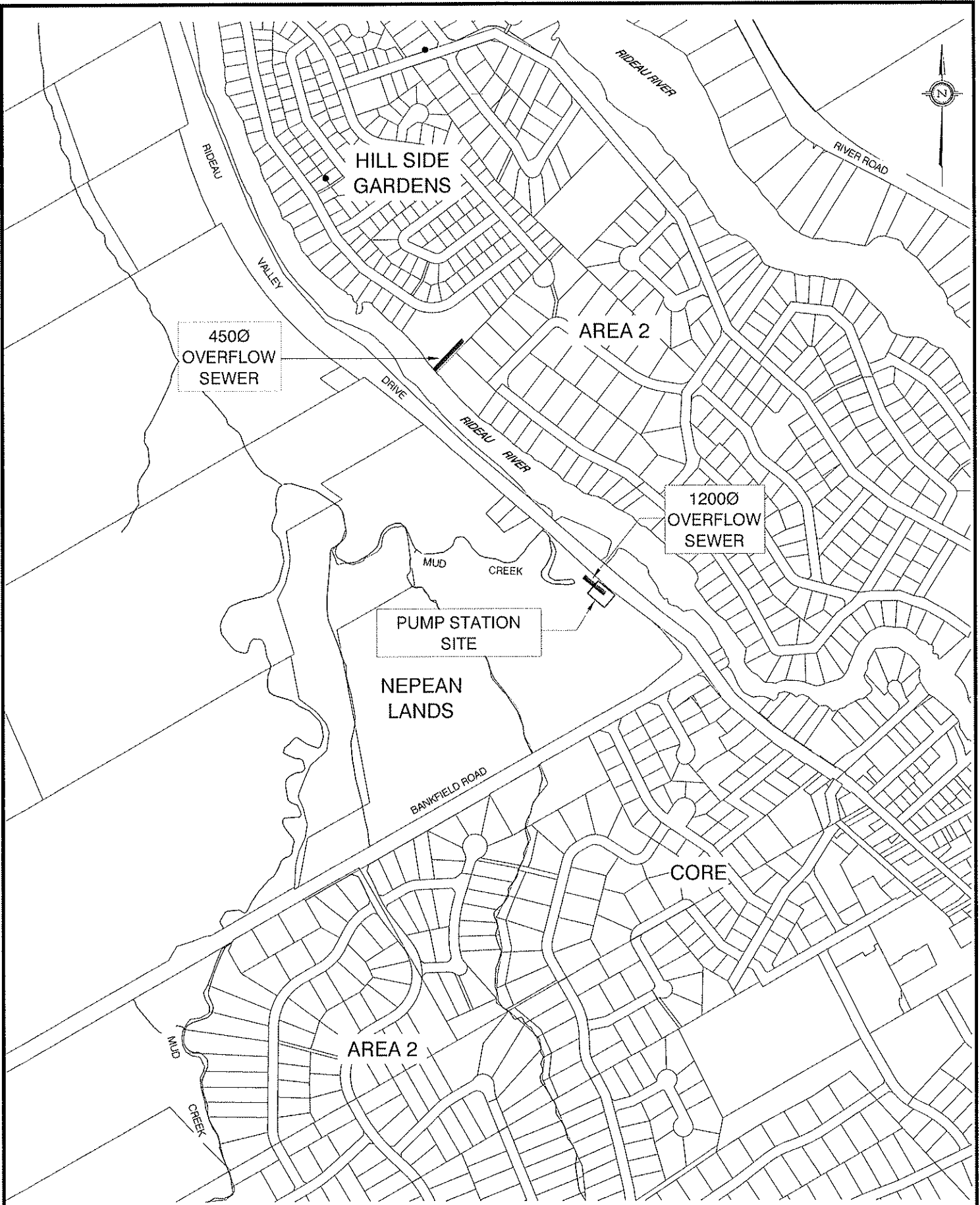
Even with the automatically controlled back up power source, the City prefers to add a third level of operation to further ensure that sewers will not surcharge to the extent that buildings and houses connected to the system are flooded. Therefore, the potential to provide an overflow to the adjacent 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_Monotick\5.9 Drawings\59civil\current\Design Brief-Sept. 2008\FIG 2-5.dwg Sheet Set: ###
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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		5497 Dickinson Circle	83.96	84.73	0.77	Pumped	
	258	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

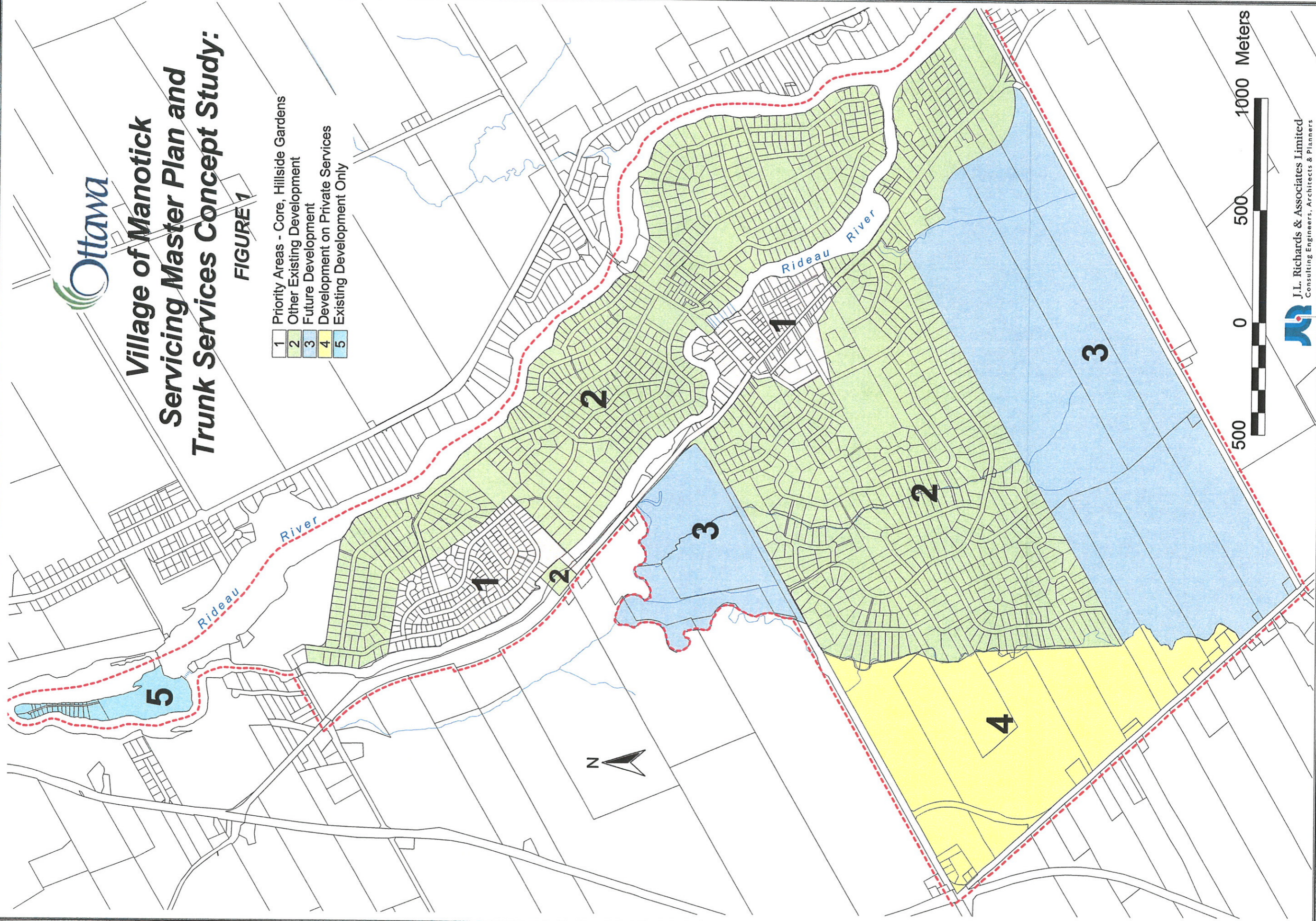
Manotick Service Areas



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

FIGURE 1

- 1 Priority Areas - Core, Hillside Gardens
- 2 Other Existing Development
- 3 Future Development
- 4 Development on Private Services
- 5 Existing Development Only



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 HARMONNON-RESIDENTIAL PEAKED AT 1.0)

2010 (Core And Hillside Gardens partially hooked up)	DESIGN (l/s)										MONITORED (l/s)											
	UNITS		AREA		POP		AVG W/O I/I		PEAK		AVG DWF W/O I/I		PK DWF		TYPICAL		ANNUAL WWF		RARE WWF			
	UNITS	AREA	POP	AREA	POP	AVG W/O I/I	PEAK	AVG DWF W/O I/I	PEAK	AVG DWF W/O I/I	PK DWF	PK DWF	TYPICAL	TYPICAL	ANNUAL WWF	ANNUAL WWF	RARE WWF	RARE WWF				
RESIDENTIAL/PAR	175	30	595	2.41	15.08	2.07	3.07	5.06	7.06	9.66	12.06	575	81.01	1947	7.89	51.03	6.76	10.81	16.20	24.30	34.83	44.55
COMM/INSTIT		15	0	8.68	13.02	2.60	3.35	3.35	4.85	6.80	8.60		31.94	18.48	36.67	5.55	7.14	10.34	14.49	18.32		
TOTALS	175	35	595	11.09	28.10	4.67	6.42	8.42	11.92	16.47	20.67	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		POP		AVG W/O I/I		PEAK		AVG DWF W/O I/I		PK DWF		TYPICAL		ANNUAL WWF		RARE WWF			
	UNITS	AREA	POP	AREA	POP	AVG W/O I/I	PEAK	AVG DWF W/O I/I	PEAK	AVG DWF W/O I/I	PK DWF	PK DWF	TYPICAL	TYPICAL	ANNUAL WWF	ANNUAL WWF	RARE 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	1138	148.61	3868	15.67	94.04	13.43	20.86	29.90	44.76	64.08	81.91
COMM/INSTIT		31.94	0	18.48	36.67	5.55	7.14	7.14	10.34	14.49	18.32		31.94	18.48	36.67	5.55	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	1137.5	180.55	3868	34.15	130.71	18.97	28.00	37.04	55.10	78.57	100.23

2030 (Core, Hillside Gardens, Minto and 100% built out)	DESIGN (l/s)										MONITORED (l/s)											
	UNITS		AREA		POP		AVG W/O I/I		PEAK		AVG DWF W/O I/I		PK DWF		TYPICAL		ANNUAL WWF		RARE WWF			
	UNITS	AREA	POP	AREA	POP	AVG W/O I/I	PEAK	AVG DWF W/O I/I	PEAK	AVG DWF W/O I/I	PK DWF	PK DWF	TYPICAL	TYPICAL	ANNUAL WWF	ANNUAL WWF	RARE 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	2837	574.54	9646	39.07	276.93	33.49	62.22	78.47	135.92	210.61	279.56
COMM/INSTIT		31.94	0	18.48	36.67	5.55	7.14	7.14	10.34	14.49	18.32		39.04	22.59	44.82	6.78	8.73	12.63	17.71	22.39		
TOTALS	1440	214.35	4896	38.32	152.27	22.55	33.26	43.92	65.35	93.22	118.94	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)	300	350
Peak Factor	Harmon (K)	Harmon (K=1.0)
Unit Population		3.4
ICI (L/hard)	15000	50000
Average	1 (non-simultaneous peak)	1.5
Peak Factor		
Inflow/Infiltration (L/hr/s)		
Dry Weather Inflow (Low)	0.05	0.08
Wet Weather Event (Typ)	0.15	0.20
Wet Weather Event (Annual)	0.28	0.30
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

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 DESIGN FLOW (l/s)	PROPOSED SEWER								
STREET	FROM MH	TO MH	RESID. UNITS			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)	CAP. l/s	PIPE (mm)	LGTH. (m)	SLOPE %	VEL. (full) m/s	AVAIL. CAP. (l/s)	AVAIL. CAP. (%)
			Singles	Towns Semis	Condo																						
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%



SANITARY FLOW PROJECTION-INTERIM AND ULTIMATE

Manotick Municipal Servicing
Main Sanitary Sewage Pump Station
City of Ottawa

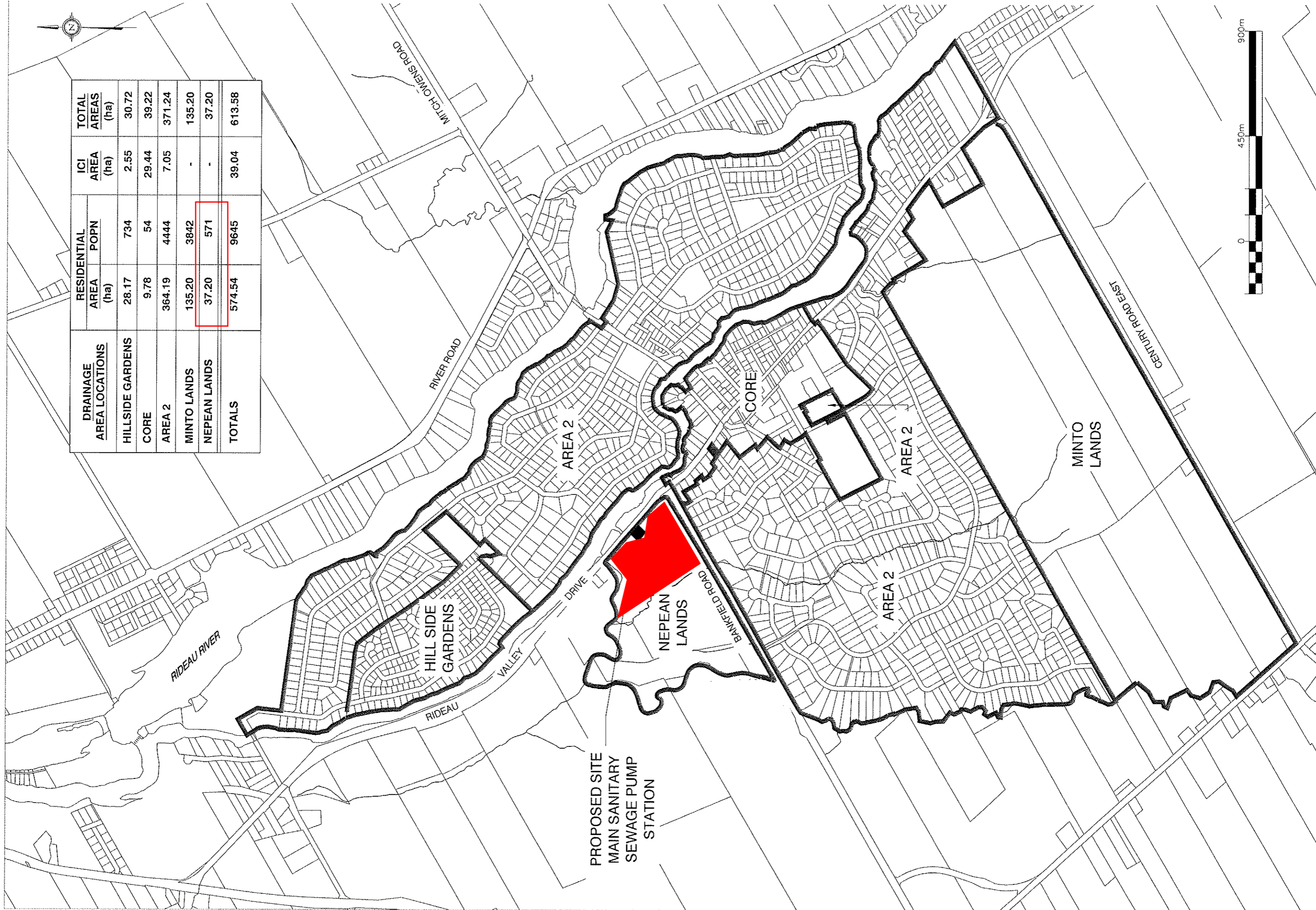
LOCATION		INDIVIDUAL RESIDENTIAL					CUMULATIVE RESID'L FLOW				CUMULATIVE ICI FLOW			INFILTRATION ALLOWANCE			TOTAL	PROPOSED SEWER DESIGN						
Street/Area	From MH	To MH	Area for Pop. (Ha)	Population	ICI Area (Ha.)	Park/OS (Ha.)	Population	Avg. Flow (l/s)	Peaking Factor	Peak Flow (l/s)	Area	Pk. Fact.	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 (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

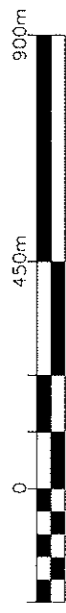
Assumed pipe loss coefficient = 0.013

Revised: Apr-08
 Revised: Sep-08

J:\1931 - Manotick\3.9 Drawings\50\ev\current\Design Brief-Sept. 2008\FIG 2-5.dwg Sheet Set: ###
 Plot Style: AIA STANDARD COLOR-HALF CTB Plot Scale: 1:1 Plotted At: Sep 28, 08 12:37 PM Printed By: DON SIURMA Lost Saved At: Sep 23, 08



DRAINAGE AREA LOCATIONS	RESIDENTIAL AREA		ICI AREA (ha)	TOTAL AREAS (ha)
	AREA (ha)	POPN		
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

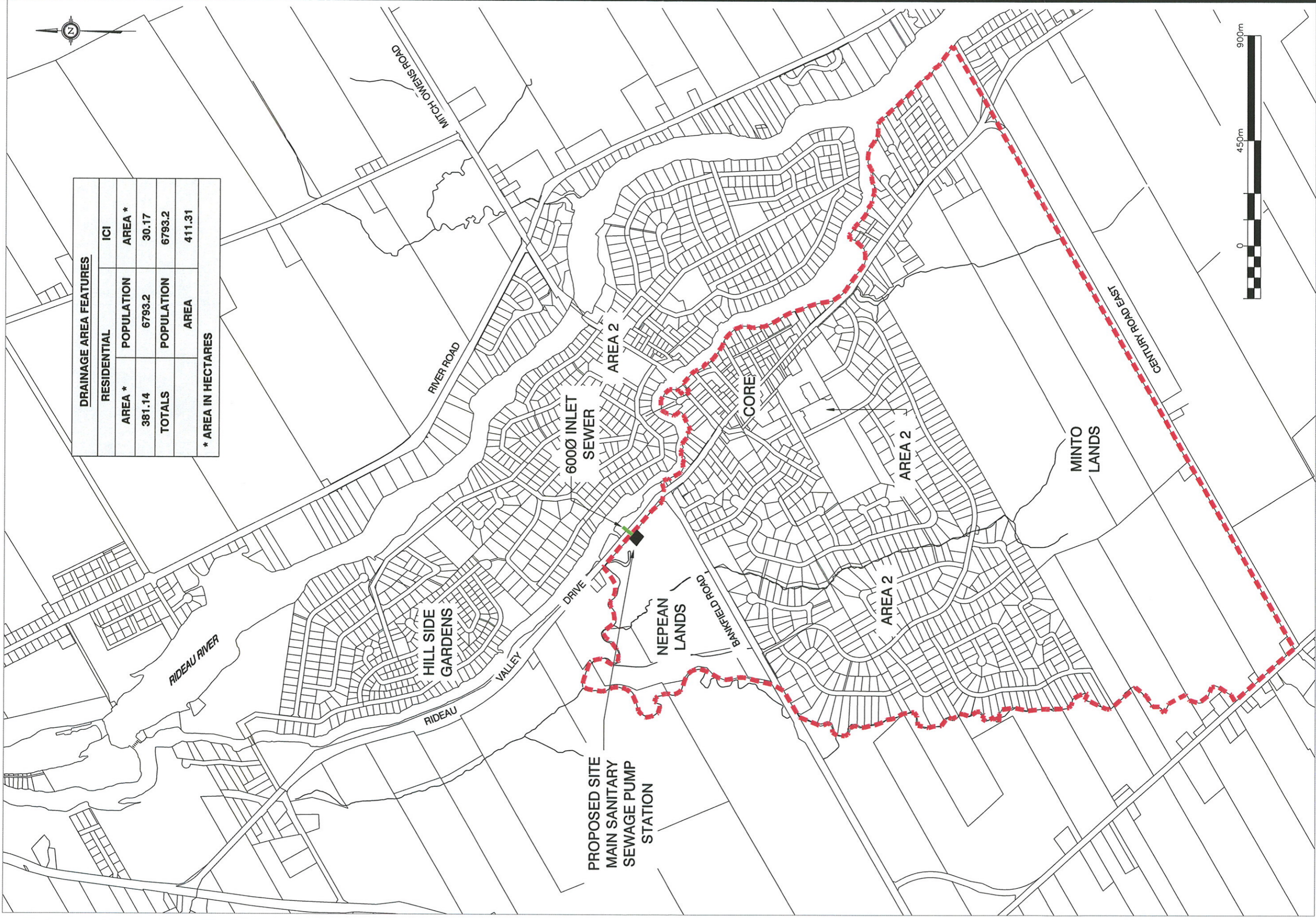


Project Title
DESIGN BRIEF
 MANOTICK MUNICIPAL SERVICING
 MAIN SANITARY SEWAGE PUMP STATION
 CITY OF OTTAWA

Drawing Title
SANITARY DRAINAGE AREAS

Project No. 11931
 Date SEPT. 2008
 FIGURE 3

J:\11931_Monotick\5.9 Drawings\59\civil\current\Design Brief-Sept. 2008\FIC 2-5.dwg Sheet Set: ###
 Plot Style: AIA STANDARD COLOR-HALF.CTB Plot Scale: 1:1 Plotted At: Sep. 29, 08 2:16 PM Printed By: DON SIURNA Last Saved By: ---- Last Saved At: Sep. 29, 08



DRAINAGE AREA FEATURES			
RESIDENTIAL		ICI	
AREA *	POPULATION	AREA *	POPULATION
381.14	6793.2	30.17	6793.2
TOTALS		TOTALS	
AREA		AREA	
* AREA IN HECTARES		* AREA IN HECTARES	
411.31		411.31	



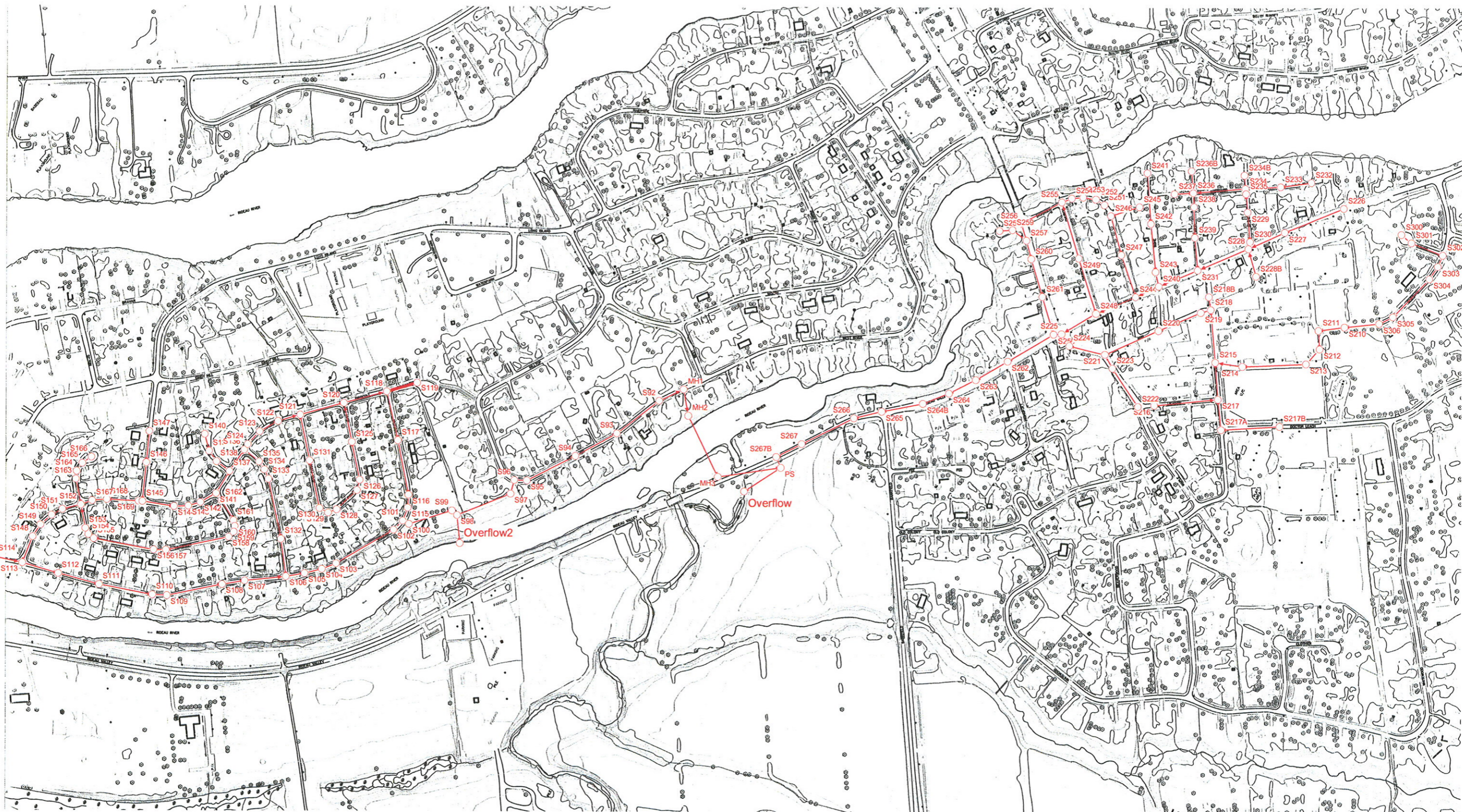
Project Title
DESIGN BRIEF
 MANOTICK MUNICIPAL SERVICING
 MAIN SANITARY SEWAGE PUMP STATION
 CITY OF OTTAWA

Drawing Title
DRAINAGE AREA PLAN
 INLET SEWER

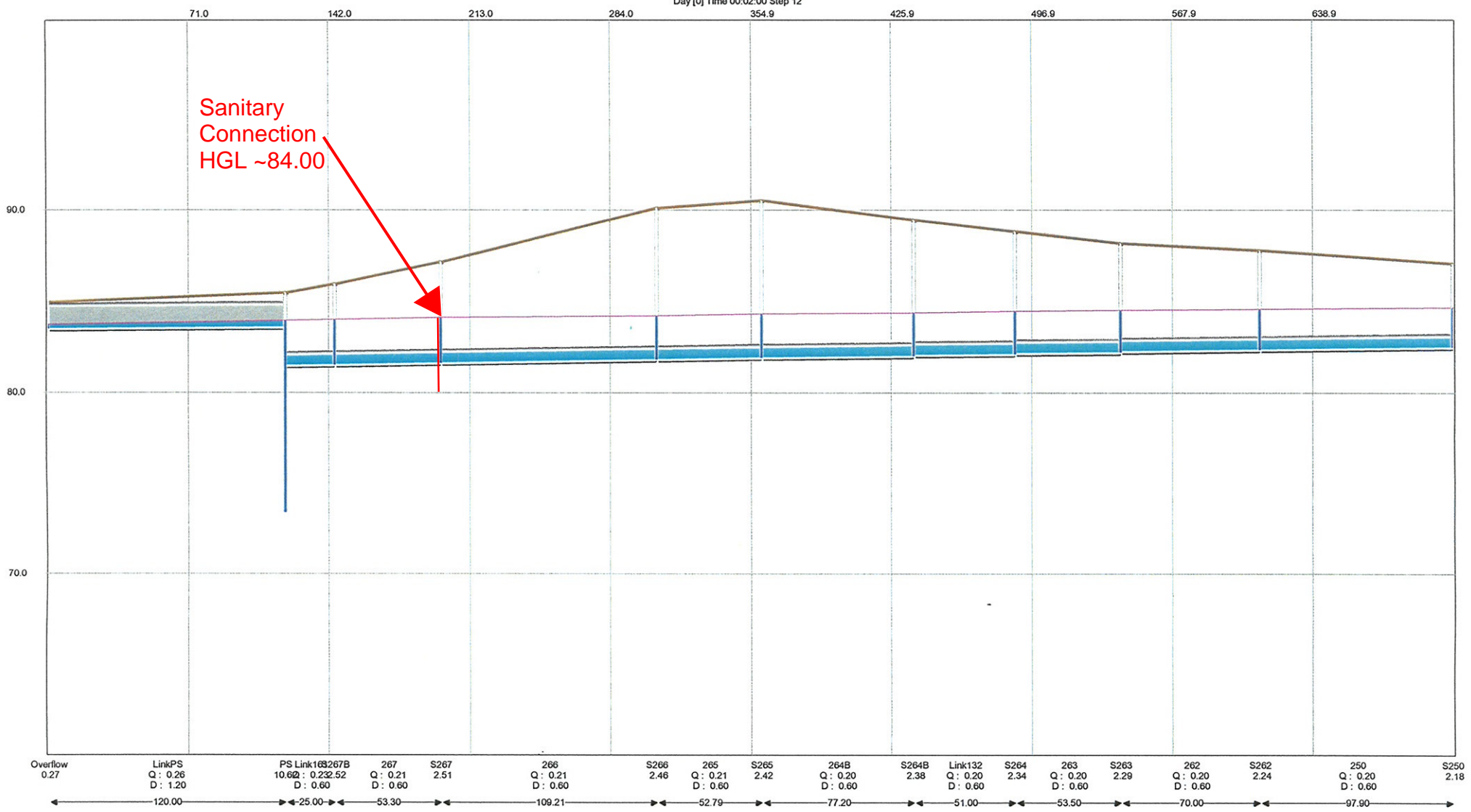
Project No. 11931
 Date SEPT. 2008
 FIGURE 6

APPENDIX D

Emergency Overflow Plan and Profiles



Overflow to S250
Day [0] Time 00:02:00 Step 12



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



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	C	Construction Material		Multiplier		1.5
		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	A	Floor Area				35,000
		Building Footprint (m ²)	5655		11,310	
		Number of Floors/Storeys	2			
		Protected Openings (1 hr) if C<1.0	No			
	F	Base fire flow without reductions				
		$F = 220 C (A)^{0.5}$				
Reductions or Surcharges						
3	(1)	Occupancy hazard reduction or surcharge		FUS Table 3	Reduction/Surcharge	29,750
		Non-combustible			-25%	
		Limited combustible	Yes		-15%	
		Combustible			0%	
		Free burning			15%	
		Rapid burning			25%	
4	(2)	Sprinkler Reduction		FUS Table 4	Reduction	0
		Adequately Designed System (NFPA 13)	No		-30%	
		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	(3)	Exposure Surcharge		FUS Table 5	Surcharge	2,975
		North Side	>30m		0%	
		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	550
					or	8,719

FUS - Fire Flow Calculations



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						
1	Construction Material		Multiplier		1.5	
	Coefficient related to type of construction C	Type V - Wood frame	Yes	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				4,400	22,000	
2	A	Building Footprint (m ²)	2200			
		Number of Floors/Storeys	2			
		Protected Openings (1 hr) if C<1.0	No			
F	Base fire flow without reductions					
Reductions or Surcharges						
3	Occupancy hazard reduction or surcharge		FUS Table 3	Reduction/Surcharge	18,700	
	(1)	Non-combustible		-25%		
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
Sprinkler Reduction		FUS Table 4	Reduction	0		
(2)	Adequately Designed System (NFPA 13)	No	-30%			
	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	4,675	
	(3)	North Side	20.1 - 30 m	10%		
		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
Date: 4/8/2024
Input By: Ben Sweet
Reviewed By: Sam Bahia
Drawing Reference: Fig 3.1 & 3.2

Legend: Input by User (Yellow) No Input Required (Grey)
 Calculated Cells → (Light Blue)

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																					
	Residential Input & Average Demand							Industrial / Commercial / Institutional (ICI) Input & Average Demand					Maximum Day & Peak Hour Demand					Design Fire Demand				
	Node	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)		
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		0.26	
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		0.35	
J06		30				81.00	0.26						0.00	2.50	1.50	0.66	5.50	2.70	1.44		0.66	
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		0.54	
J10	10	12				66.40	0.22						0.00	2.50	1.50	0.54	5.50	2.70	1.18		0.54	
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		0.22	
J14	7					23.80	0.08						0.00	2.50	1.50	0.19	5.50	2.70	0.42		0.19	
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		0.33	
J17	20					68.00	0.22						0.00	2.50	1.50	0.55	5.50	2.70	1.21		0.55	
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	Singles	Semis / Towns	Apts (2-BR)	Apts (1-BR)	Apts (Avg)
Population Equiv.	3.4	2.7	2.1	1.4	1.8
Daily 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
	300	3.60	5.50
	450	3.00	5.50
Modified	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
10,000	Low Density - Singles/Towns 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.		
20,000	High Density Wood Frame 4-Storey		
5,000	Fire-Resistive Podium/Multi-Storey		
30,000	High Contiguous / Hazard Areas		
< 45,000	Max FUS		

Maximum Pressure During Average Day (AVDY) Conditions

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
 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

Minimum Pressure During Peak Hour (PKHR) Conditions

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
Acceptable (\Rightarrow 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



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

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
 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 led 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/structure¹, measured in accordance with Table 1, shall be not less than the RFF.

4. Standard Practice

For the vast majority of developments, hydrant spacing as indicated in Section 4.5, Table 4.9, Ottawa Design Guidelines – Water Distribution, are sufficient to meet the RFF. This has been verified by evaluating approved development plans representing a

¹ Although NFPA 1 considers hydrant contribution at distances of up to 1000ft (305 m), Ottawa Fire Services (OFS) would need two pumpers to deliver flow from such a distance (one pumper midway – acting as a booster). Moreover, OFS cautioned that some redundancy is advisable to account for accessibility limitations in emergency situations, wind effects, etc. Therefore 150 m was considered as the maximum contributing distance

Appendix I: Guideline on Coordination of Hydrant Placement with Required Fire Flow

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.

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 m² area).

Appendix I: Guideline on Coordination of Hydrant Placement with Required Fire Flow

- 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.

Appendix I: Guideline on Coordination of Hydrant Placement with Required Fire Flow

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

Geotechnical Investigation

Proposed Residential Development

4386 Rideau Valley Drive
Ottawa, Ontario

Prepared for Uniform Developments

Report PG5828-1 Revision 4 dated April 4, 2024

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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.

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.

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.

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 Miradrain G100N or Delta Drain 6000.

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.

Backfilling Recommendations – Rear Yard of Lots 5 and 6

It is understood that backfilling the face of the slope within the rear yards of lots 5 and 6 to match the surrounding subgrade is being considered. The following backfilling program is recommended. Our backfilling recommendations take into consideration the subsoils profile within the working area at the top of the slope and the proposed finished grading at the proposed lots. The backfilling operation should be completed under dry weather conditions (specifically for the clay placement portion of the program) and above freezing temperatures.

- The side slopes within the area under consideration should be stepped to provide a 1.5H:1V profile with maximum 500 mm high steps. All existing topsoil, sediment and fill should be removed.
- A workable, stiff brown silty clay (moisture contents varying between 20 to 40%) should be placed in maximum 300 mm loose lifts **under dry conditions and in above freezing temperatures** to in-fill the slope. Every lift should be adequately compacted using a sheepsfoot roller and approved by Paterson personnel during placement. The side slopes should be restored to a finish grade to provide a minimum slope of 3H:1V or shallower.
- The grading of the slope should promote gravity drainage.
- It is vital to maintain vegetative cover to prevent long-term erosion.
- **Inspections During Construction:** Periodic inspections during the backfilling operation should be completed by Paterson personnel to confirm the above noted recommendations are adhered to.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

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.

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.

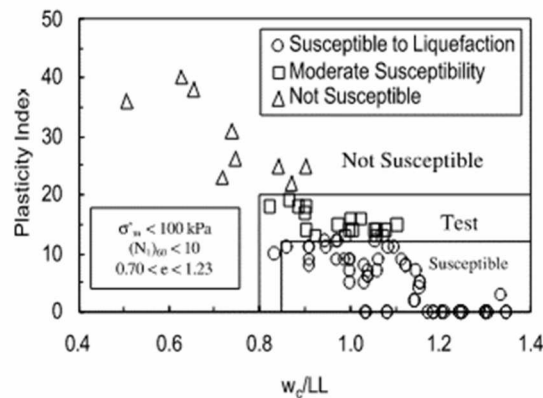


Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

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.

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. No active erosion or sloughing of the banks were observed at the time of our visit.

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.

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

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. The Wilson Cowan Drain did not appear to be affected by active erosion at the time of our site visit. Furthermore, it should be noted that the Wilson Cowan Drain is not considered to be a permanent natural watercourse and is generally dry for most of the year. Based on the anticipated soils, and the nature of the existing watercourse and drain, a toe erosion allowance of 5 m and 1 m should be applied from the watercourse edge for Mud Creek Watercourse and Wilson Cowan Drain, respectively.

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.

It should be further noted 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 is recommending 1m for toe erosion 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 5m for toe erosion as 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. 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.

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 m³ of available soil volume while a medium tree must be provided with a minimum of 30 m³ 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 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, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site 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.



Kevin A. Pickard, P.Eng.



David J. Gilbert, 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

9 Auriga Drive, Ottawa, Ontario, K2E 7T9

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

DATUM Geodetic

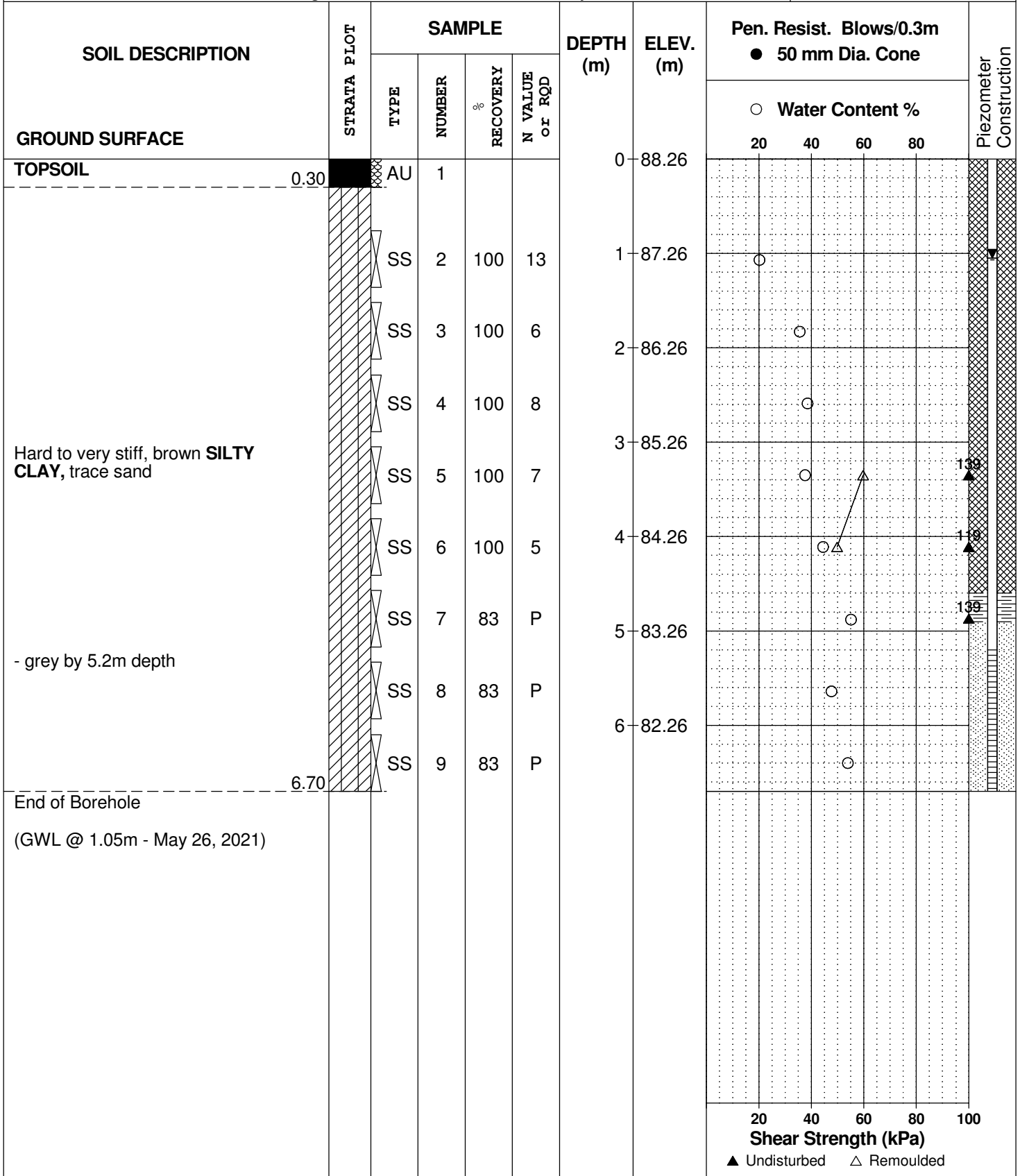
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REMARKS

HOLE NO. **BH 1-21**

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021



DATUM Geodetic

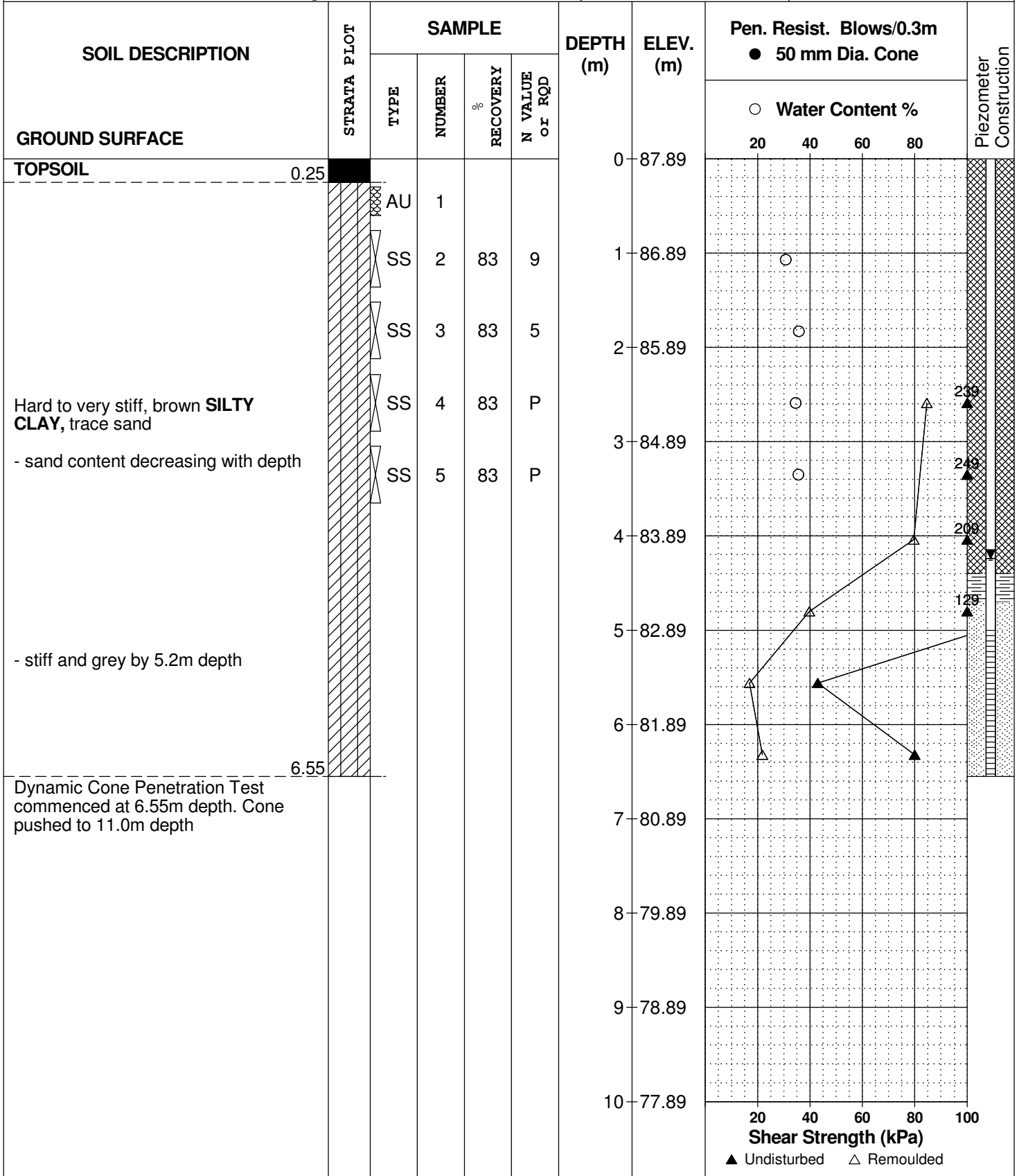
REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. **PG5828**

HOLE NO. **BH 3-21**



9 Auriga Drive, Ottawa, Ontario, K2E 7T9

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

DATUM Geodetic

FILE NO. **PG5828**

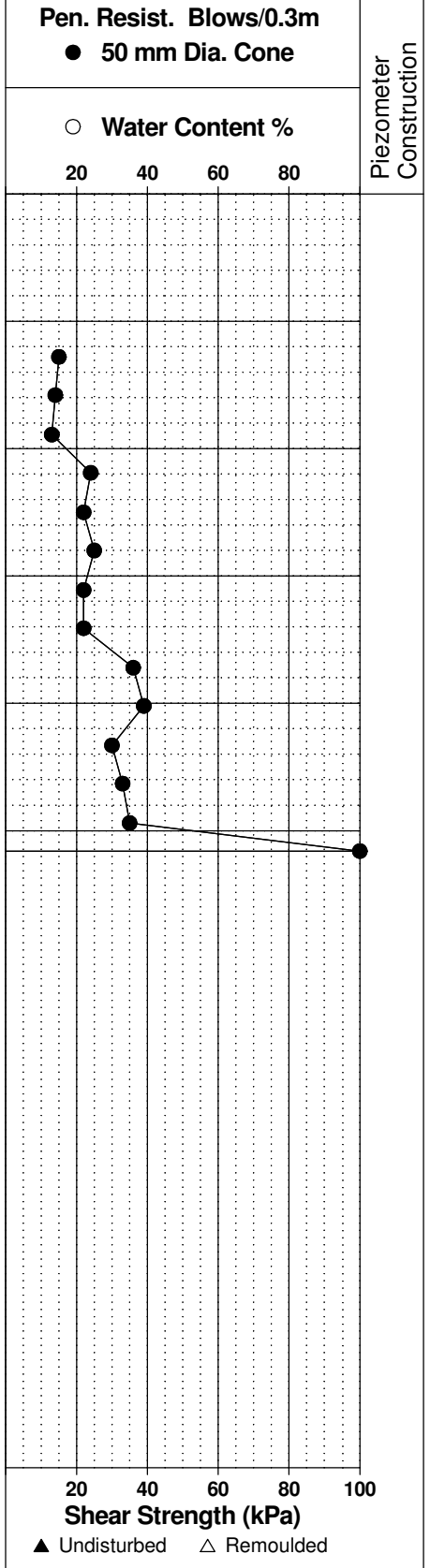
REMARKS

HOLE NO. **BH 3-21**

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

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



DATUM Geodetic

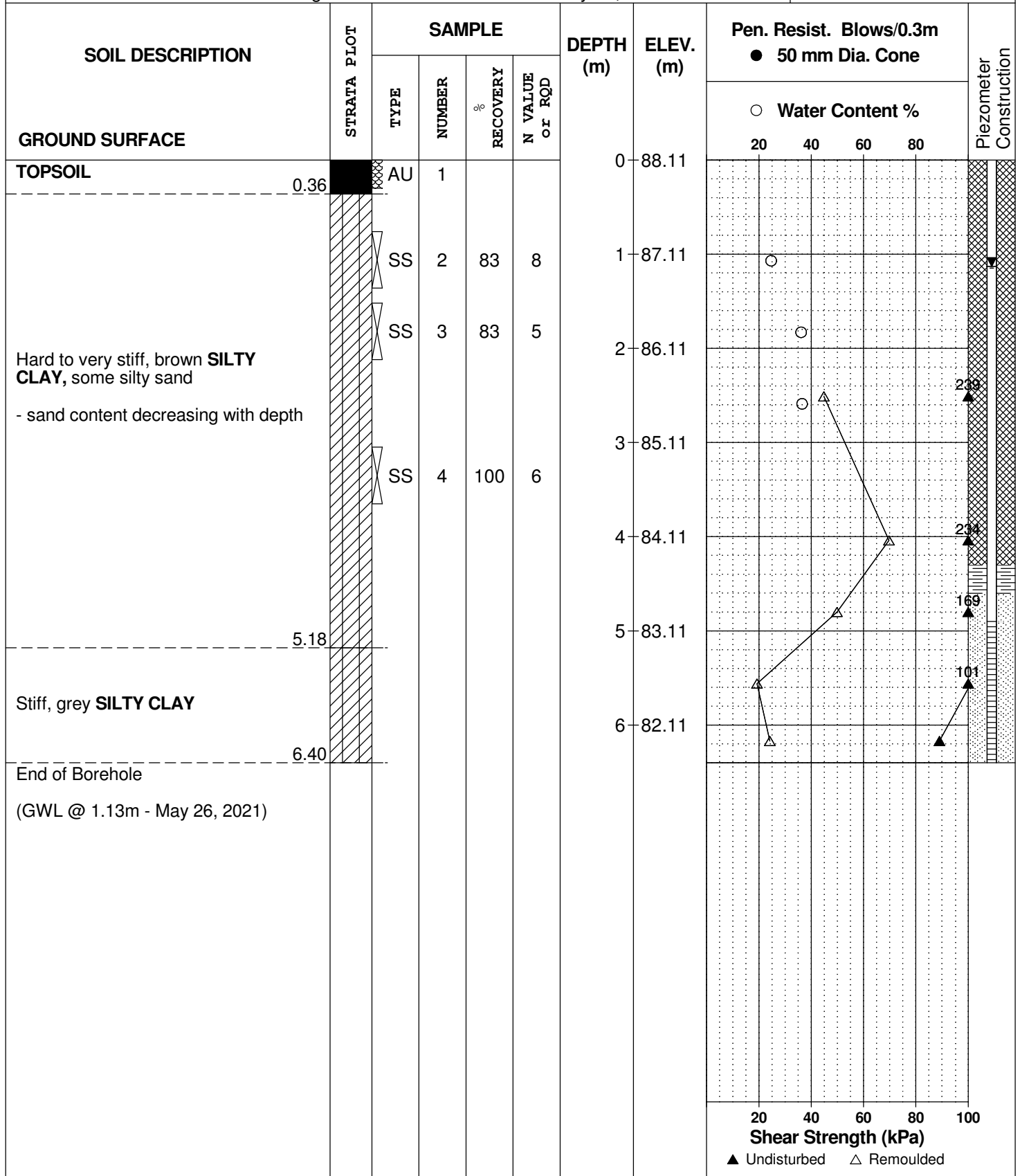
FILE NO. **PG5828**

REMARKS

HOLE NO. **BH 4-21**

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021



9 Auriga Drive, Ottawa, Ontario, K2E 7T9

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

DATUM Geodetic

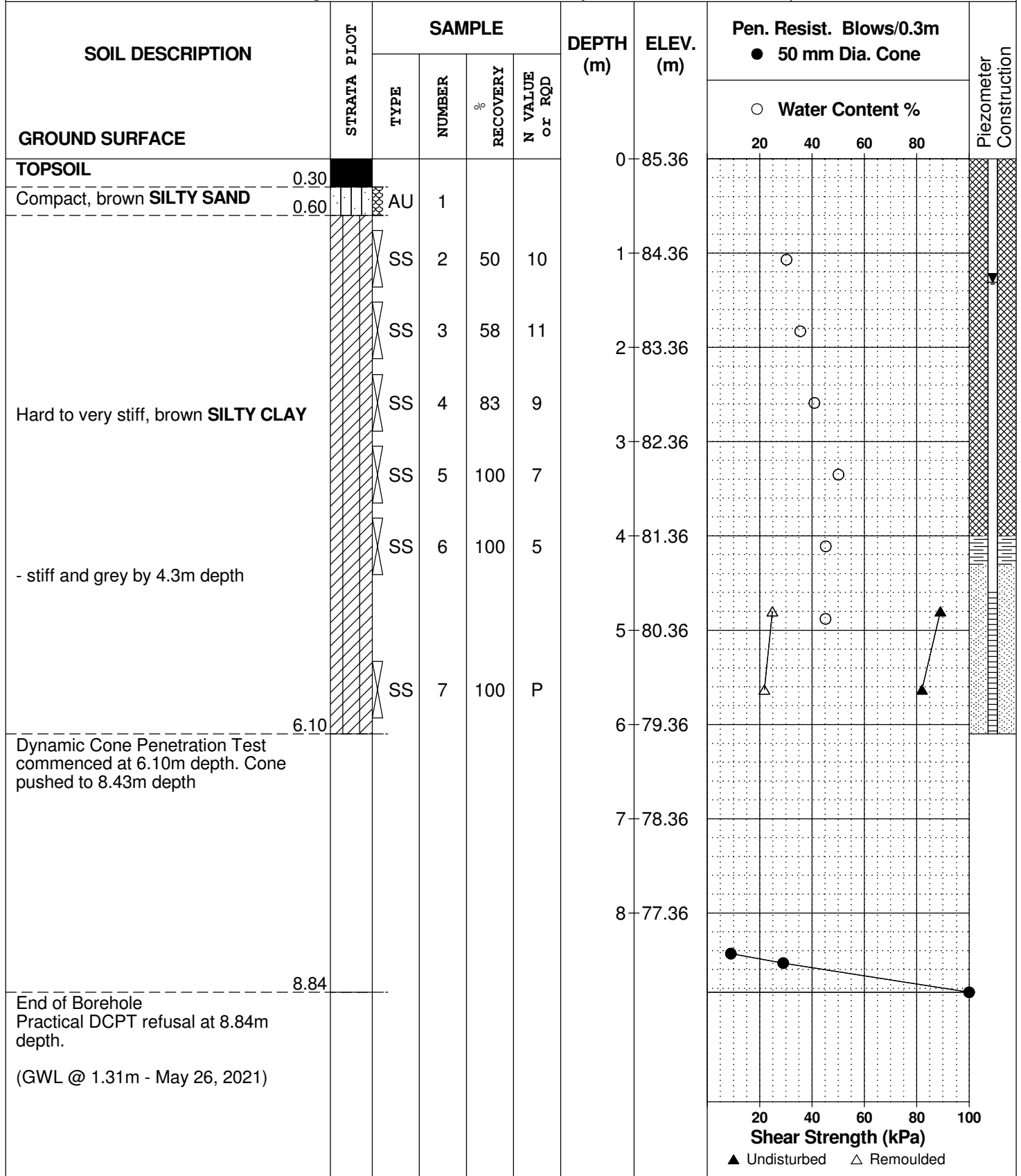
FILE NO. **PG5828**

REMARKS

HOLE NO. **BH 5-21**

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021



DATUM Geodetic

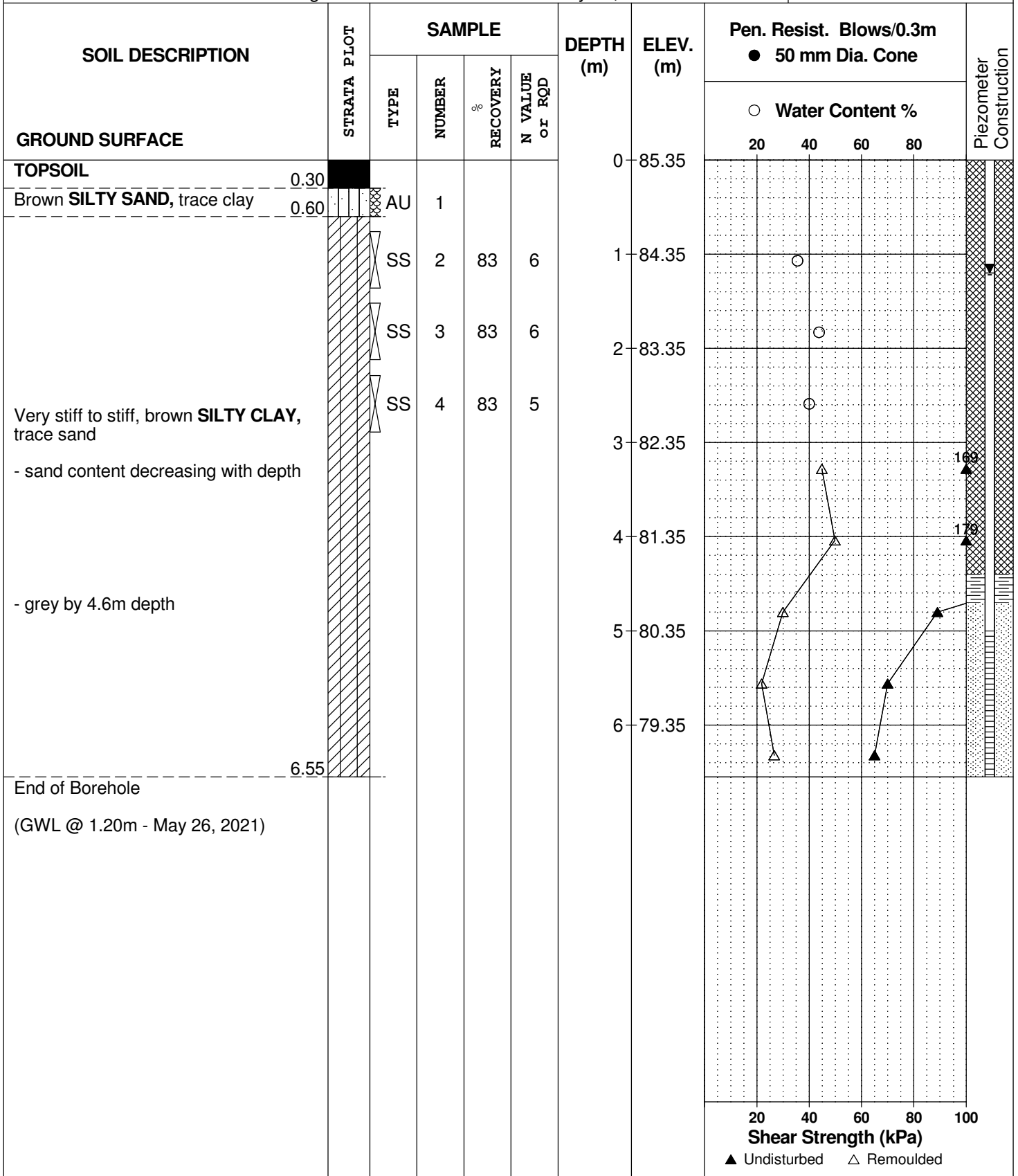
REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 19, 2021

FILE NO. **PG5828**

HOLE NO. **BH 6-21**



9 Auriga Drive, Ottawa, Ontario, K2E 7T9

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

DATUM Geodetic

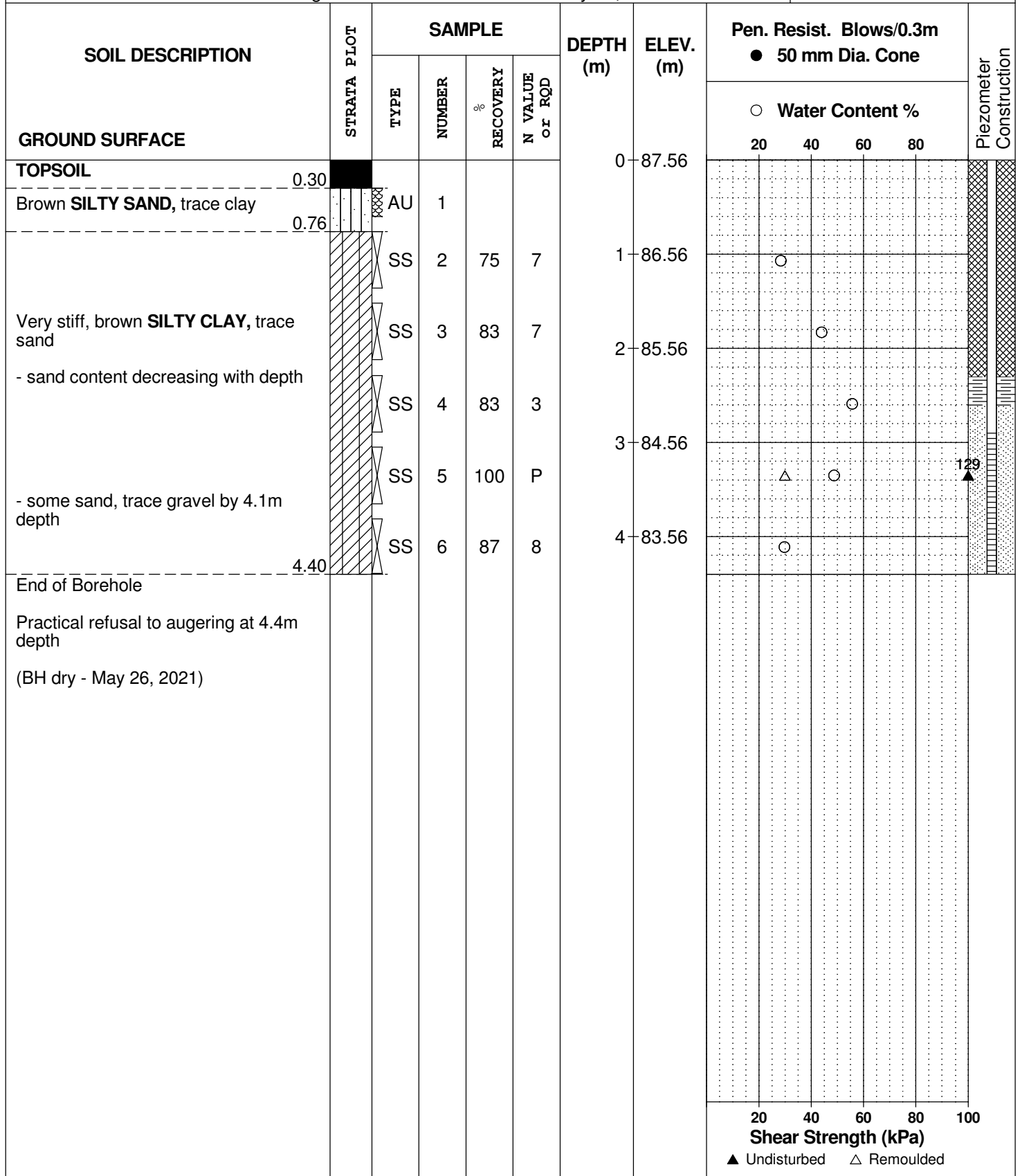
FILE NO. **PG5828**

REMARKS

HOLE NO. **BH 7-21**

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021



DATUM Geodetic

FILE NO. **PG5828**

REMARKS

HOLE NO. **BH 8-21**

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %					
GROUND SURFACE								20	40	60	80		
TOPSOIL	0.20					0	91.32						
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	4	87.32						
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders		SS	6	62	39	5	86.32						
		SS	7	50	27	6	85.32						
		SS	8	42	26	7	84.32						
		SS	9	42	21	8	83.32						
End of Borehole	6.70												
(GWL @ 2.90m - May 26, 2021)													

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

DATUM Geodetic

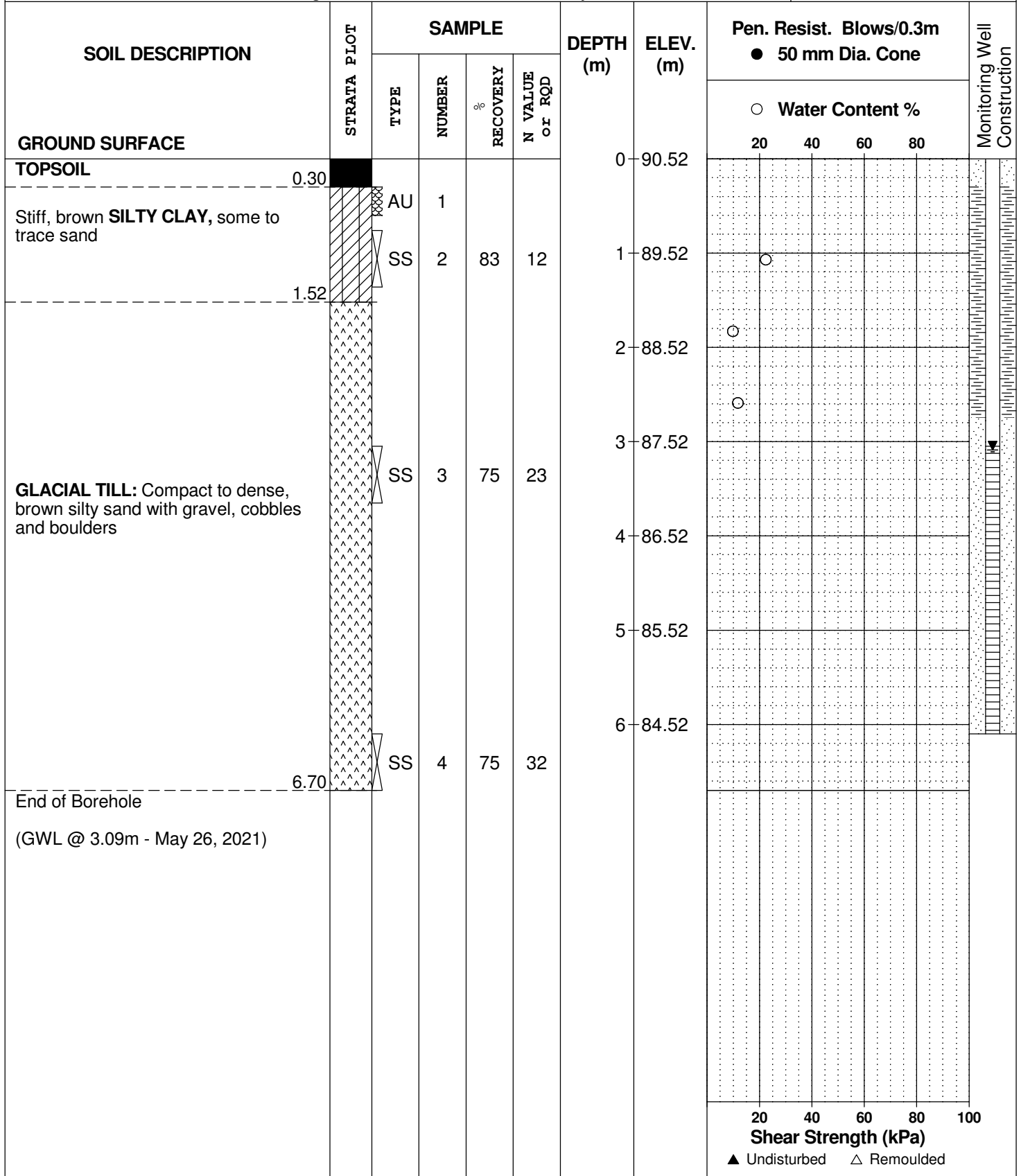
REMARKS

BORINGS BY Track-Mount Power Auger

DATE May 20, 2021

FILE NO. **PG5828**

HOLE NO. **BH 9-21**



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)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

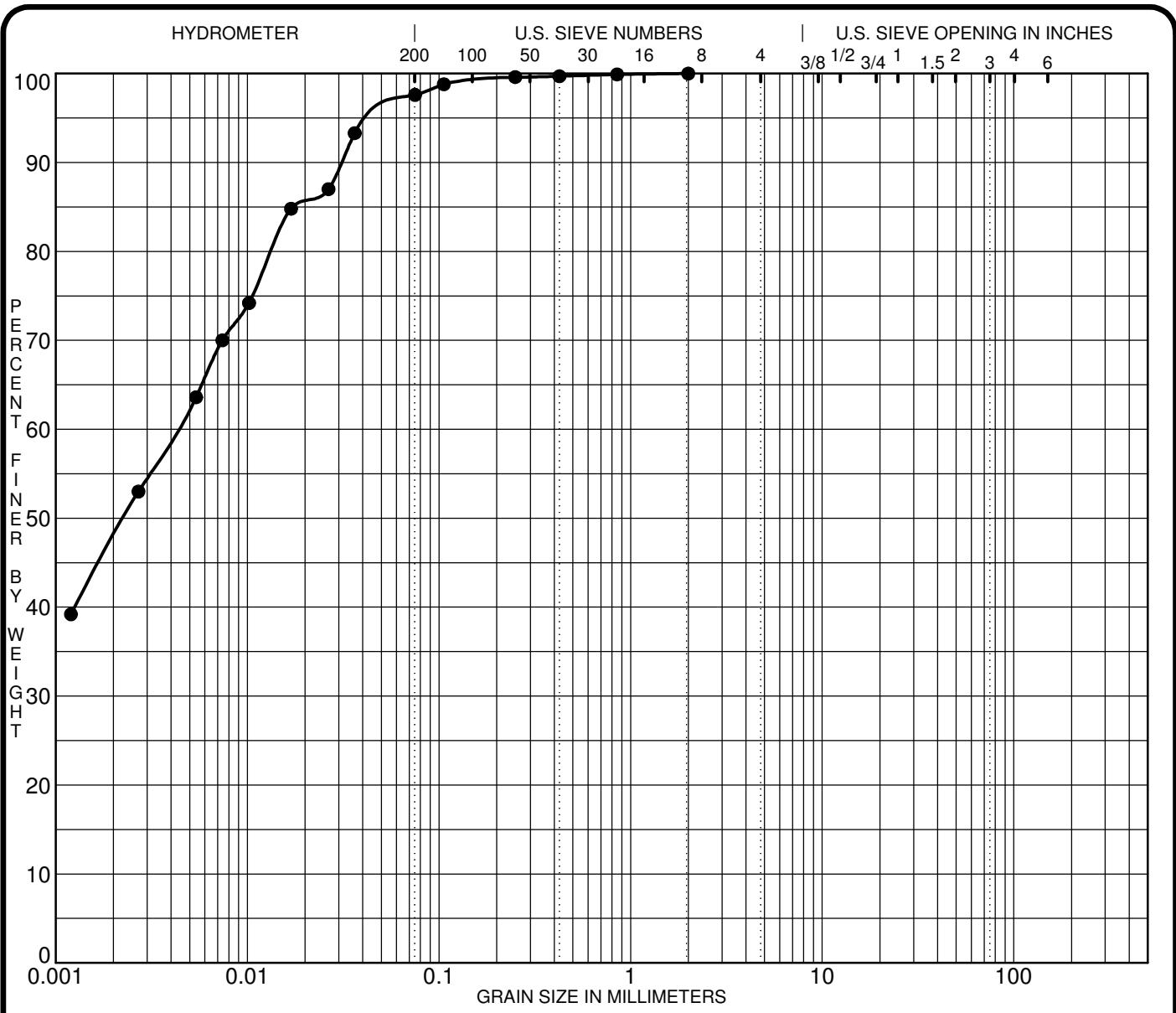
C_c and C_u are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

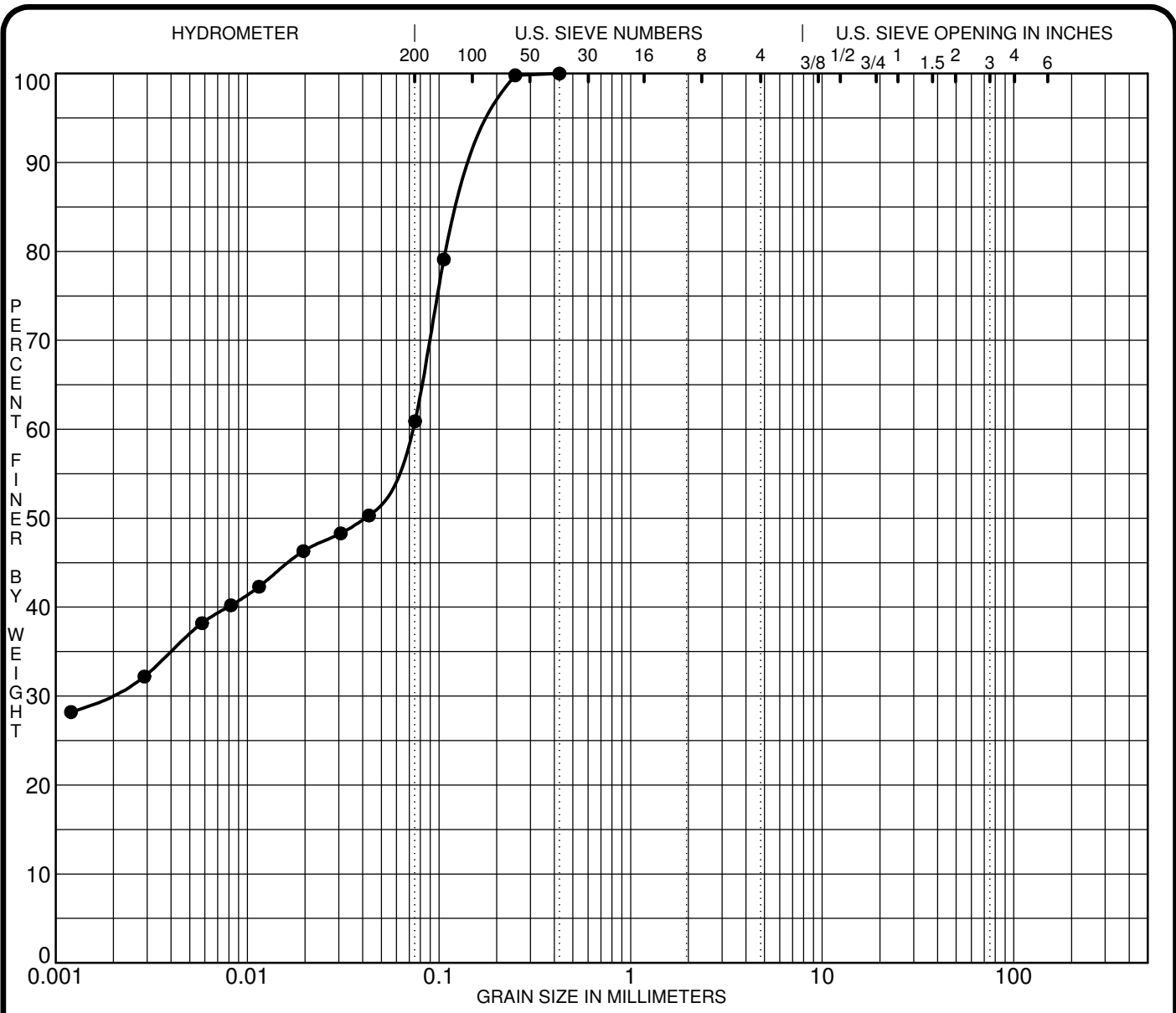
Specimen Identification	Classification				MC%	LL	PL	PI	Cc	Cu
● BH 1-21 SS 4	CH - Inorganic clays of high plasticity									
☒										
▲										
★										
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay		
● BH 1-21 SS 4	2.00	0.00			0.0	2.4	97.6			
☒										
▲										
★										

CLIENT Uniform Urban Developments
 PROJECT Geotechnical Investigation - Proposed Residential Development

FILE NO. PG5828
 DATE 19 May 21

paterosongroup Consulting Engineers
 9 Auriga Drive, Ottawa, Ontario, K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

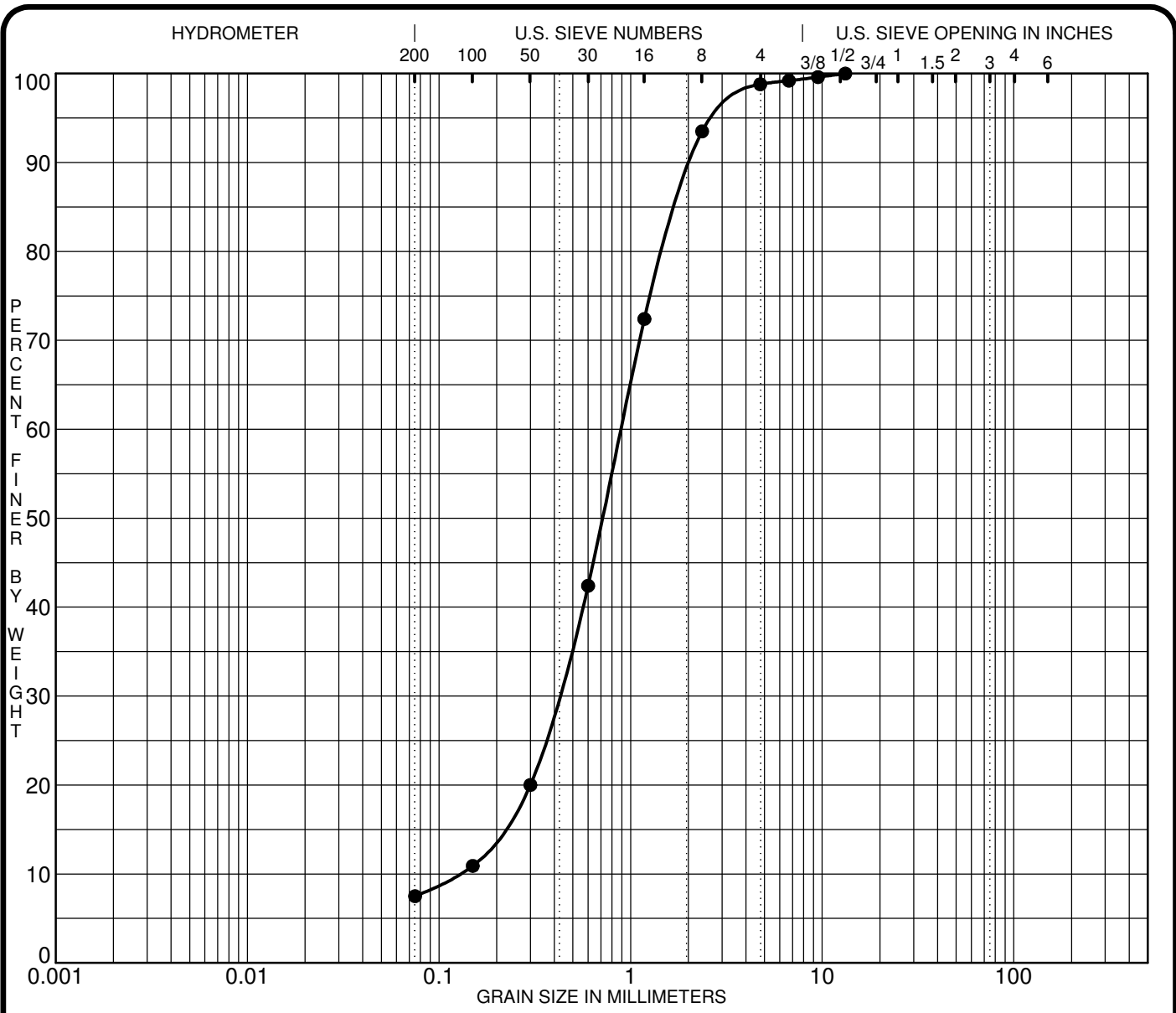
Specimen Identification	Classification				MC%	LL	PL	PI	Cc	Cu
● BH 4-21 SS 2	CL - Inorganic clays of low plasticity									
☒										
▲										
★										
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay		
● BH 4-21 SS 2	0.43	0.07	0.002		0.0	39.1	60.9			
☒										
▲										
★										

CLIENT Uniform Urban Developments
 PROJECT Geotechnical Investigation - Proposed Residential Development

FILE NO. PG5828
 DATE 19 May 21

paterongroup Consulting Engineers
 9 Auriga Drive, Ottawa, Ontario, K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

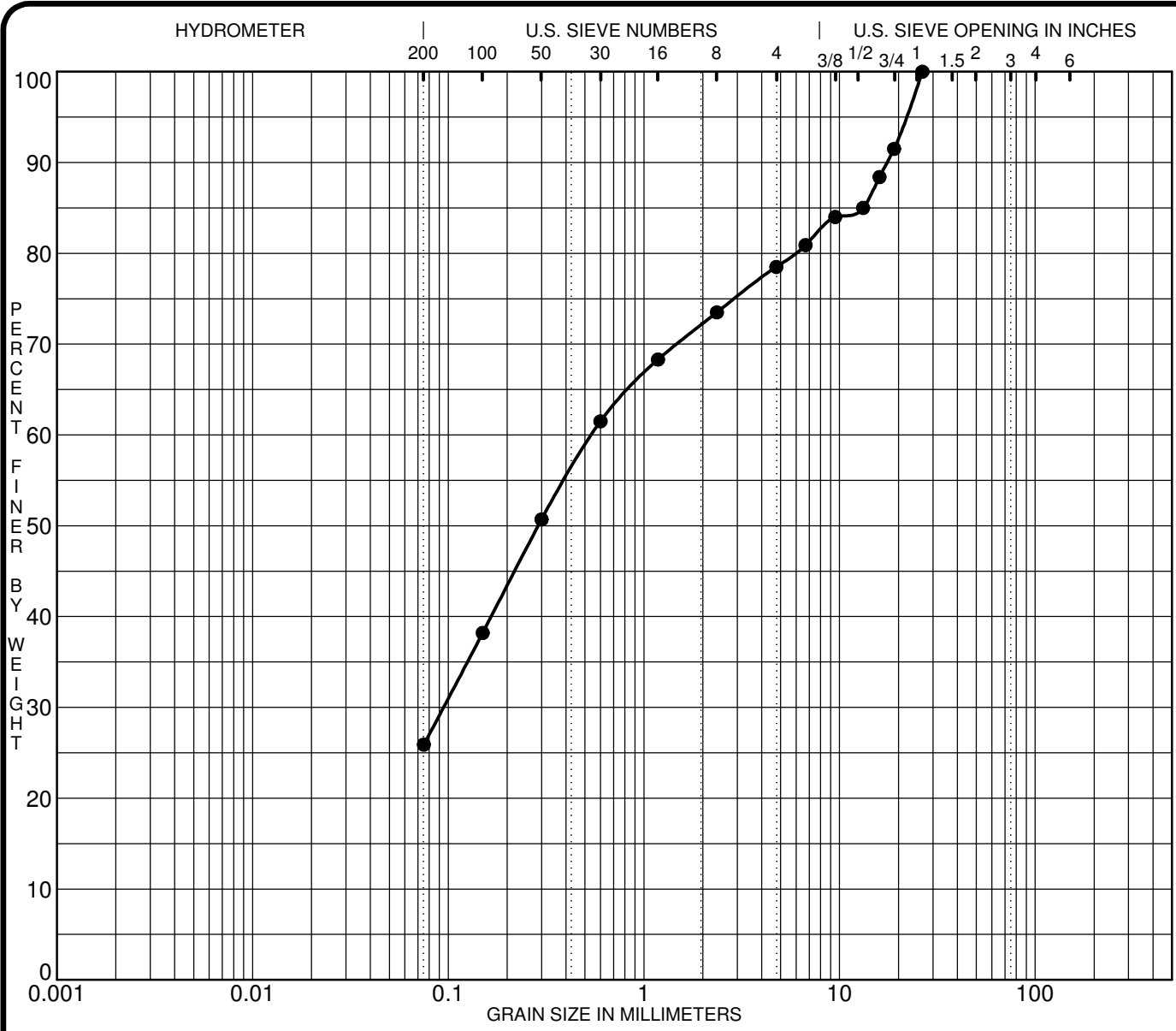
Specimen Identification	Classification					MC%	LL	PL	PI	Cc	Cu
● BH 6-21 SS 4	CH - Inorganic clays of high plasticity									1.50	7.1
☒											
▲											
★											
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay			
● BH 6-21 SS 4	13.20	0.89	0.409	0.1249	1.2	91.3	7.5				
☒											
▲											
★											

CLIENT Uniform Urban Developments
 PROJECT Geotechnical Investigation - Proposed Residential Development

FILE NO. PG5828
 DATE 19 May 21

paterosongroup Consulting Engineers
 9 Auriga Drive, Ottawa, Ontario, K2E 7T9

GRAIN SIZE DISTRIBUTION



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

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 FILE NO. PG5828
 PROJECT Geotechnical Investigation - Proposed Residential Development DATE 20 May 21

paterongroup Consulting Engineers
 9 Auriga Drive, Ottawa, Ontario, K2E 7T9

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Report Date: 28-May-2021

Client: Paterson Group Consulting Engineers

Order Date: 21-May-2021

Client PO: 29757

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

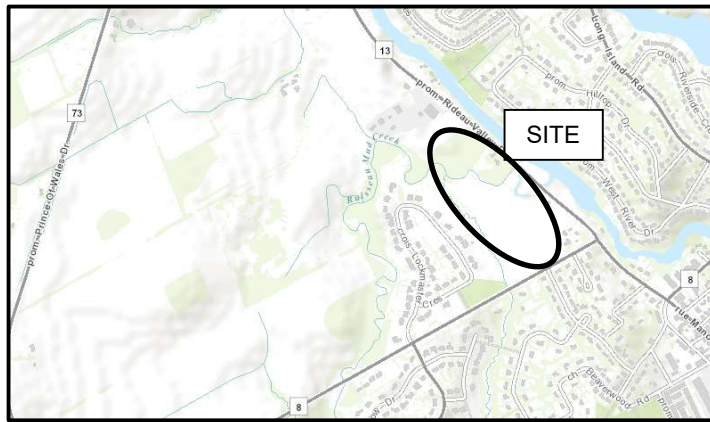


FIGURE 1

KEY PLAN

Figure 2 - Section A - Existing Conditions - Static Loading

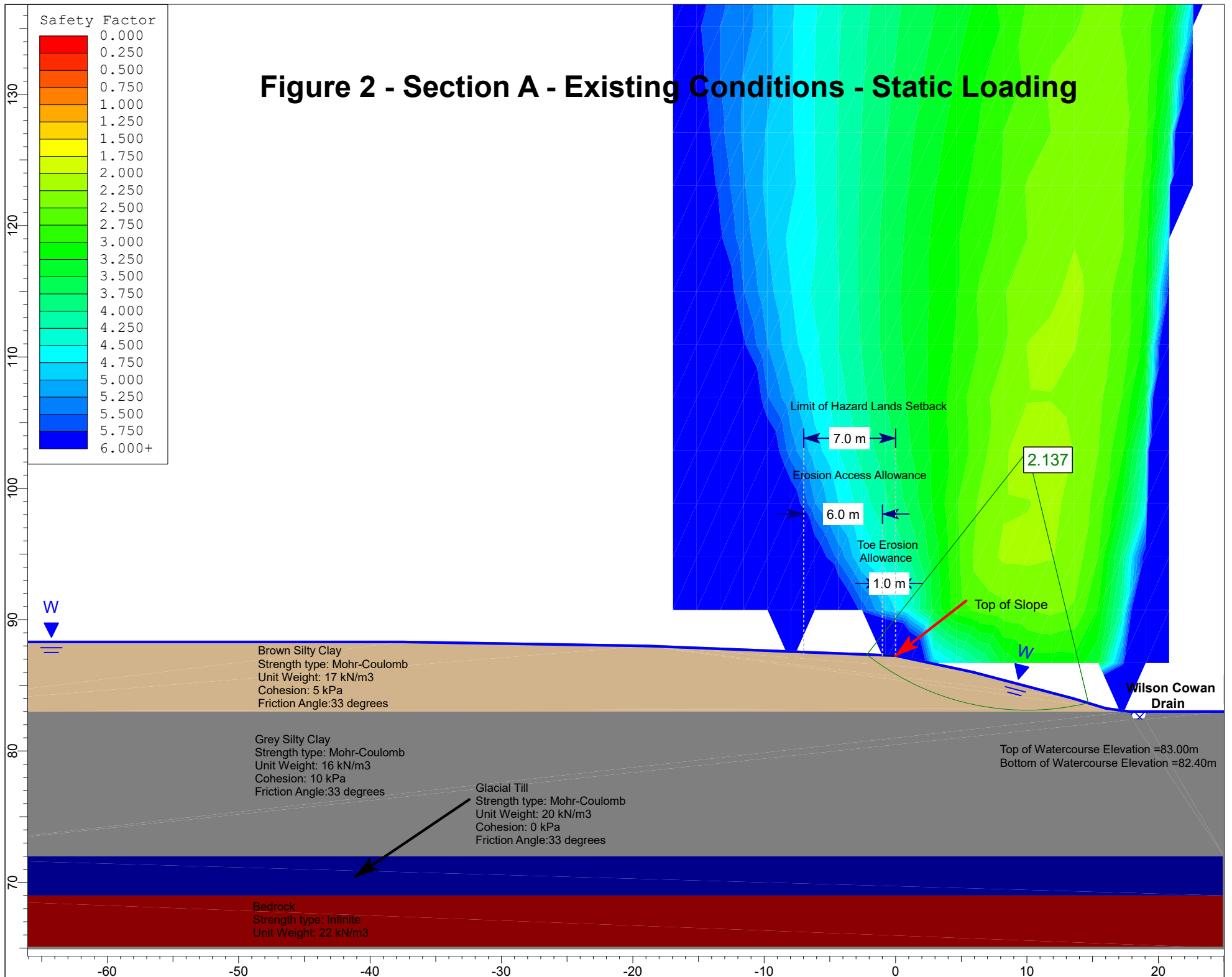


Figure 3 - Section A - Existing Conditions - Seismic Loading

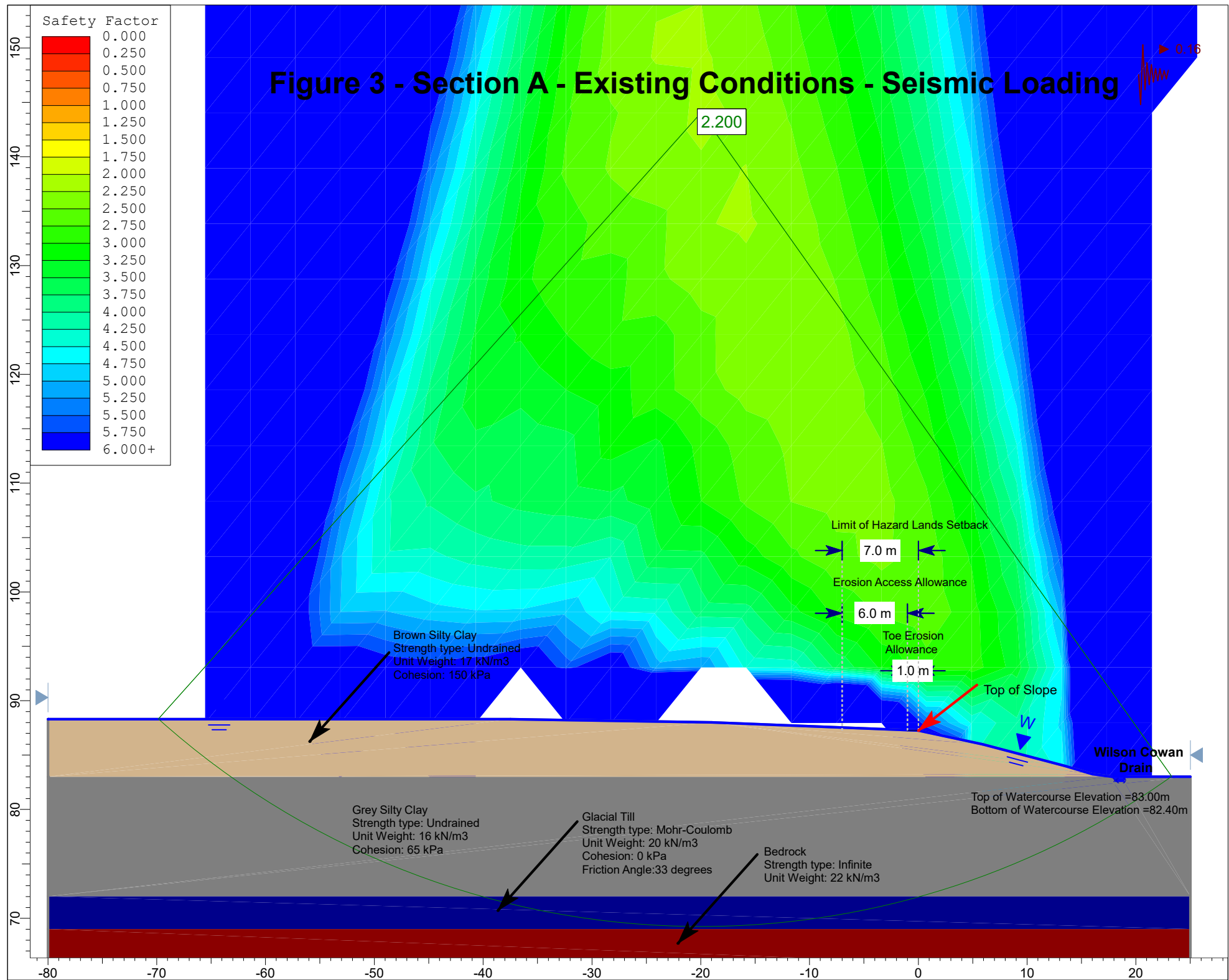


Figure 4 - Section B - Existing Conditions - Static Loading

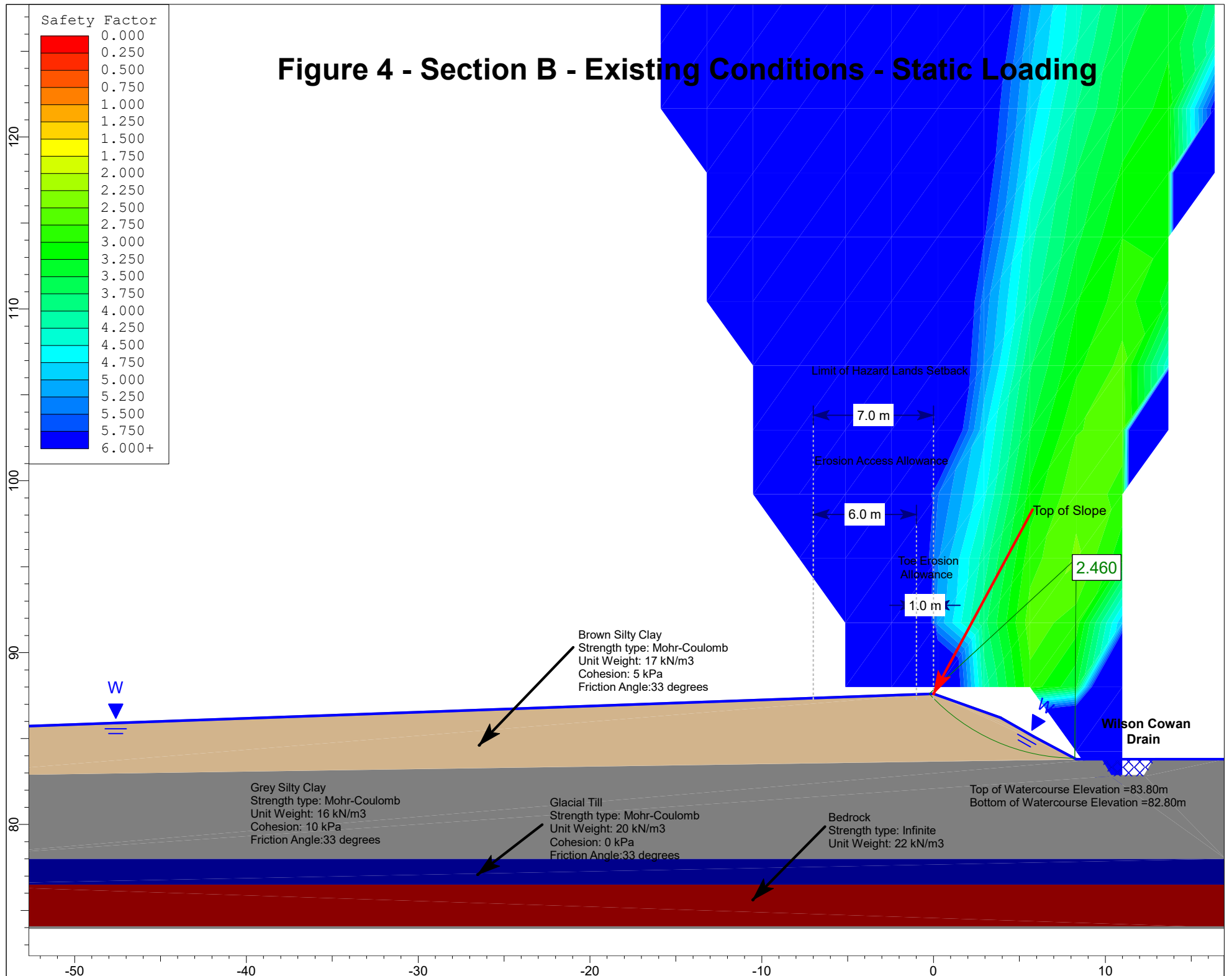


Figure 5 - Section B - Existing Conditions - Seismic Loading

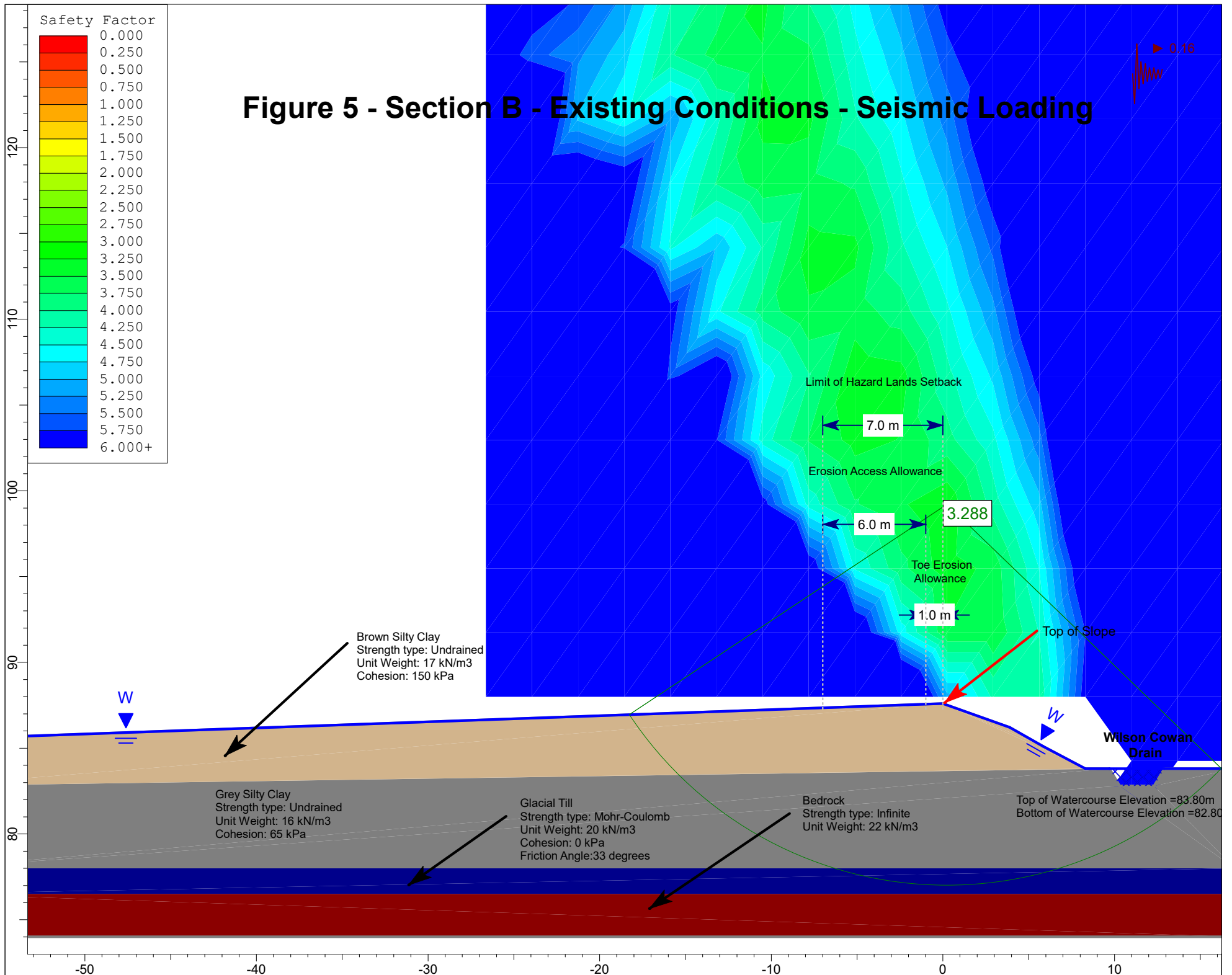


Figure 6 - Section C - Existing Conditions - Static Loading

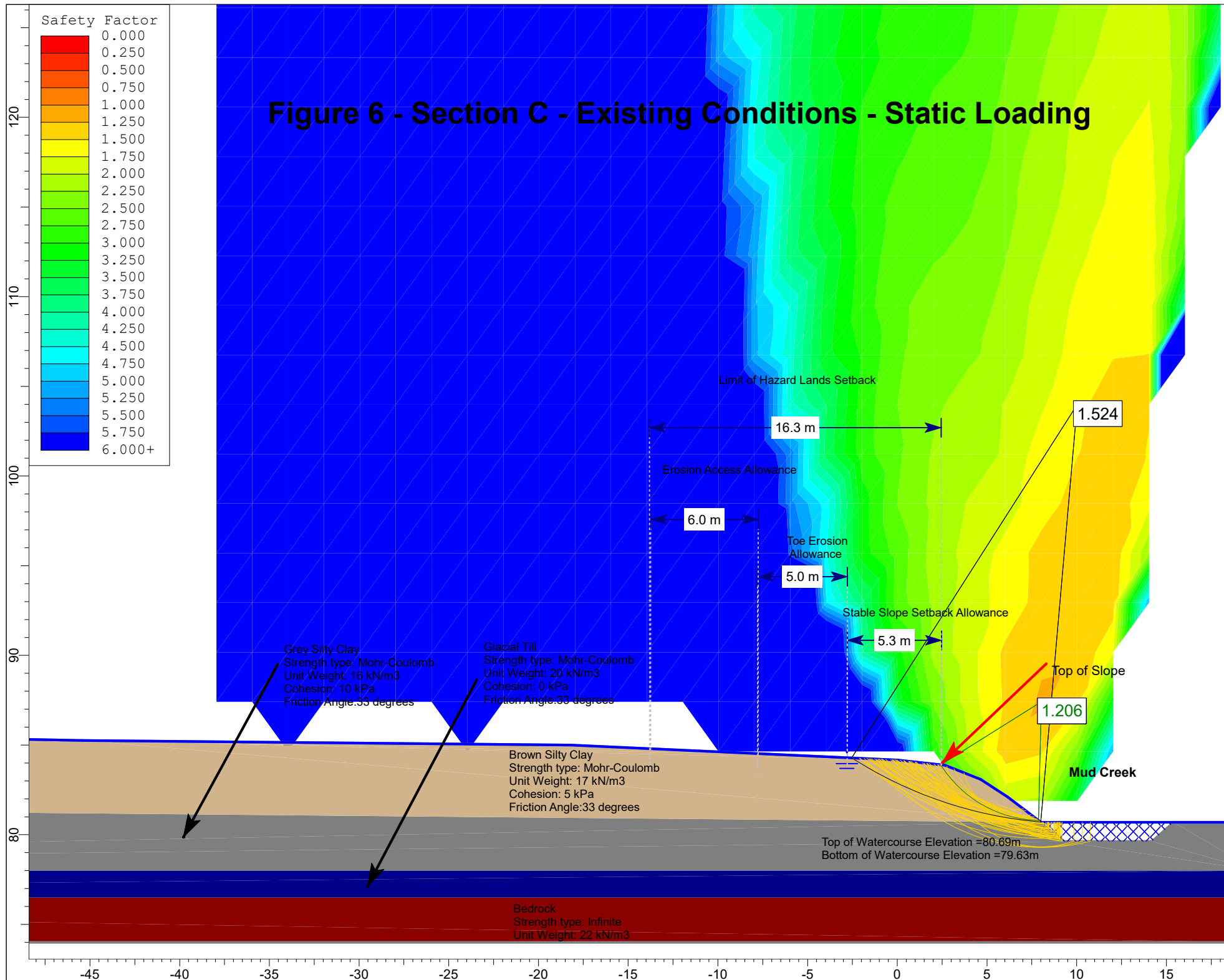


Figure 7 - Section C - Existing Conditions - Seismic Loading

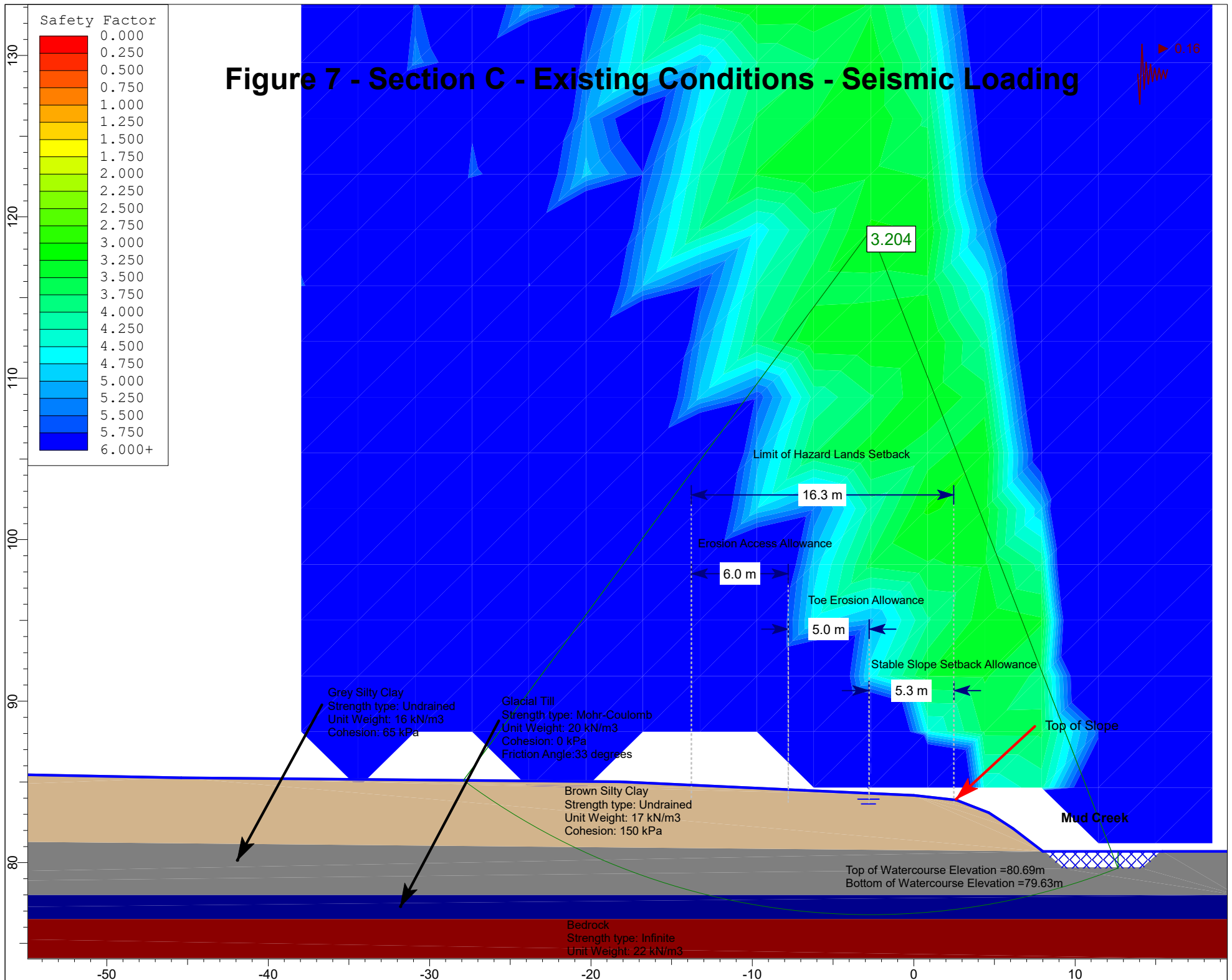


Figure 8 - Section D - Existing Conditions - Static Loading

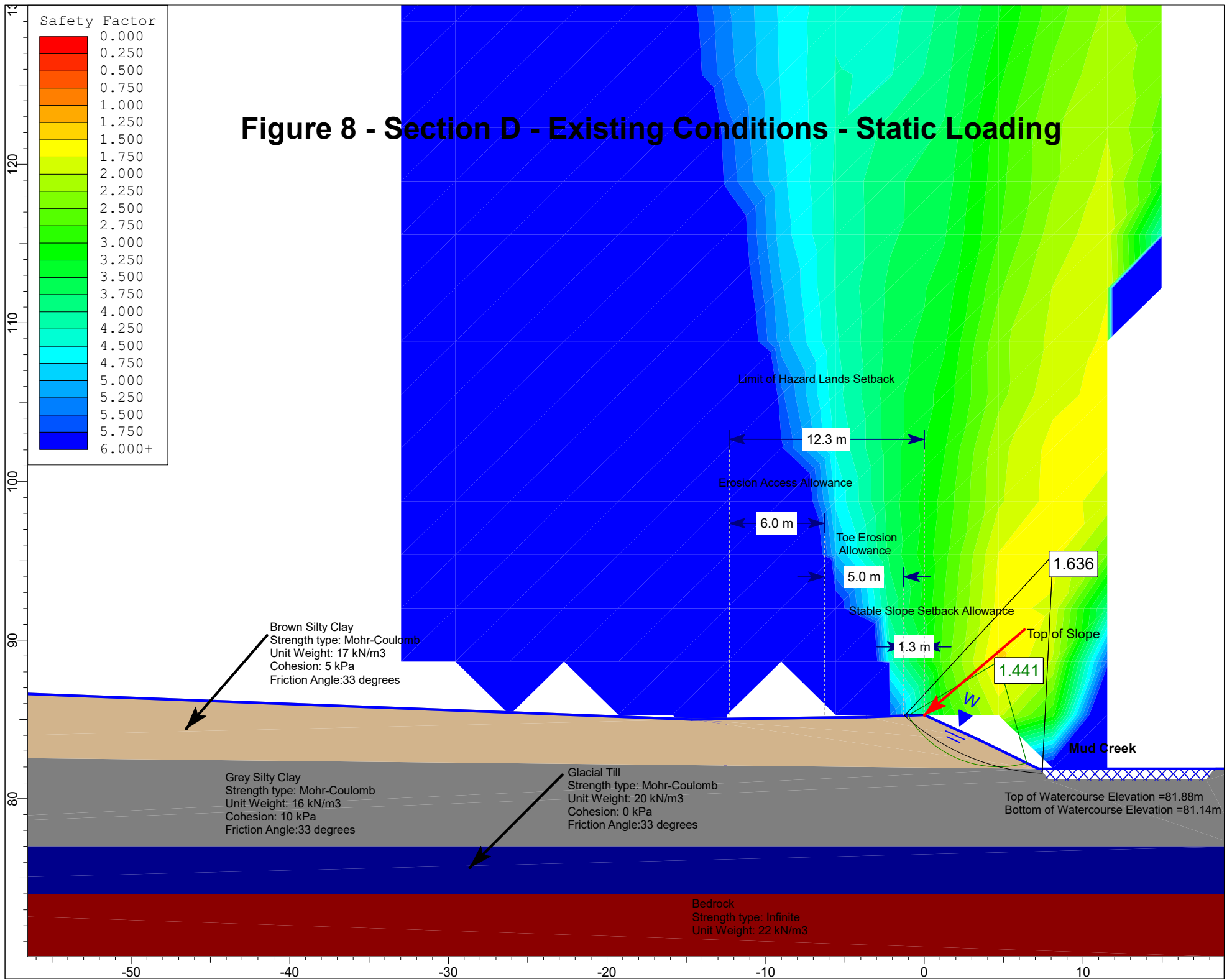


Figure 9 - Section D - Existing Conditions - Seismic Loading

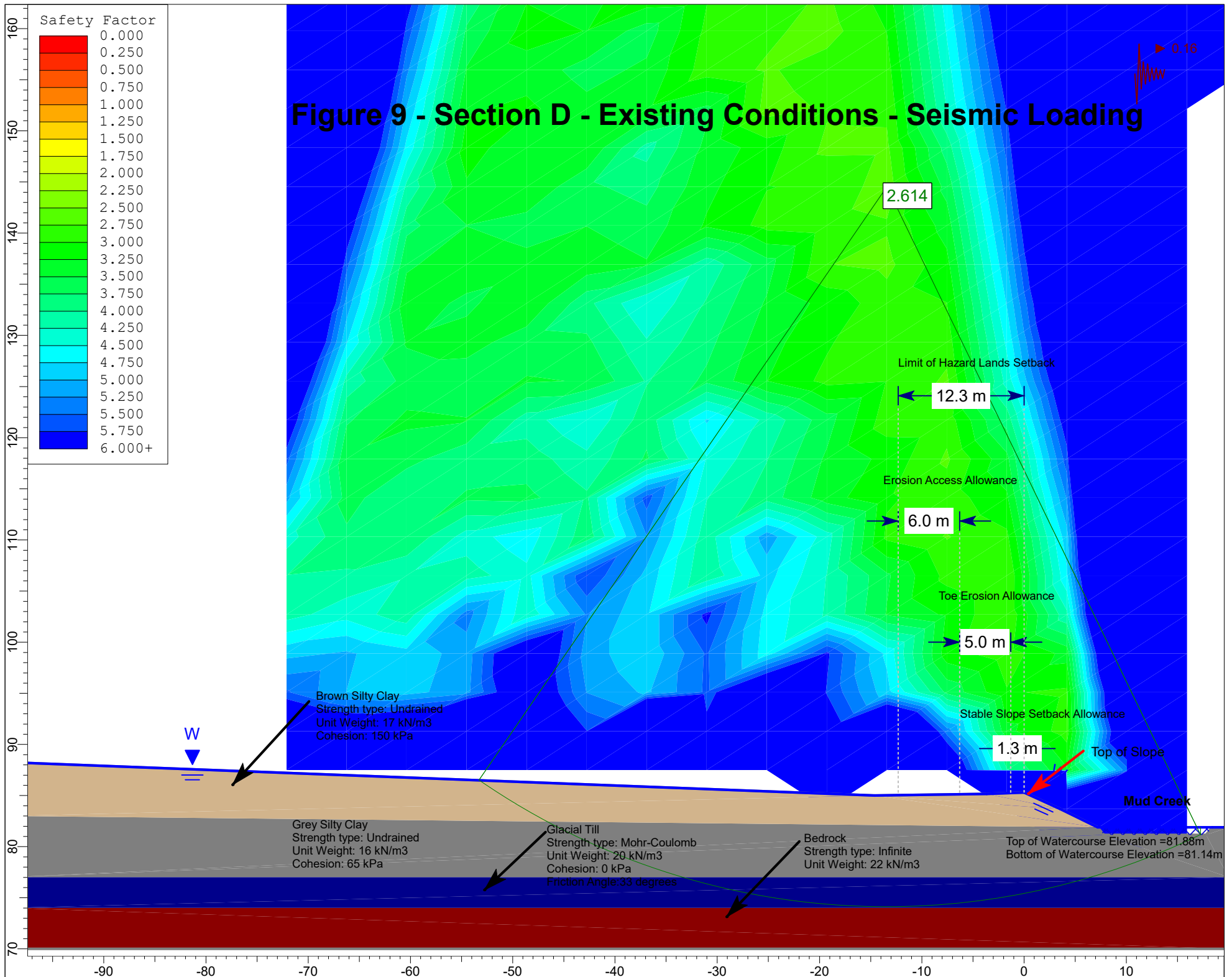


Figure 10 - Section A - Proposed Conditions - Static Loading

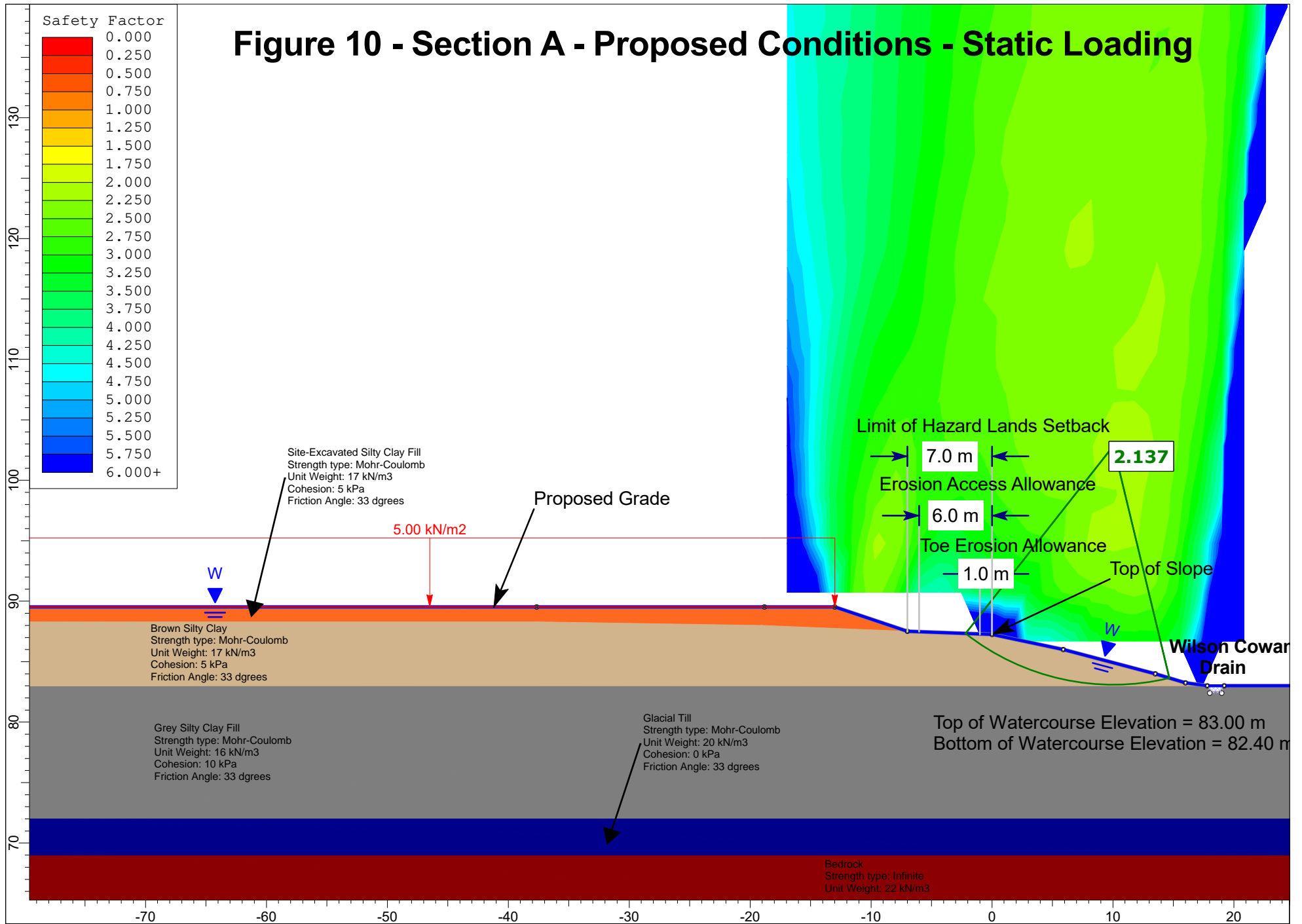


Figure 11 - Section A - Proposed Conditions - Seismic Loading

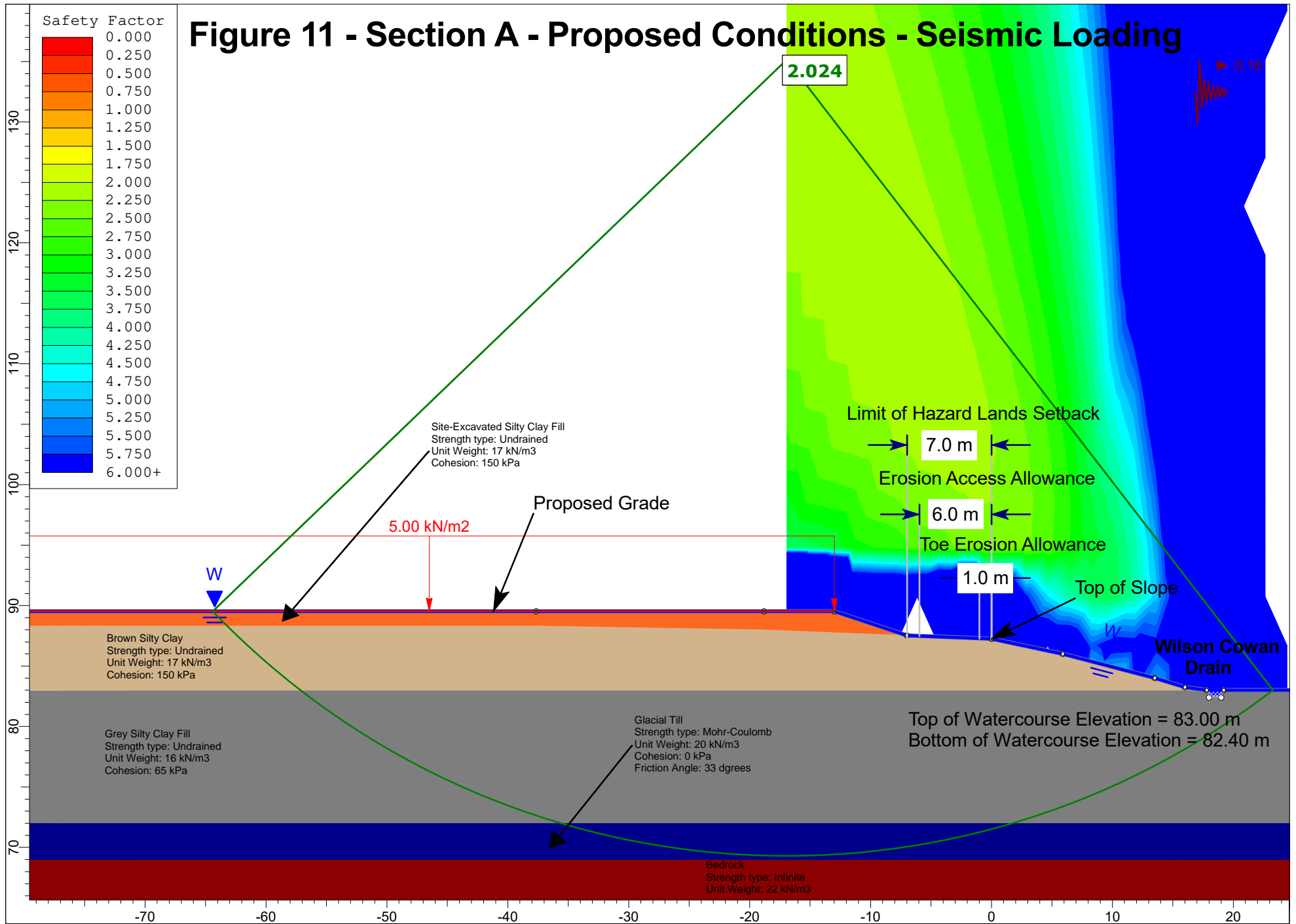


Figure 12 - Section B - Proposed Conditions - Static Loading

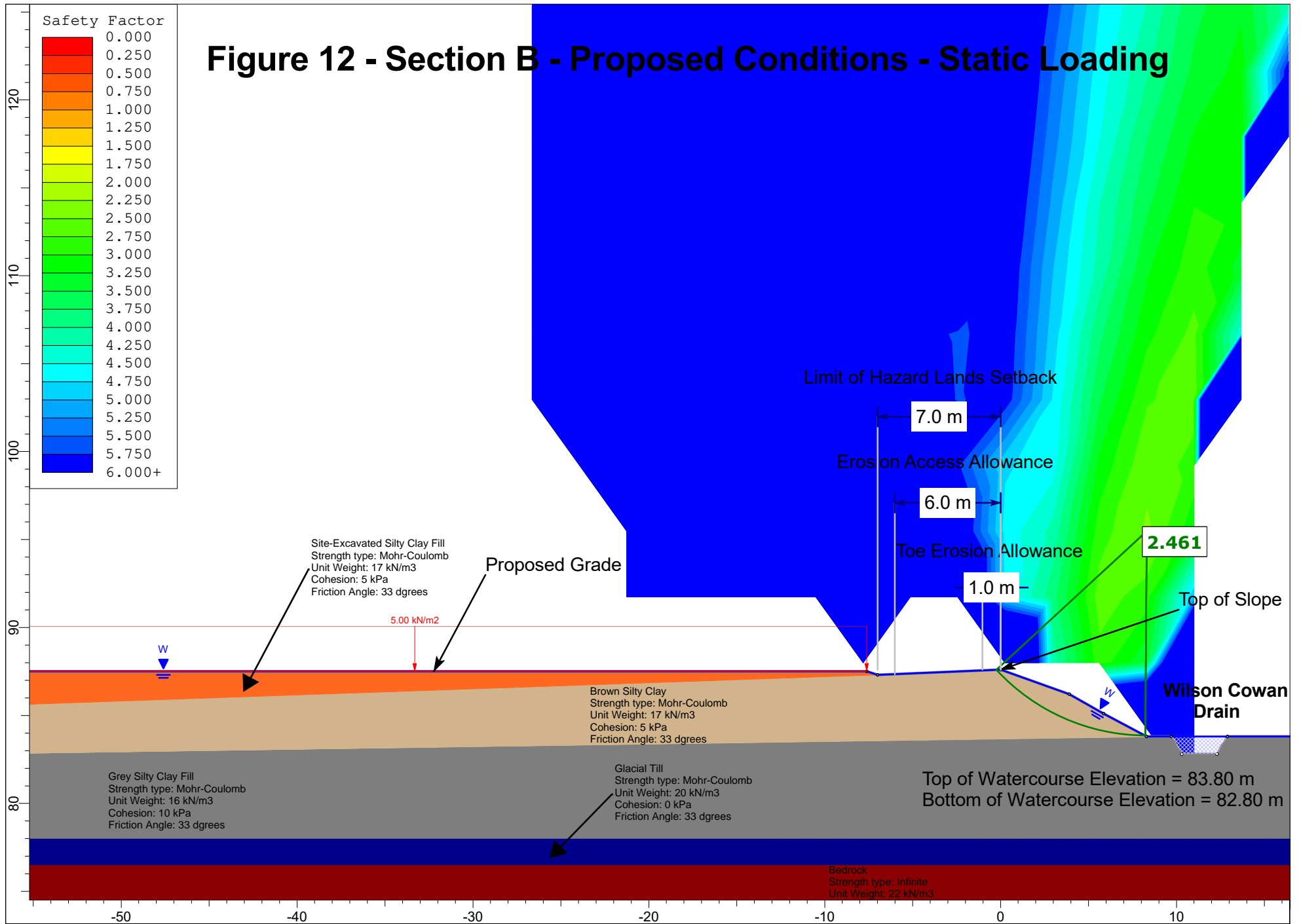


Figure 13 - Section B - Proposed Conditions - Seismic Loading

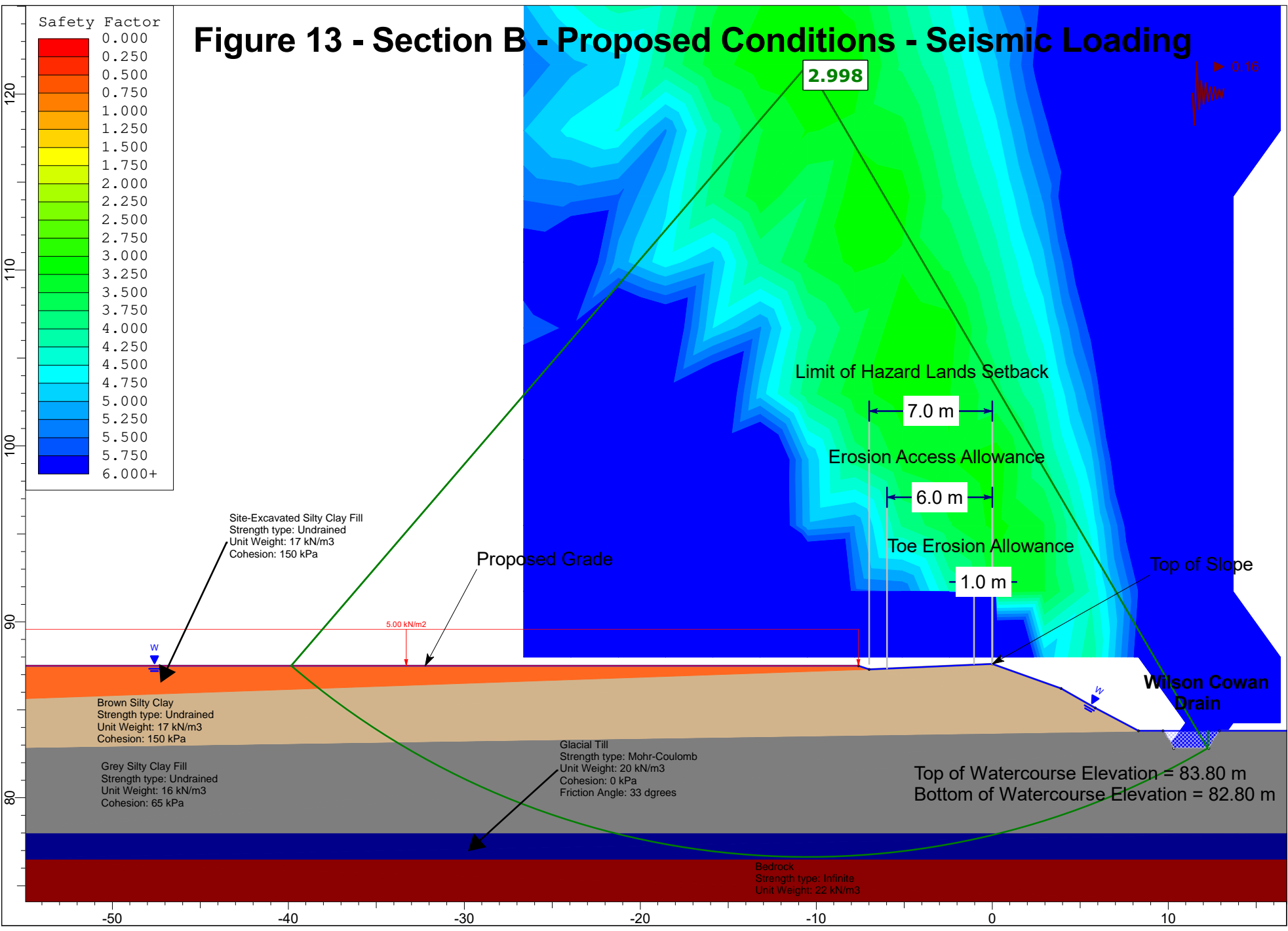


Figure 14 - Section C - Proposed Conditions - Static Loading

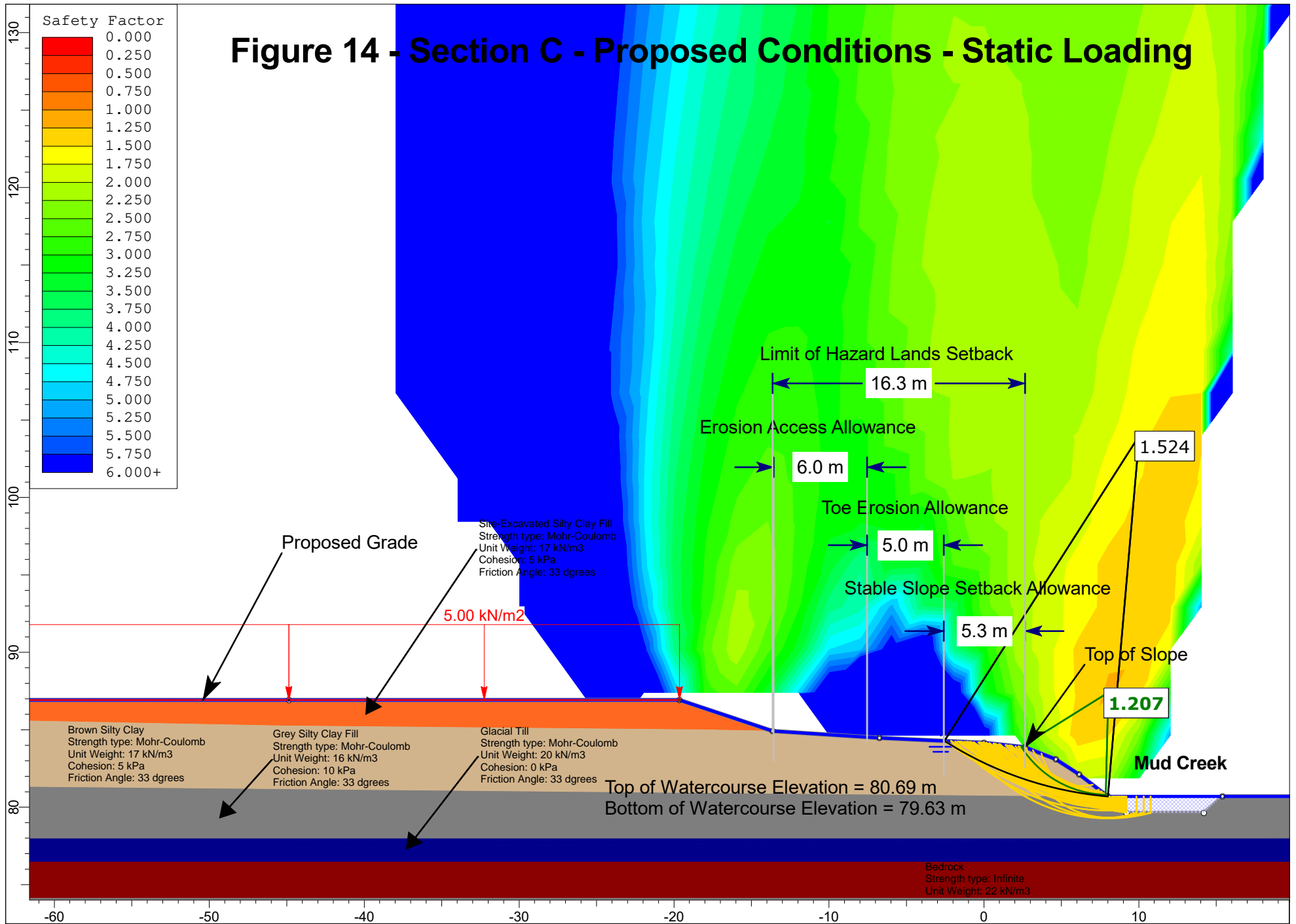


Figure 15 - Section C - Proposed Conditions - Seismic Loading

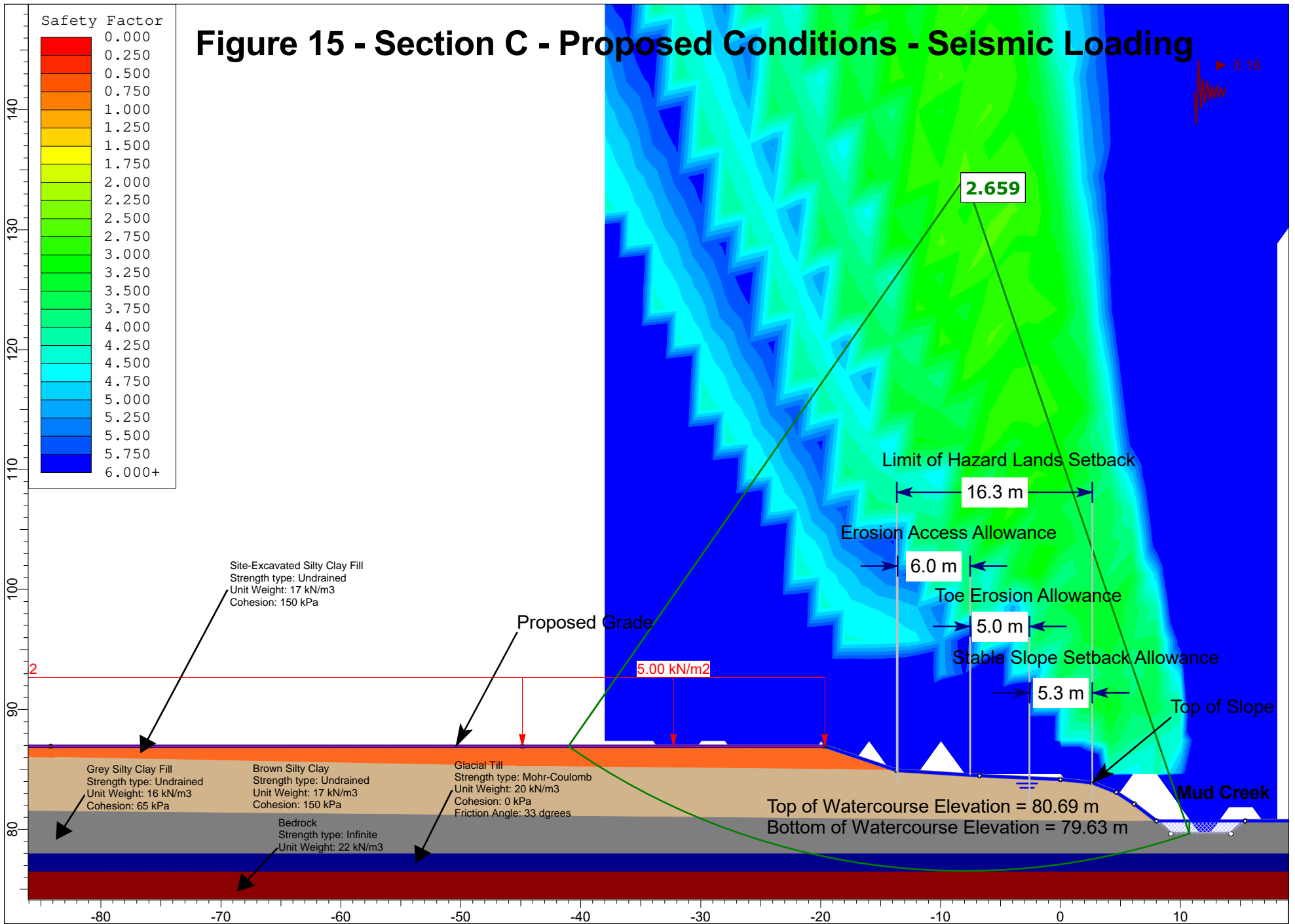


Figure 16 - Section D - Proposed Conditions - Static Loading

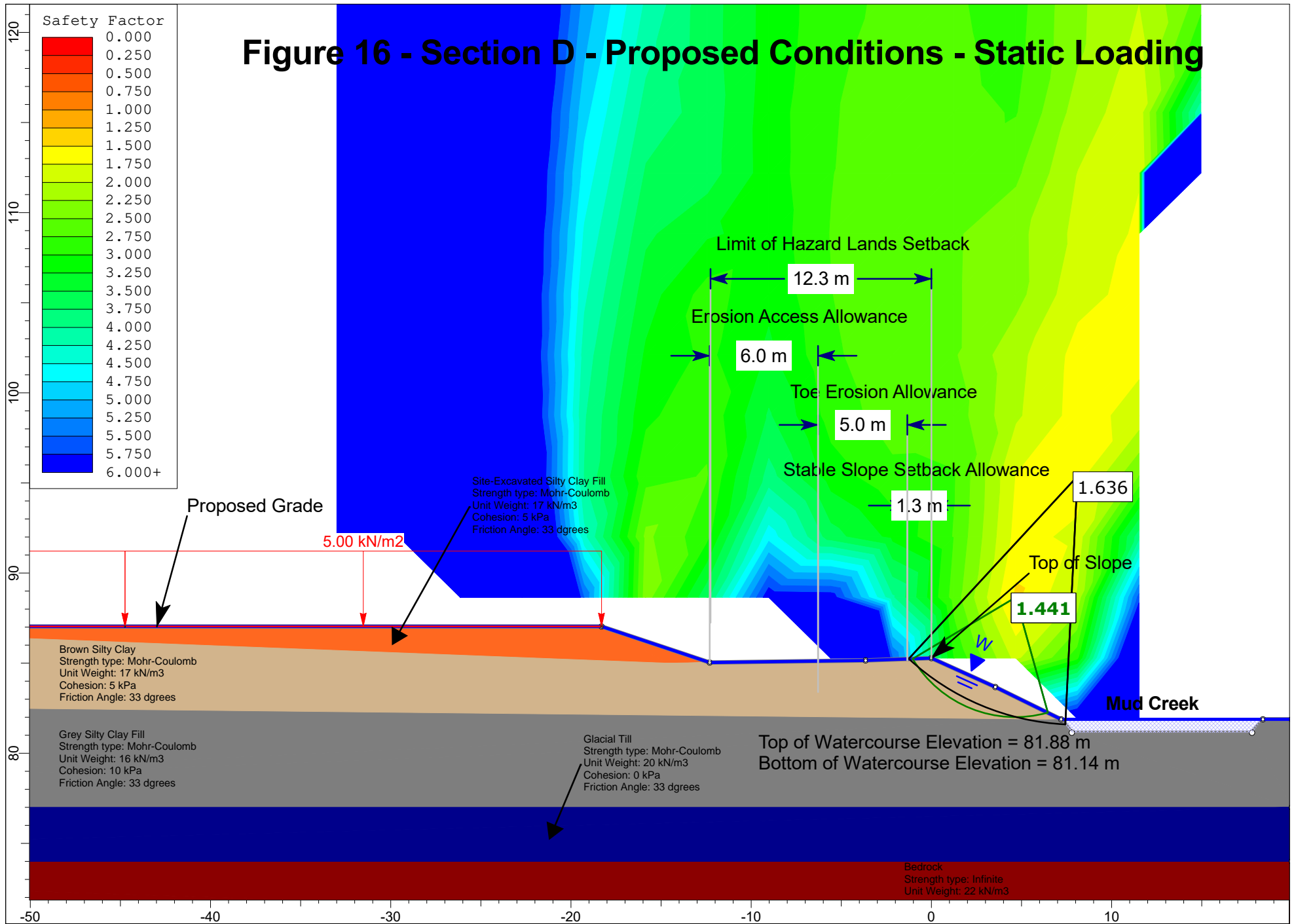


Figure 17 - Section D - Proposed Conditions - Seismic Loading

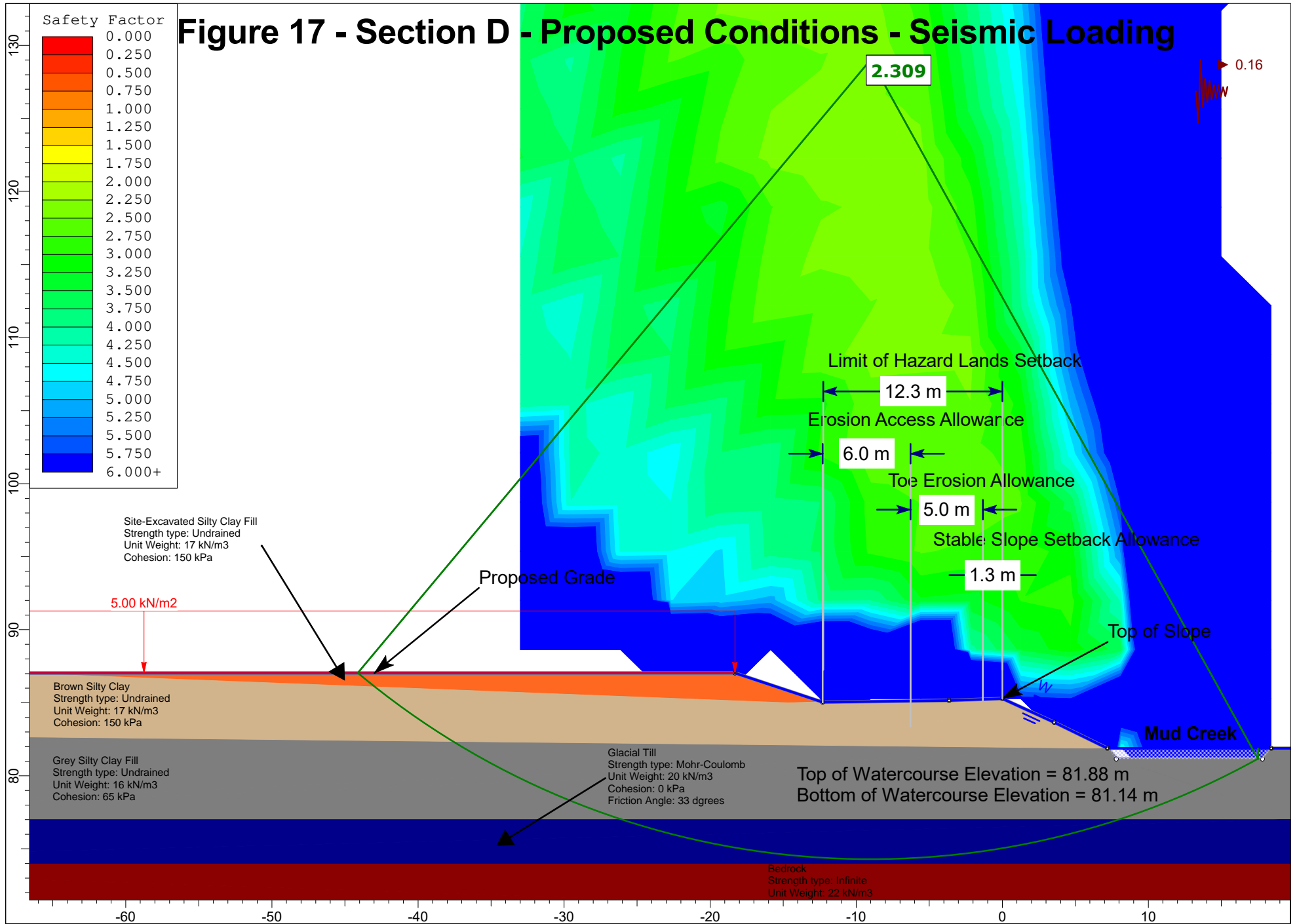


Photo 1: Area located at the bottom of the slope along the south-west portion of the subject site. Area is well vegetated and sloped gradually towards the valley floor.



Photo 2: Area along creek tributary 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 creek tributary appeared to be flowing very slowly and/or ponding.



Photo 3: Area along creek tributary 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 creek tributary 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 creek tributary appeared to be flowing very slowly and/or ponding.



Photo 5: Area along creek tributary 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 creek tributary appeared to be flowing very slowly and/or ponding.



Photo 6: Area along creek tributary 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 creek tributary and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along the tributary 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 creek tributary and west portion of the subject site. Area appeared to be well vegetated with a slightly steeper bank along the tributary 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 creek tributary and north-west portion of the subject site. Area appeared to be well vegetated with a gentle flow throughout the tributary at the time of site visit. Gradual slope observed from subject site to the valley floor.



Photo 10: Area of intersection of tributary along west portion of subject site and Ruisseau Mud Creek. Area of Ruisseau 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 Ruisseau 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 oversteepening of banks also observed. Creek appeared to be flowing very slowly at the time of site visit.



Photo 12: Area of Ruisseau 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 Ruisseau 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 Ruisseau 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.

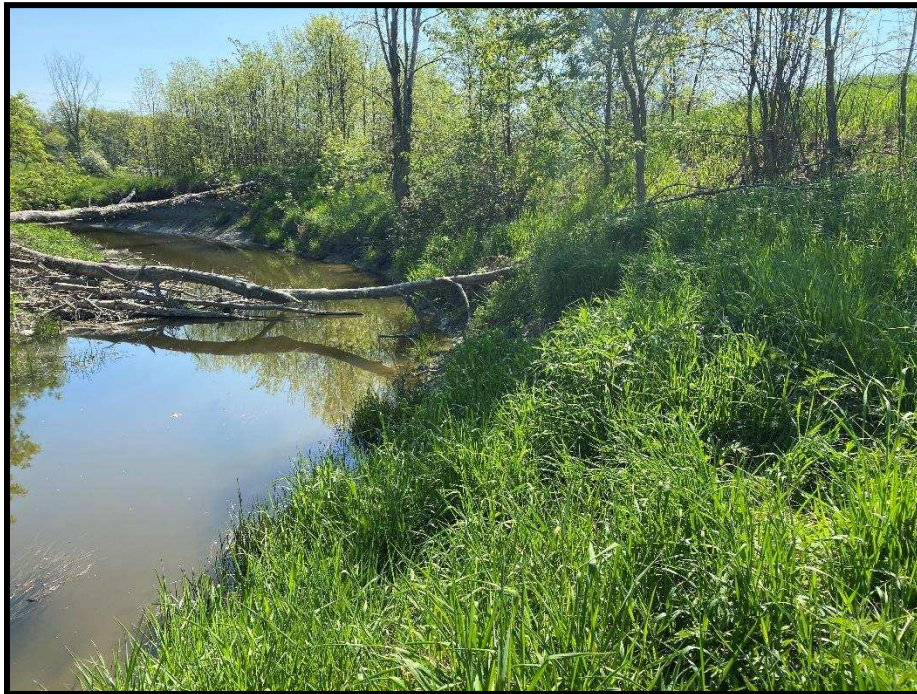


Photo 15: Area of Ruisseau Mud Creek along northern portion of subject site. Photographed area appeared to have banks exposed to streams flow along with slumping and undercutting of banks at the time of our site visit.



Photo 16: Area of Ruisseau 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 Ruisseau 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.

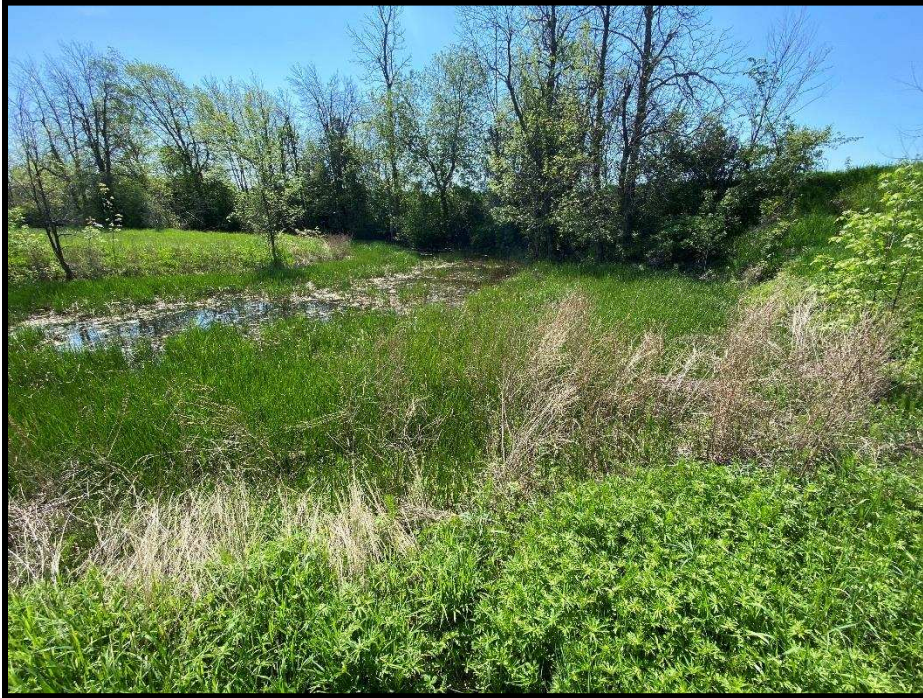
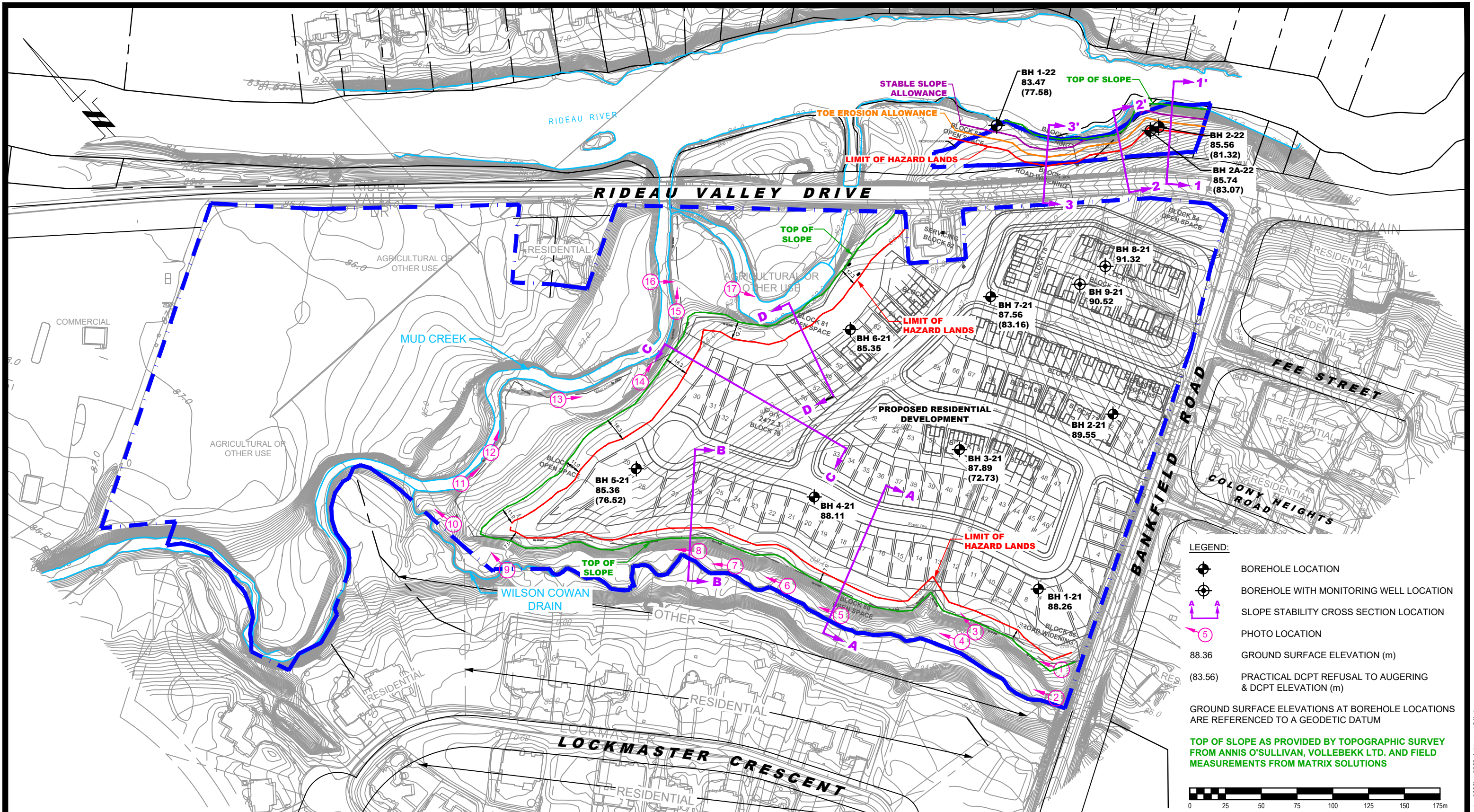


Photo 17b: Close-up of Photo 17 - Area of Ruisseau Mud Creek valley floor along north-east portion of subject site. Photographed area appeared to contain well rooted vegetation and mature trees. No erosion observed along toe of slope at time of site visit.





LEGEND:

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- SLOPE STABILITY CROSS SECTION LOCATION
- 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



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

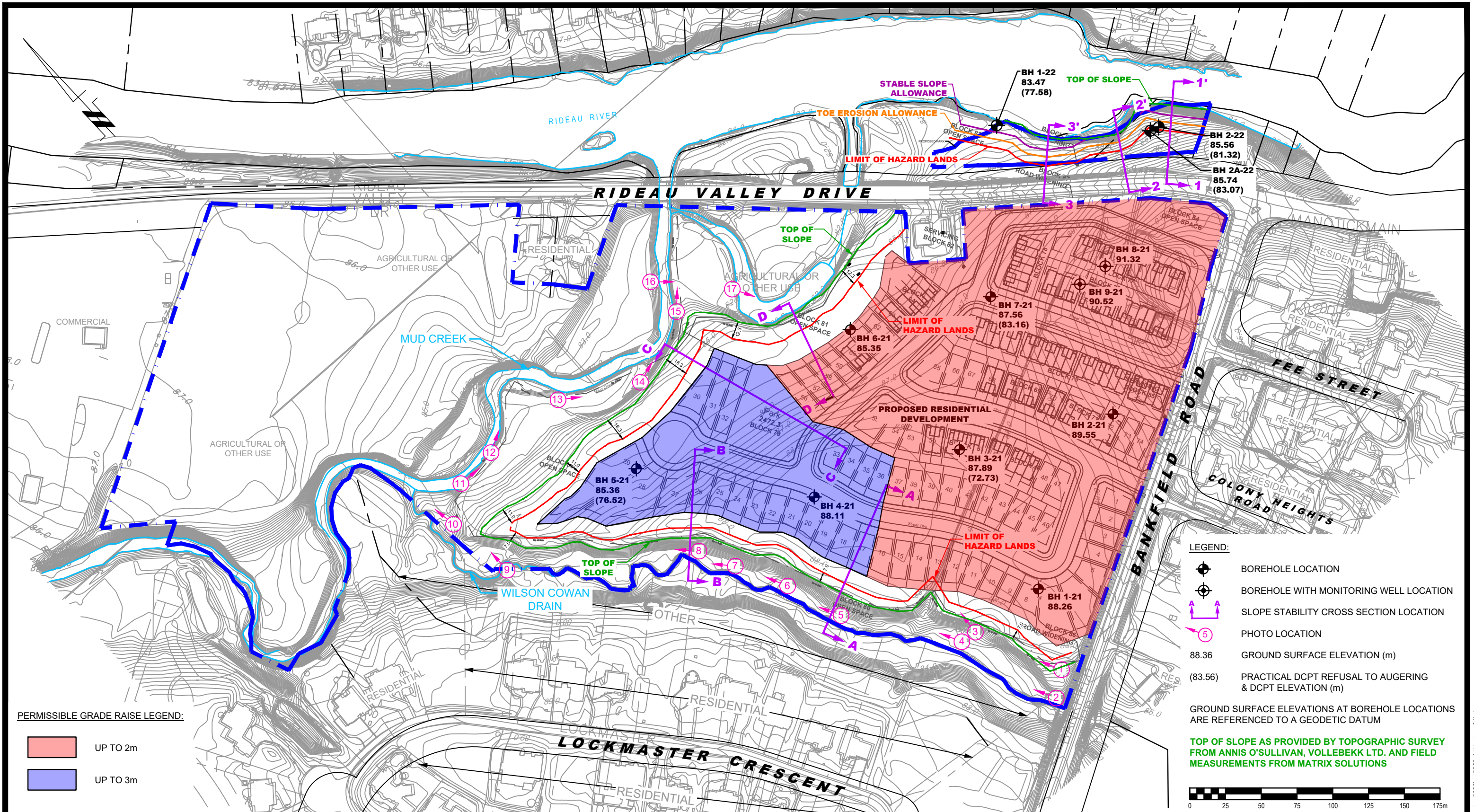
UNIFORM DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4386 RIDEAU VALLEY DRIVE

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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NO.	REVISIONS	DATE	INITIAL

APPENDIX 3

RELEVANT REPORTS



PATERSON GROUP

Consulting Engineers

9 Auriga Drive
Ottawa, Ontario
K2E 7T9

Tel: (613) 226-7381

Geotechnical Engineering
Environmental Engineering
Hydrogeology
Materials Testing
Building Science
Rural Development Design
Retaining Wall Design
Noise and Vibration Studies

patersongroup.ca

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/m³)	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/m³)	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.

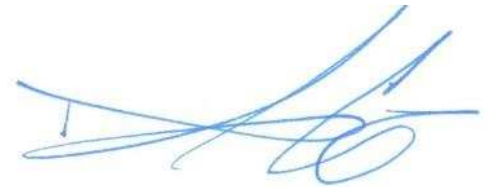
We trust this report meets your present requirements.

Best Regards,

Paterson Group Inc.



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)

DATUM Geodetic

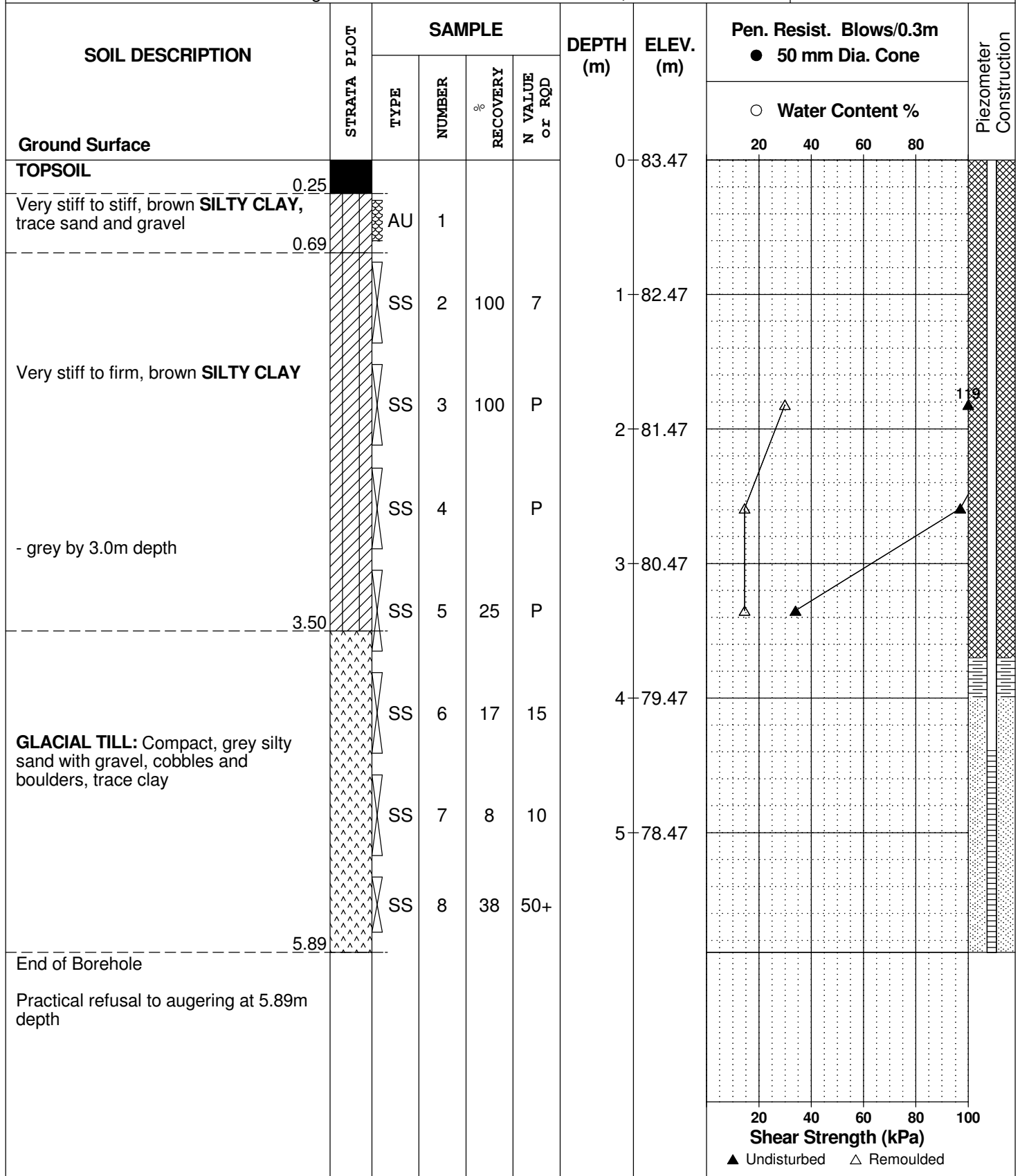
REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

FILE NO.
PG5828

HOLE NO.
BH 1-22



DATUM Geodetic

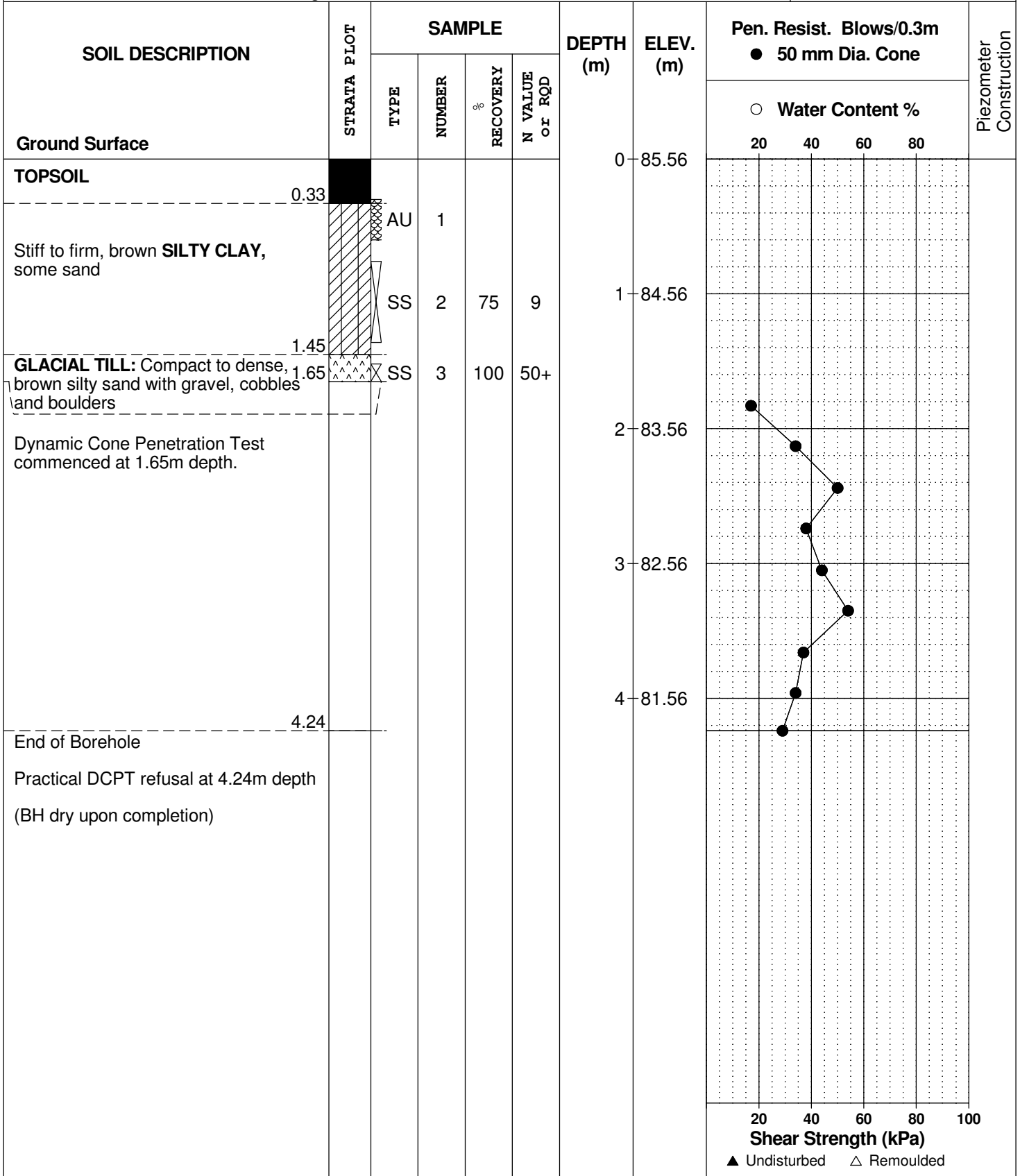
REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 16, 2022

FILE NO.
PG5828

HOLE NO.
BH 2-22



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)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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Figure 1 - Section 1-1 - Existing Conditions - Static Loading

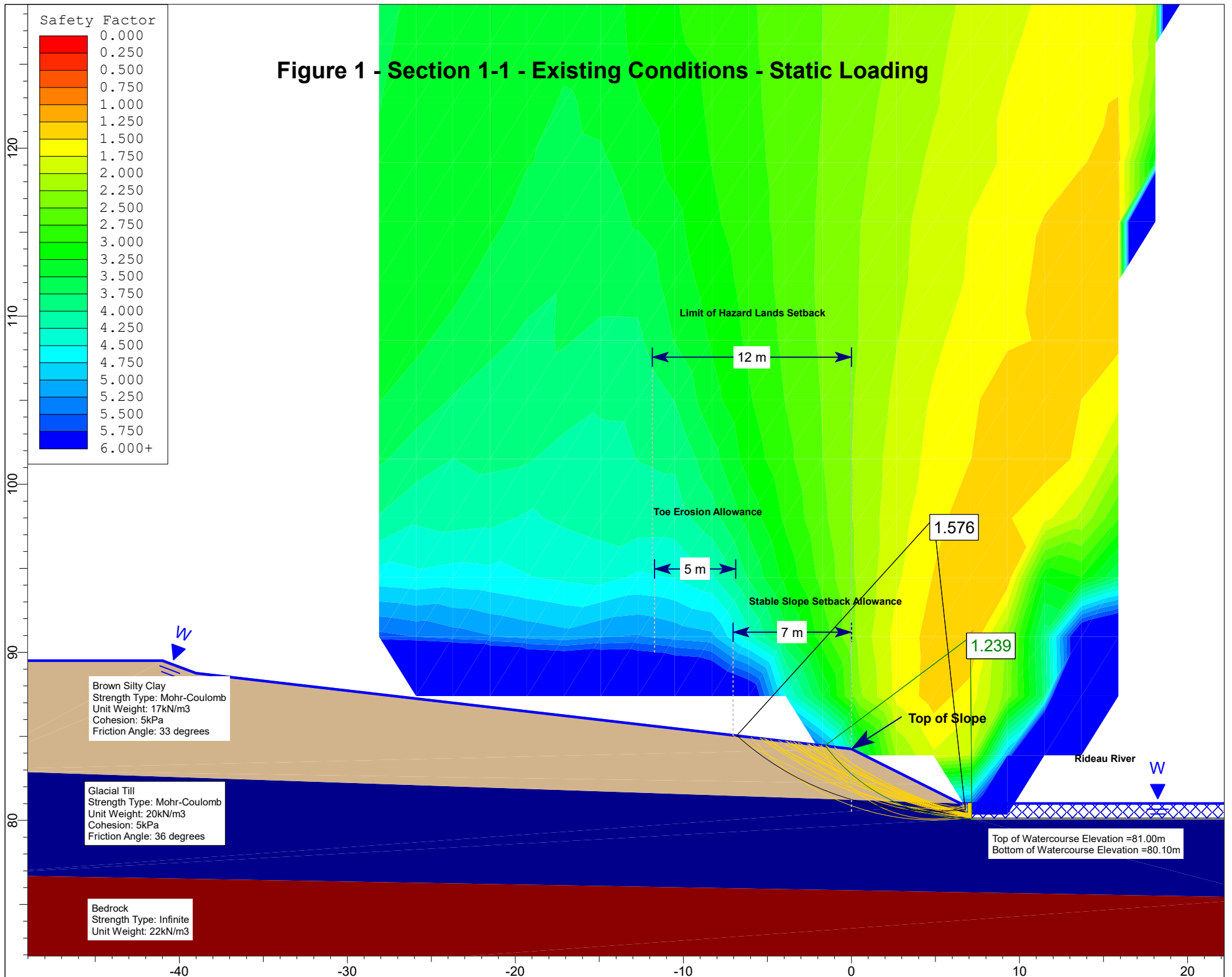


Figure 2 - Section 1-1 - Existing Conditions - Seismic Loading

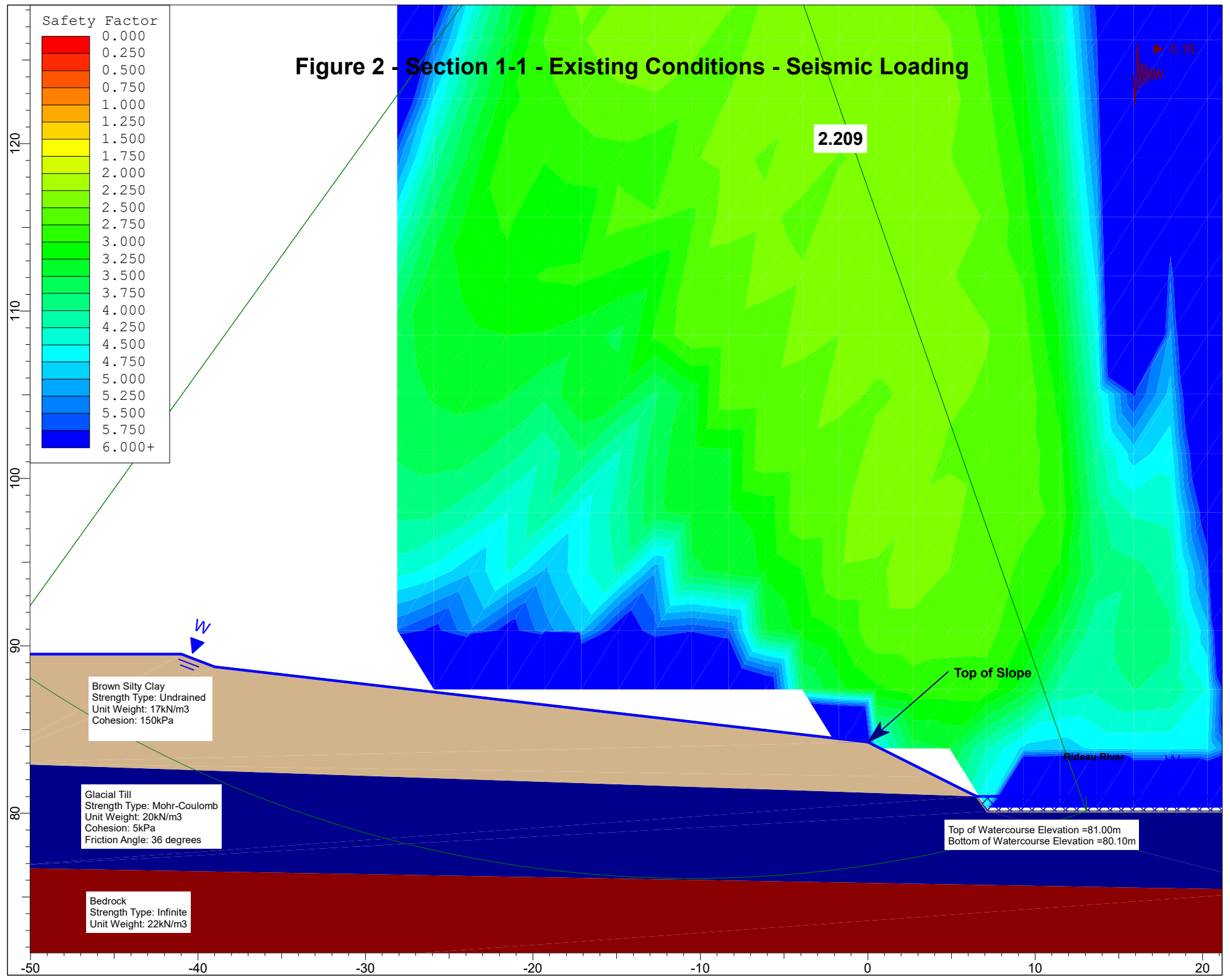


Figure 3 - Section 2-2 - Existing Conditions - Static Loading

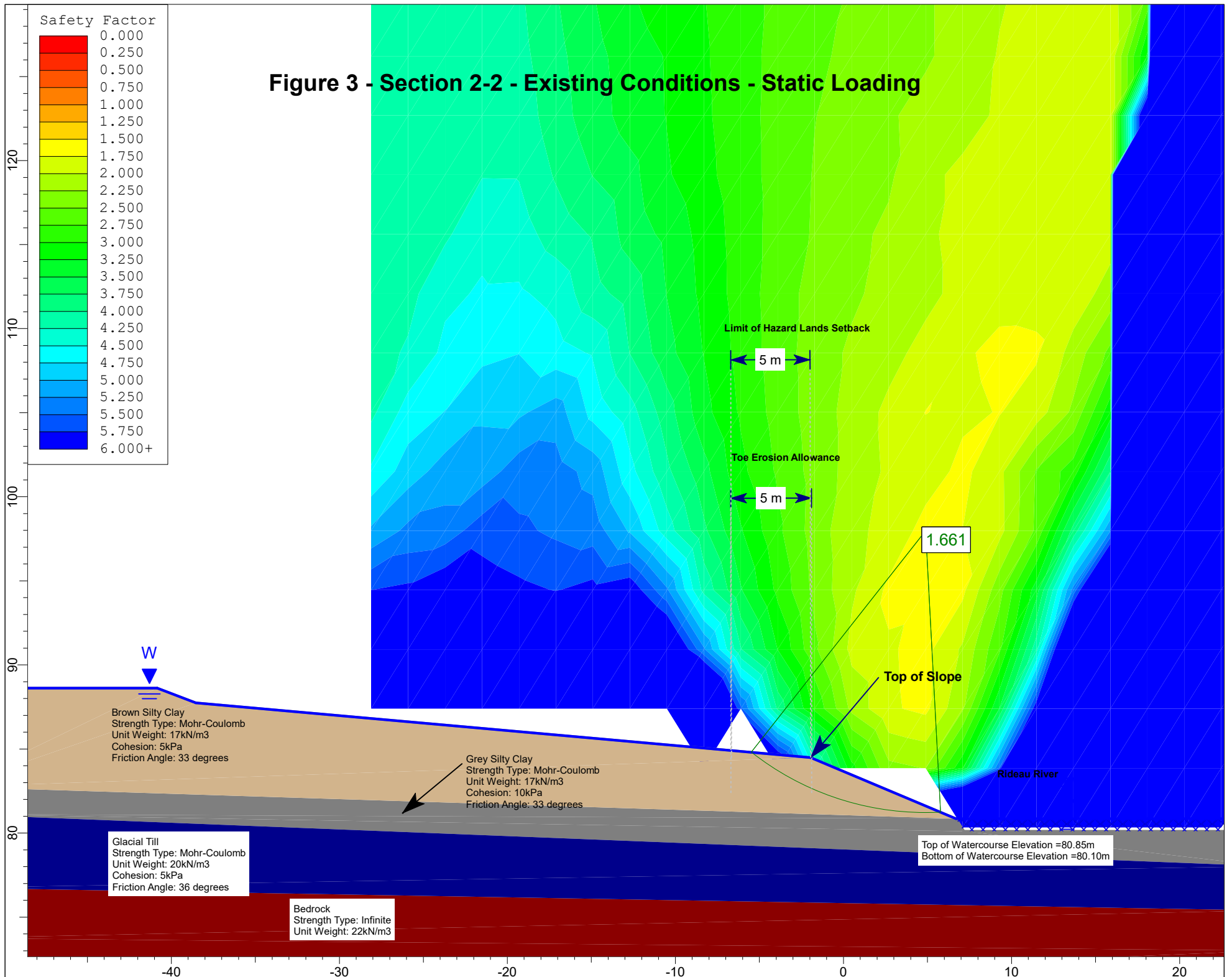


Figure 4 - Section 2-2 - Existing Conditions - Seismic Loading

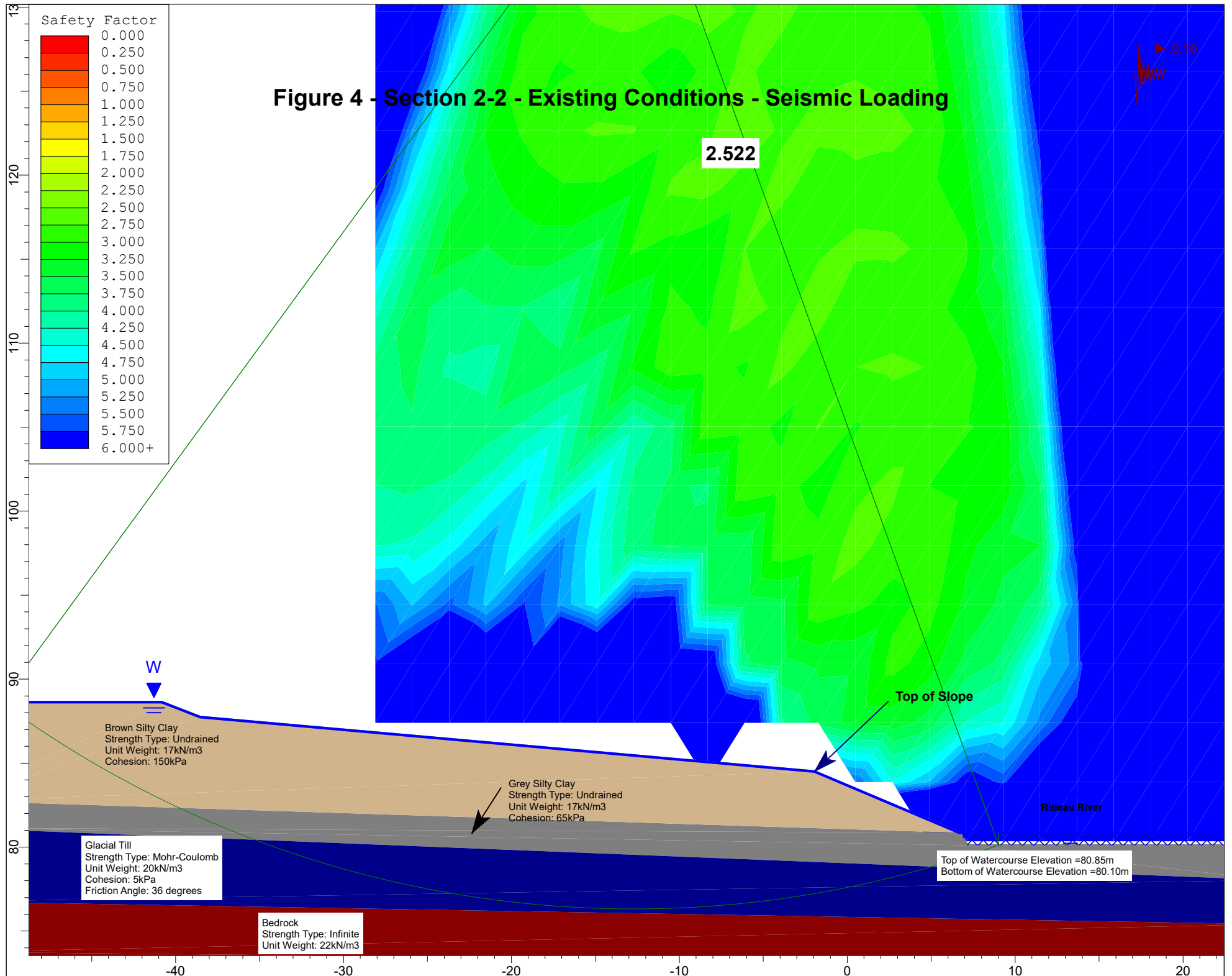


Figure 5 - Section 3-3 - Existing Conditions - Static Loading

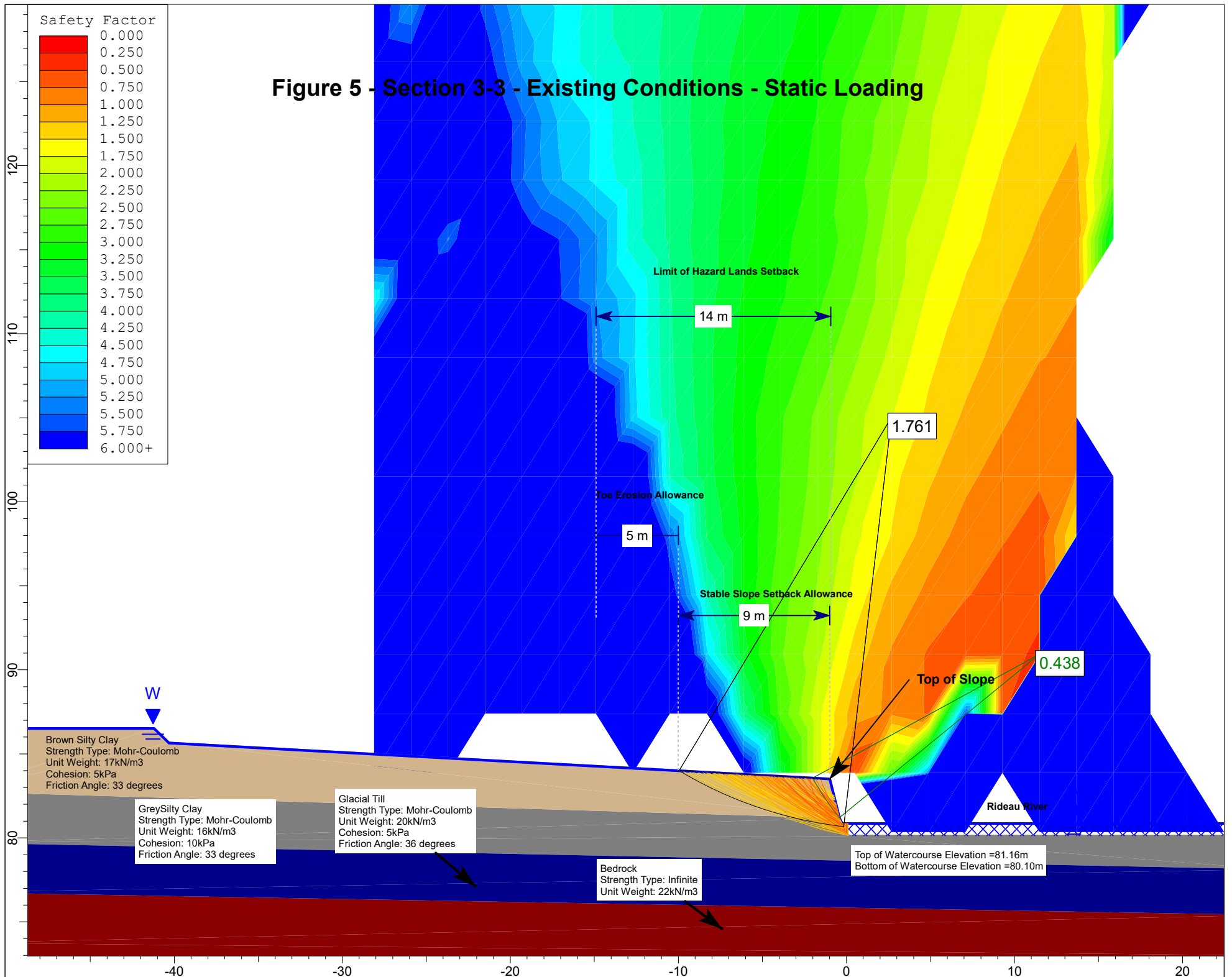
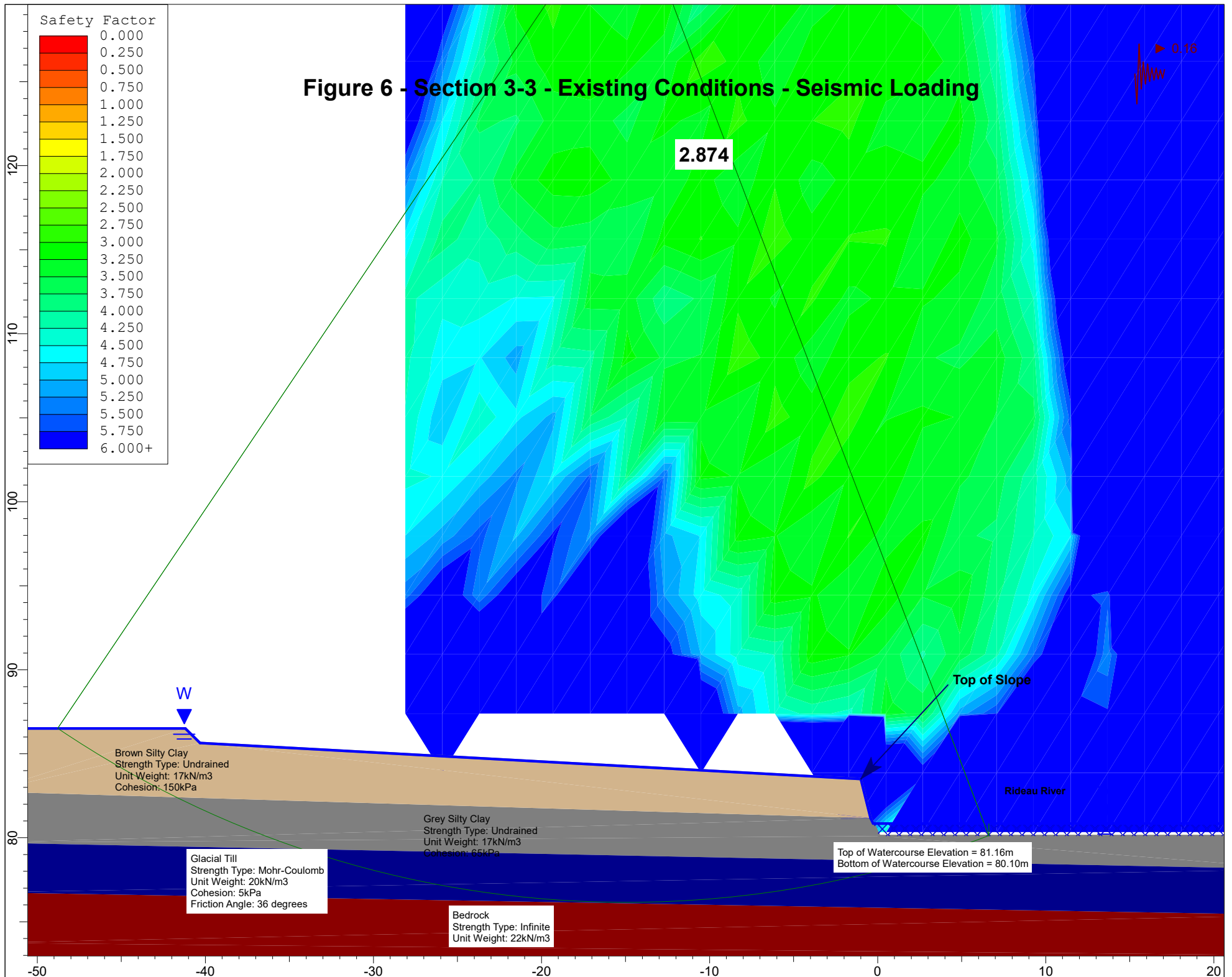
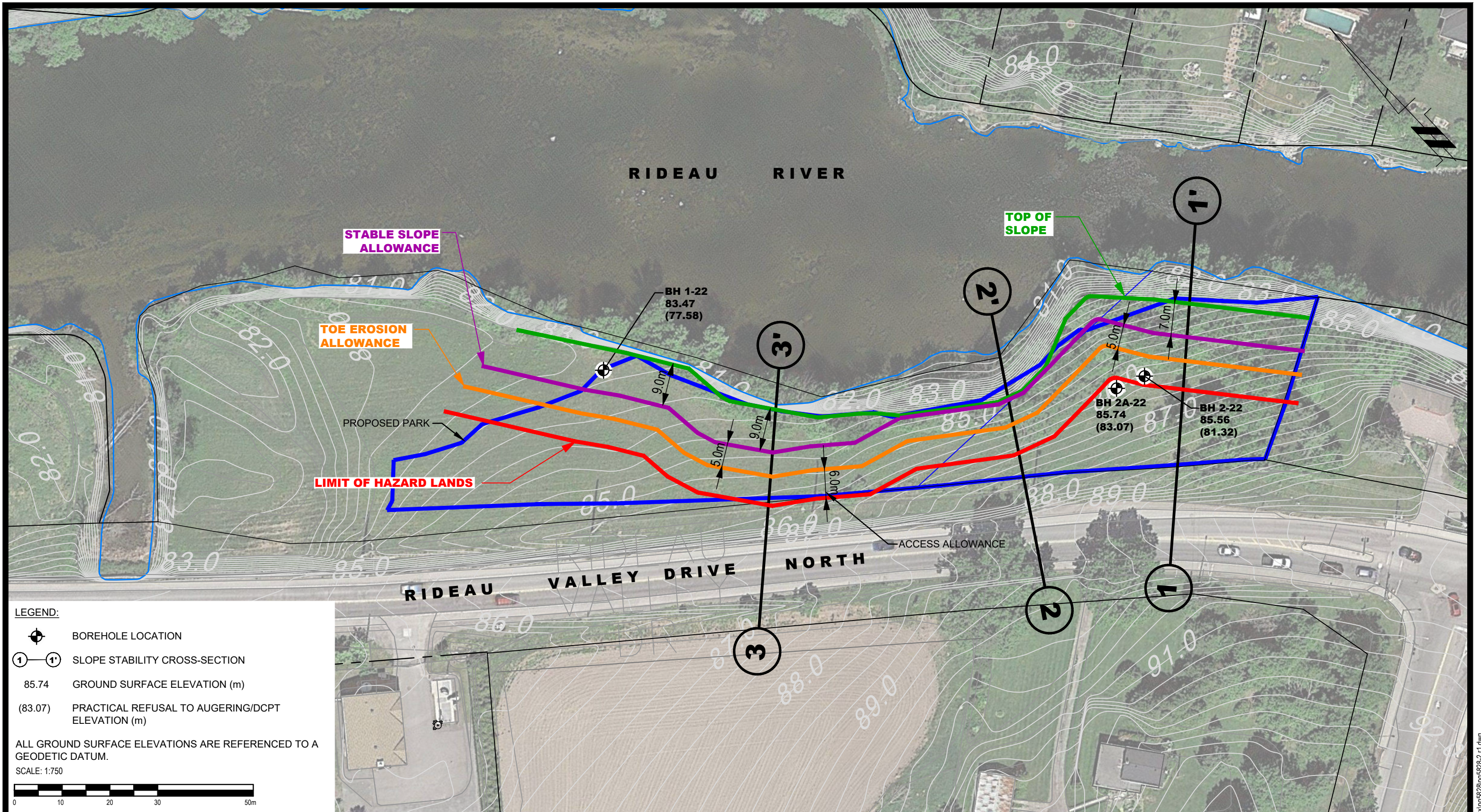


Figure 6 - Section 3-3 - Existing Conditions - Seismic Loading





LEGEND:

- BOREHOLE LOCATION
- 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

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

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.:	

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 for 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 et 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.

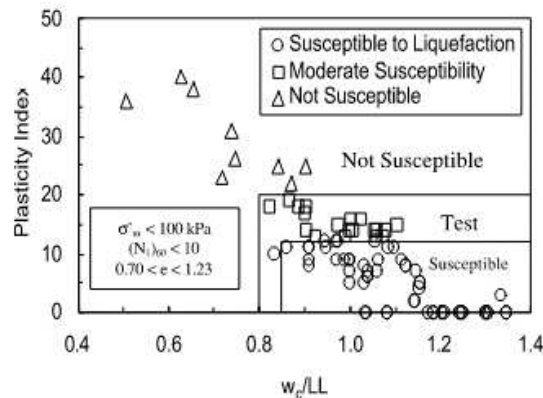


Figure 1. Criteria for evaluating liquefaction susceptibility of fine-grained soils (Bray et al. 2004).

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 Cowan 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 proposed 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.



David J. Gilbert, P.Eng.





**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 proposed 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 proposed 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 rationale 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 rationale, 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.

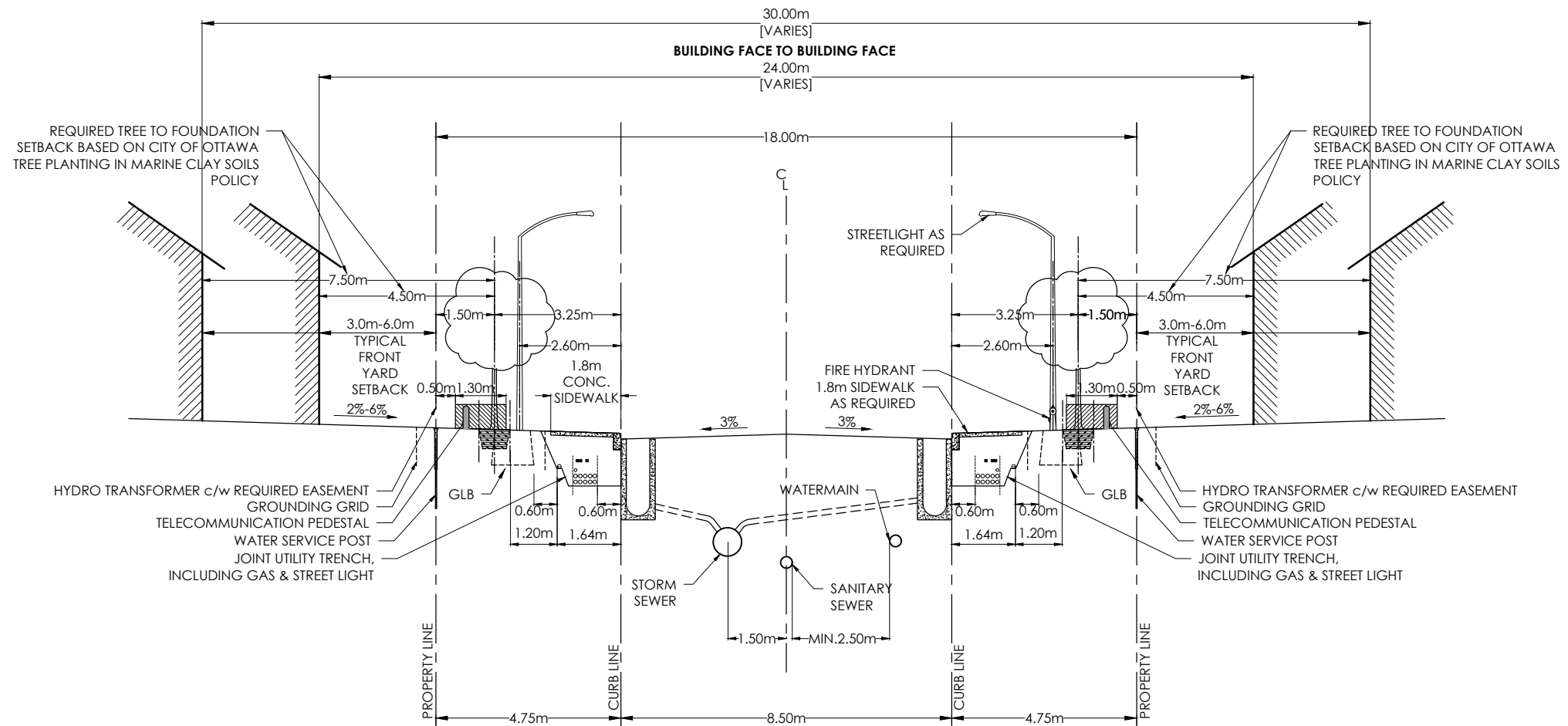



David J. Gilbert, P.Eng.

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. 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.



18.0m ROW CROSS SECTION

REV. DATE: AUG. 2022

DWG. No. ROW-18.0