Appendix A WATER SUPPLY SERVICING

A.1 DOMESTIC WATER DEMAND ESTIMATE

3856, 3866, and 3876 Navan Road - Domestic Water Demand Estimates

- Based on Site Plan by Temprano & Young Architects Inc.

Building ID	Area (m^2)	Population	Daily Rate of Demand ($L/m^2/day$)	Avg Day Demand [2]		Max Day Demand [3]		Peak Hour Demand [3]	
				(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Church & Service Building	2,431		28,000	4.7	0.08	7.1	0.12	12.8	0.21
Total Site :				**4.7**	**0.08**	**7.1**	**0.12**	**12.8**	**0.21**

Water demand criteria used to estimate peak demand rates for institutional areas are as follows:

1 maximum day demand rate = 1.5 x average day demand rate

2 peak hour demand rate = 1.8 x maximum day demand rate

A.2 FIRE FLOW REQUIREMENTS PER FUS

Fire Flow Calculations as per Ontario Building Code (Appendix A)

Job#	1604-01122	Designed by:	TKR
Date	14-Feb-19	Checked by:	KLS

$$Q = KVS_{tot}$$

Q = Volume of water required (L)
V = Total building volume (m3)
K = Water supply coefficient from Table 1
S_{tot} = Sotal of spatial coefficeint values from property line exposures on all sides as obtained from the formula

$$S_{tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + S_{side4}]$$

1	Type of construction	Building Classification		Water Supply Coefficient
	Non-Combustible without Fire-Resistance Ratings	A-2, B-1, B-2, B-3, C, D		16
2	Area of one floor (m^2)	number of floors	height of ceiling (m)	Total Building Volume (m^3)
	1000	1	11	11,000
3	Side	Exposure Distance (m)	Spatial Coefficient	Total Spatial Coeffecient
	North	52	0	1.3
	East	7	0.3	
	South	0	0	
	West	52	0	
4	Established Fire Safety Plan?	Reduction in Volume (%)		Total Volume Reduction
	yes	30%		30%
5				**Total Volume 'Q' (L)**
				160,160
				Minimum Required Fire Flow (L/min)
				4,500

*NOTE: South spatial coefficient reduced to 0 as the south exterior building wall maintains a fire rating of 2.0h or more and has no unprotected openings.

Assumed established fire safety plan for church
Avg height of church 11.0m

Fire Flow Calculations as per Ontario Building Code 2006 (Appendix A)

Job#	1604-01122	Designed by:	TKR
Date	14-Feb-19	Checked by:	KLS

$$Q = KVS_{tot}$$

Q = Volume of water required (L)
V = Total building volume (m3)
K = Water supply coefficient from Table 1
S_{tot} = Sotal of spatial coefficeint values from property line exposures on all sides as obtained from the formula

S_{tot} =1.0 + [S_{side1} + S_{side2} + S_{side3} + S_{side4}]

1	Type of construction	Building Classification		Water Supply Coefficient
	Non-Combustible without Fire-Resistance Ratings	A-2, B-1, B-2, B-3, C, D		16
2	Area of one floor (m^2)	number of floors	height of ceiling (m)	Total Building Volume (m^3)
	920	1	6	5,520
3	Side	Exposure Distance (m)	Spatial Coefficient	Total Spatial Coeffiecient
	North	0	0	1.25
	East	28	0	
	South	7.5	0.25	
	West	20	0	
4	Established Fire Safety Plan?	Reduction in Volume (%)		Total Volume Reduction
	no	0%		0%
5				**Total Volume 'Q' (L)**
				110,400
				Minimum Required Fire Flow (L/min)
				3,600

*NOTE: North spatial coefficient reduced to 0 as the north exterior building wall maintains a fire rating of 2.0h or more and has no unprotected openings.

Based on fire separation as provided on the site plan
Average height of 6.0m assumed.

A.3 BOUNDARY CONDITIONS

BOUNDARY CONDITIONS

Boundary Conditions For: 3856, 3866 and 3876 Navan Rd

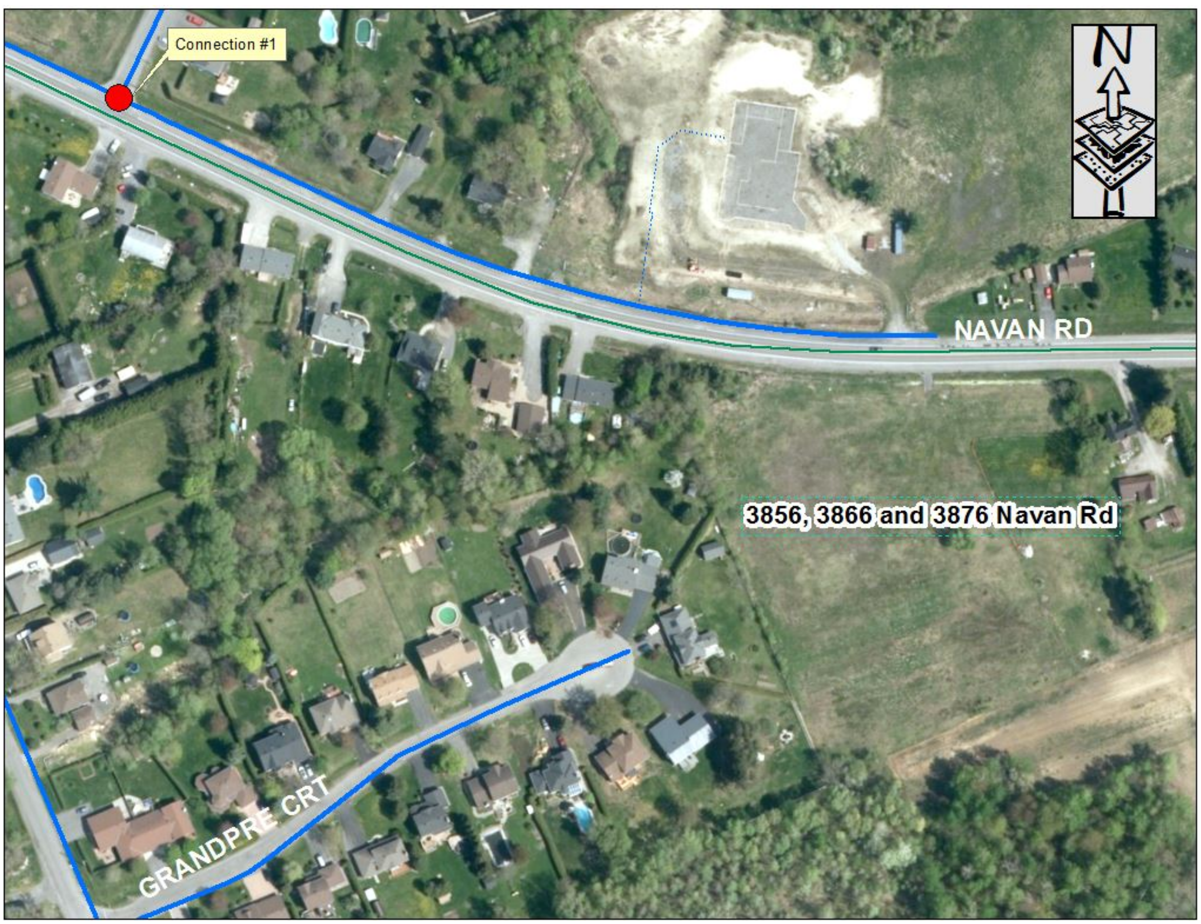
Date of Boundary Conditions: 2018-Aug-23

Provided Information:

Scenario	Demand	
	L/min	L/s
Average Daily Demand	15.0	0.3
Maximum Daily Demand	22.8	0.4
Peak Hour	40.8	0.7
Fire Flow #1 Demand	6,000	100.0

Number of Connections: 1

Location:



BOUNDARY CONDITIONS

Results:

Connection #: 1

Demand Scenario	Head (m)	Pressure[1] (psi)
Maximum HGL	130.7	63.2
Peak Hour	126.8	57.6
Max Day Plus Fire (6,000) L/min	125.3	55.5

[1]Elevation: **86.260 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.

A.4 HYDRAULIC MODEL ANALYSIS

Hydraulic Model Results - Average Day Analysis

Junction Results

ID	Demand	Elevation	Head	Pressure	
	(L/s)	(m)	(m)	(psi)	(Kpa)
2	0.00	86.70	133.50	66.53	458.71
3	0.08	86.20	133.50	67.24	463.61

Pipe Results

ID	From Node	To Node	Length	Diameter	Roughness	Flow	Velocity
			(m)	(mm)		(L/s)	(m/s)
3	7001	2	325	155	100	0.08	0
4	2	3	60	204	110	0.08	0

Hydraulic Model Results -Peak Hour Analysis

Junction Results

ID	Demand	Elevation	Head	Pressure	
	(L/s)	(m)	(m)	(psi)	(Kpa)
2	0.00	86.70	128.50	59.42	409.69
3	0.21	86.20	128.50	60.13	414.58

Pipe Results

ID	From Node	To Node	Length	Diameter	Roughness	Flow	Velocity
			(m)	(mm)		(L/s)	(m/s)
3	7001	2	325	155	100	0.21	0.01
4	2	3	60	204	110	0.21	0.01

Hydraulic Model Results -Fire Flow Analysis

ID	Static Demand	Static Pressure		Static Head	Fire-Flow Demand	Residual Pressure		Available Flow at Hydrant	Available Flow Pressure	
	(L/s)	(psi)	(Kpa)	(m)	(L/s)	(psi)	(Kpa)	(L/s)	(psi)	(Kpa)
2	0.00	54.87	378.32	125.30	75	-16.00	-110.32	51.10	20	137.90
3	0.12	55.58	383.21	125.30	75	-18.17	-125.28	50.68	20	137.90

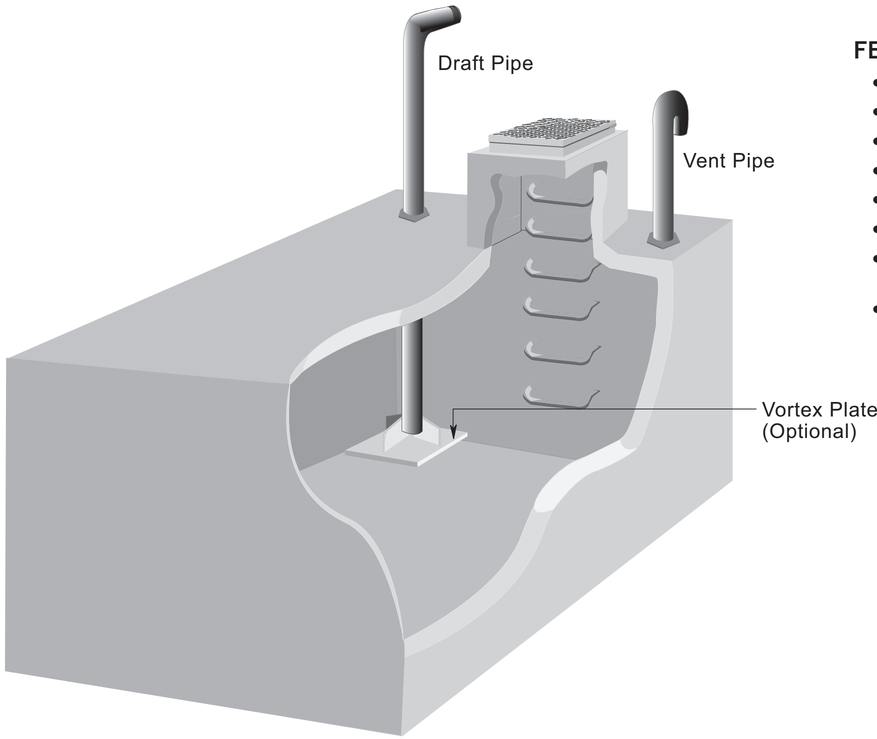
A.5 WATER STORAGE TANK

TYPICAL PRECAST FIRE WATER RESERVOIR

WILKINSON HEAVY PRECAST LIMITED
DUNDAS, ONTARIO 905-628-5611
www.wilkinsonheavyprecast.com

CONSTRUCTION DETAILS

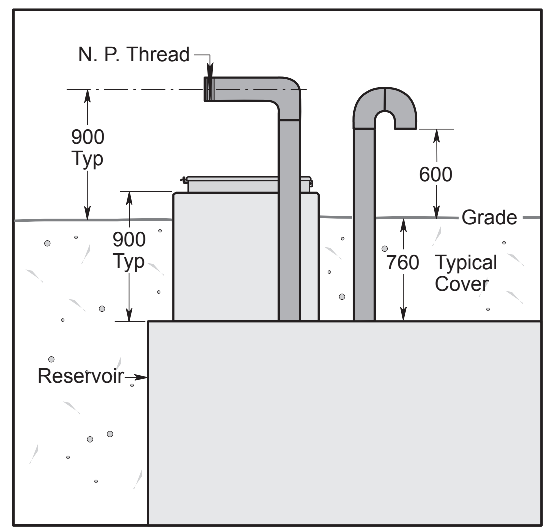
Concrete: 35 MPa at 28 Days, 5 to 8% Air Entrainment.



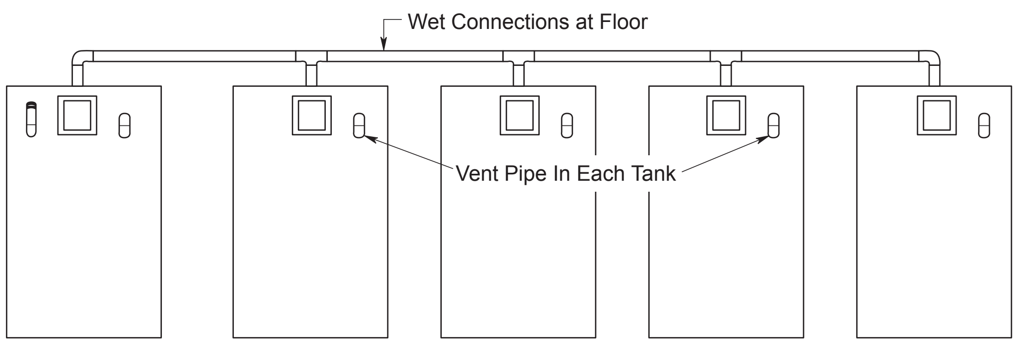
FEATURES

- Draft pipe and vent are steel pipe sized to suit, hot dip galvanized after fabrication.
- Vent pipe incorporates a stainless steel insect screen.
- Precast concrete access riser with frame and hinged, drip proof and lockable cover.
- Aluminum ladder rungs to the floor.
- Draft pipe draws from 75 mm above tank floor.
- Typical tank capacities from 25,000 to 114,000 Litres.
- For more capacity special fittings can be provided to connect any number of tanks in series.
- Contact the factory for optional cross connection and ventilation configurations.

TYPICAL APPURTENANCE DETAILS



TYPICAL SERIES CONNECTION



Dimensions in mm
N.T.S.

WARNING ! IMPROPER INSTALLATION ESPECIALLY IN UNSTABLE SOILS CAN RESULT IN THE STRUCTURAL FAILURE OF THIS PRODUCT

9 May, 2008

50,000 LITRE PRECAST WASTEWATER HOLDING TANK MODEL H50.1S

WILKINSON HEAVY PRECAST LIMITED
DUNDAS, ONTARIO 905-628-5611
www.wilkinsonheavyprecast.com

CONSTRUCTION DETAILS *

Concrete: 35 MPa at 28 Days, 5 to 8% Air Entrainment.

Reinforcing: 15 M bars at 250 mm centres each way in walls, roof and floor.
Eight extra 15 M bars around each roof access opening.
Minimum cover over reinforcing steel - 25 mm.

Weight:

Top Section	22,000 kg
Bottom Section	22,000 kg
Total	44,000 kg

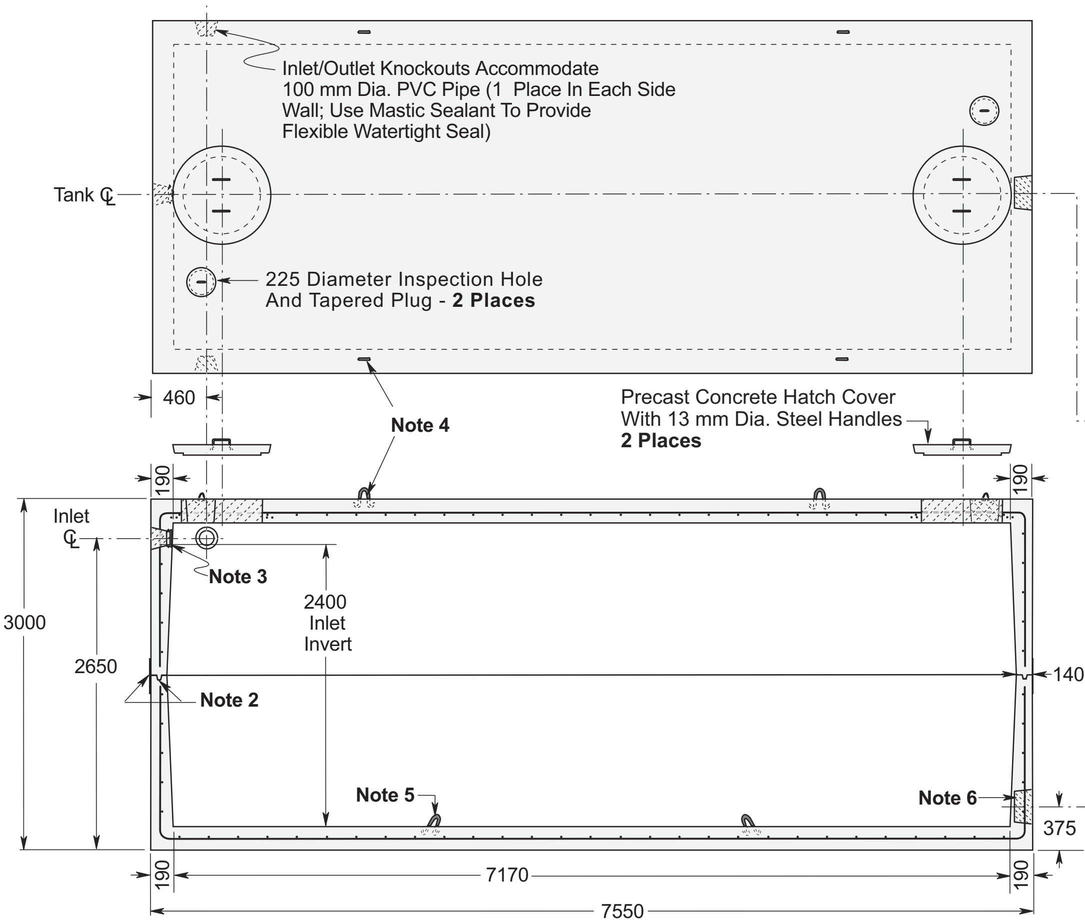
Actual Capacity: 19,277 Litres Per Vertical Metre.
50,120 Litres to Underside of Roof.
46,265 Litres to Invert of Inlet.

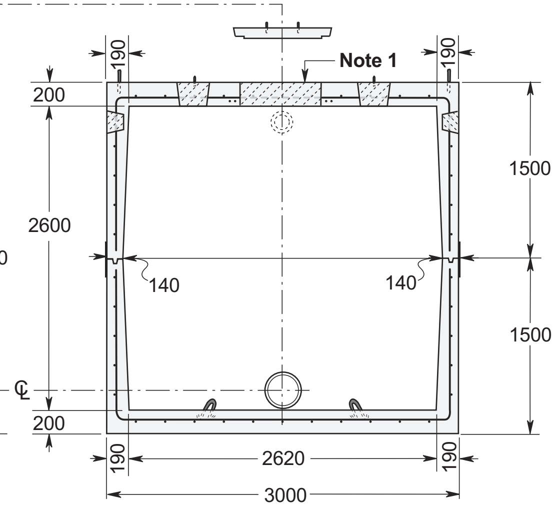
NOTES

1. Large 685 mm diameter roof access openings facilitate tank maintenance. Unless otherwise specified/ordered this tank will be shipped with 840 mm diameter concrete hatch covers. Please note that each cover weighs approximately 125 kg and must be handled only with suitable mechanical lifting equipment. Please see Access Riser section for available options.
2. Close tolerance of tongue and groove joint and fibrous mastic sealant ensures a solid structural and watertight seal. Primer and Mastic Band are supplied with each tank for application to the external surface of the tank over the joint between sections. This band is to be applied by the installing contractor.
3. Flexible watertight inlet pipe connector to accommodate 150 mm diameter PVC pipe.
4. Top section lifting points four places.
5. Bottom section lifting points four places.
6. Knockout suitable to accommodate a flexible, watertight pipe connector for 150 or 200 mm diameter PVC pipe.

This Tank Is Available With:

- Aluminum ladder rungs to the floor. Consult with the factory as to how this will effect the size and location of the access opening.
- Mechanical connection of the tank sections to enhance water tightness and resistance to frost heave.





Dimensions in mm
N.T.S.

* Commensurate with a **1.2 Metre** burial over the top slab in firm soil away from any area of vehicular traffic.

For recommended installation procedures refer to Wilkinson Installation Guidelines and Lifting and Assembly Instructions.

WARNING ! IMPROPER INSTALLATION ESPECIALLY IN UNSTABLE SOILS CAN RESULT IN THE STRUCTURAL FAILURE OF THIS PRODUCT

14 April 2011

Appendix B WASTEWATER SERVICING

B.1 SANITARY DEMAND

Geotechnical Engineering

Environmental Engineering

Archaeological Studies

Hydrogeology

Geological Engineering

Materials Testing

Building Science

patersongroup

Private Wastewater Servicing Brief

Proposed St. Joseph's Coptic Orthodox Church
3856, 3866, 3876 Navan Road
Ottawa, Ontario

Prepared For

Stantec Consulting Ltd.

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

November 9, 2016

Report PH3209-REP.01

TABLE OF CONTENTS

PAGE

1.0 INTRODUCTION

1.1 Terms of Reference

Paterson Group Inc. (Paterson) was retained by Stantec Consulting Ltd. to carry out a preliminary site assessment to determine the adequacy of the subject property to support a private onsite sewage system. The objective of this assessment is to demonstrate that the proposed development can be serviced by an onsite sewage system which is designed in conformance with the pertinent regulations. These works are being carry out in support of a rezoning application.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and recommendations pertaining to the onsite sewage system for the subject development as understood at the time of writing this report.

1.2 Background

The proposed development will be located on three (3) parcels of land having civic addresses of 3856, 3866 and 3876 Navan Road, Ottawa. The legal description of these parcel is Part 1, Part 2, and Part 3 of Plan 4R-26690 located on Part of Lot 7, Concession 11 in the former Township of Cumberland, now in the City of Ottawa. It is being proposed to merge these properties to create a single property, hereafter referred to as the subject property.

It is being proposed to construct a church on the subject site along with the associated parking areas and onsite services. The site is located in an area of the city where municipal water is available however, no municipal sanitary or storm services are available. As such, it is being proposed that the development be serviced by a private onsite sewage system.

2.0 SITE DESCRIPTION

2.1 Surface Conditions

The subject property is approximately 1.42 ha. in size. The property is vacant of any structures and is mostly grass covered. The property fronts onto Navan Road and the existing grades along the front property line of the site are located approximately 1 m below the road grade. The site is relatively flat sloping gently downwards from the east property line to the west.

2.2 Subsurface Conditions

A subsurface investigation was carried out on the subject lands by LRL Associates Ltd. (LRL) in support of the severance application to create the three land parcels. These works were carried out in May, 2012, the results of which are recorded in LRL File Ref. No. 120289, dated August 29, 2012.

Based on the findings of the LRL investigation, the subsoil profile underlying the site consists of topsoil followed by sand. Clay was encountered below the sand stratum at approximately 2.6 m to 2.9 m depth, at some of the test locations. The results of the sieve analysis carried out on samples of the sand deposit indicate that the sand consists of a poorly graded fine sand (SP). The total overburden thickness is estimated to be in excess of 24 m.

Groundwater was encountered in the sand stratum at approximately 0.8 m to 1.1 m below the ground surface at the time of the LRL field investigation.

3.0 SEWAGE SYSTEM DESIGN

3.1 General Development Constraints

The proposed development will be serviced by a municipal water supply and a private onsite sewage system. The property is surrounded mostly by low density residential dwellings and agricultural lands. There are no special constraints warranted for the proposed development serviced by a private sewage system other than those related to the regulatory requirements under Part 8 of the Ontario Building Code (OBC).

3.2 Estimate of Daily Sewage Flow

The wastewater generated by the proposed church will be of domestic quality, consisting primarily of grey water and blackwater uses generated by the washroom facilities. An occupancy type of analysis is considered to be best suited for estimating the total daily design sanitary sewage flow (TDDSSF) for a facility of this nature.

The weekend sewage flows for the church are expected to vary significantly from the weekday sewage flows. Considering the large fluctuation in the weekly flows it is recommended that the proposed sewage system incorporate a balancing tank system to even out the daily sewage flow to the leaching bed throughout the week. In doing so, the area requirements for the leaching bed are reduced.

Based on the proposed occupancy of the church the TDDSSF, calculated in accordance with Table 8.2.2.3.B of the OBC, is listed below.

SUMMARY OF ESTIMATED TOTAL DAILY DESIGN SEWAGE SYSTEM FLOW			
Building Use	**Flow Generator**	**Weekday Flow (L/day)**	**Weekend Flow (L/day)**
Main Church	382 seats @ 8 L/seat	-	3056
Chapel	200 seats @ 8 L/seat	1600	-
Classrooms(1)	10 of 10 students for 4 hr max. @ 30 L/student per 8hr	-	1500
Multi Purpose Hall(2)	260 seats max. @ 8L/seat	1600	2080
Guest Rooms(3)	2 rooms @200 L ea.	400	400
Employees	6 @ 75 L ea.	450	450
ESTIMATED TDDSSF		4050	7486

(1) Classrooms are used for Sunday school and are comprised of 5 to 10 students for 2 hour duration only.
(2) Multi Purpose Hall is used by the congregation only typically after main church service.
(3) Guest rooms are used periodically for visiting priests.

The TDDSSF for the proposed church is estimated to be approximately 7,500 L. Sewage systems having a design daily sewage flow of 10,000 L or less are regulated under Part 8 of the Ontario Building Code (OBC). The regulatory authority for Part 8 of OBC in the City of Ottawa is the Ottawa Septic System Office (OSSO).

Considering the large fluctuation in the weekday and weekend sewage flows, it is recommended that the sewage system design incorporate a balancing tank, with a time controlled pumping system, to balance/even out the daily flow applied to the leaching bed. With the use of a balancing system, the septic tankage should be sized for the peak daily flow of 7,500 L and the leaching bed can be sized for a daily flow of 5,000 L.

3.3 Preliminary Sewage System Design Concept

While the detailed engineering design works have not yet commenced for the design of the sewage system, a preliminary sewage system layout concept has been prepared. The purpose of this drawing is to illustrate that a sewage system can be accommodated on the subject site which meets all the pertinent regulatory sizing and separation criteria. Reference should be made to the attached Conceptual Sewage System Layout Plan, Drawing No. PH3209-1. The client should be aware that the attached concept drawing is preliminary only and the final location of the sewage system components and leaching bed area may vary depending on the final grading and site layout plan for the development and the results of a site specific investigation.

For preliminary purposes the sizing criteria for a Class 4 sewage system with a conventional absorption trench style leaching bed has been used. This type of leaching bed requires the greatest footprint of all the OBC approved styles of beds. Other types of Class 4 sewage systems, such as tertiary wastewater treatment systems, could potentially be used at this site and would require a significantly reduced area.

The upper soil stratum underlying the subject site consists of a sand having an estimated saturated hydraulic conductivity of the order of 10^{-3} to 10^{-4} cm/sec, with corresponding percolation time (T-time) of less than 15 mins/cm. As such, the in situ sand layer underlying the site is considered to be suitable for a native mantle associated with a fill-based Class 4 leaching bed system.

A Class 4 septic tank sewage system with a partially raised absorption trench style leaching bed can be installed to service the proposed church. The leaching bed will be required to be partially raised to meet the specified OBC separation distance from the water table. The minimum length of distribution pipe required for the leaching bed is determined by the formula QT/200, where "Q" is the design sewage flow and "T" is the percolation rate of the leaching bed fill. Based on the design sewage flow of 5,000 L/day, a minimum distribution pipe length of 200 m would be required, assuming the percolation rate of the imported leaching bed fill used is 8 min/cm. By way of example, a conventional absorption trench style leaching bed may consist of 2 cells of 6 runs of 17 m length each, having a total distribution pipe length of 204 m. The surficial sand stratum underlying the site will serve as a native mantle.

As it can be seen on the Conceptual Sewage System Layout Plan a leaching bed, as described above, can be easily accommodated on the subject site and meet all the regulatory separation distances.

3.0 APPROVALS FOR THE SEWAGE SYSTEM

The sewage system for the proposed development will be regulated under Part 8 of the OBC. A Sewage System Permit will be required to be obtained from the Ottawa Septic System Office (OSSO) for the construction of the sewage system. As part of the permit application process it will be required that a detailed field investigation be carried out in the area of the proposed sewage system and design drawings for the proposed system be prepared.

4.0 CONCLUSION

Based on the results of these works, it is our opinion that the subject site can accommodate a new Class 4 sewage system to service the proposed church development. Furthermore, the proposed site development concept has allotted sufficient area for a sewage system which satisfies all the regulatory separation criteria.

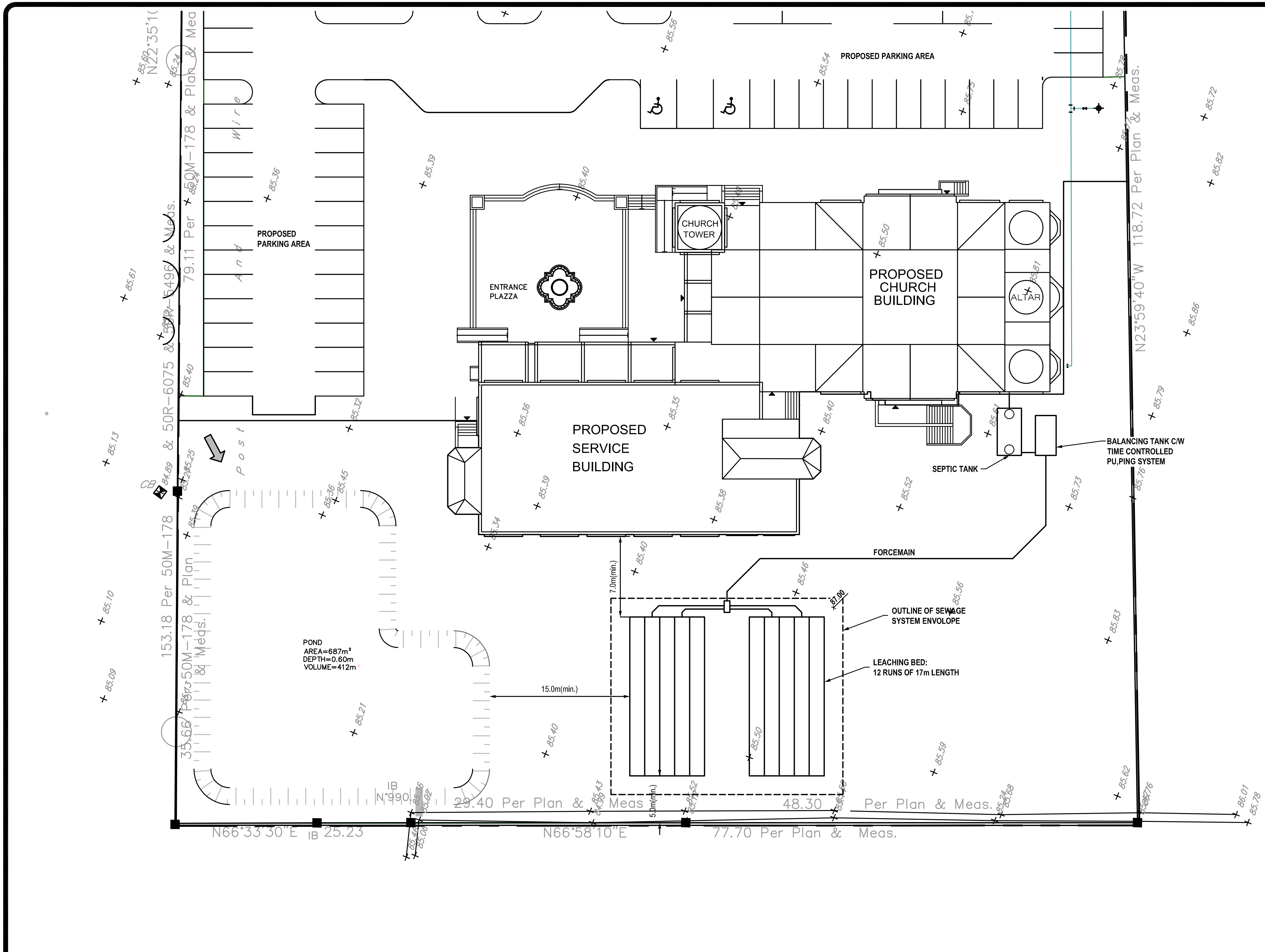
The present report applies only to the project described in this document and is preliminary in nature. Use of this report for purposes other than those described herein or by person(s) other than St. Joseph's Coptic Orthodox Church, or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Should you have any questions regarding this submission, please do not hesitate to contact the undersigned.

Sincerely,

PATERSON GROUP INC.

Albert Van Schie, C.E.T.
Senior Associate



BENCHMARK INFORMATION:

TBM: Top of fire hydrant in front of subject property
Assumed Elevation = 86.87m

REFERENCE:

Base Plan and Topographic Information obtained from Site Servicing and Grading Plan No. SSGP-1, dated November, 2016, by Stantec Consulting Ltd.

10/11/16	Issued with Report PH3209-REP.01	0
DD/MM/YY	DESCRIPTION	REV.

Consultant:

patersongroup
consulting engineers

Client:

STANTEC CONSULTING LTD.

Project:

PROPOSED ST. JOSEPH`S COPTIC ORTHODOX CHURCH

3856, 3866, &3876 NAVAN ROAD OTTAWA (CUMBERLAND), ONTARIO

Drawing:

CONCEPTUAL SEWAGE SYSTEM LAYOUT PLAN

Scale: 1:400	Drawn by: HV
Date: 11/2016	Checked by: AVS

Drawing No.: **PH3209-1**

p:\autocad drawings\hydrogeology\ph32xx\ph3209-1.dwg

Appendix C STORMWATER MANAGEMENT

C.1 RATIONAL METHOD CALCULATIONS

Stormwater Management Calculations

File No: **160410200**
Project: **Navan Coptic Church**
Date: **14/2/2019**

SWM Approach:
Post-development to Pre-development flows

Post-Development Site Conditions:

Overall Runoff Coefficient for Site and Sub-Catchment Areas

Runoff Coefficient Table								
Catchment Type	**Sub-catchment Area ID / Description**		**Area (ha) "A"**		**Runoff Coefficient "C"**		**"A x C"**	**Overall Runoff Coefficient**
Roof	BLDG	Hard	0.173		0.9	0.155		
		Soft	0.000		0.2	0.000		
	Subtotal			0.173			0.15543	0.900
Controlled - Tributary	POND	Hard	0.808		0.9	0.727		
		Soft	0.076		0.2	0.015		
	Subtotal			0.883			0.742056	0.840
Uncontrolled - Tributary	UNC-1, UNC-2	Hard	0.000		0.9	0.000		
		Soft	0.067		0.2	0.013		
	Subtotal			0.067			0.0133172	0.200
Total				**1.123**			**0.911**	
Overall Runoff Coefficient= C:								**0.81**

Total Roof Areas	**0.173 ha**
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.883 ha**
Total Tributary Area to Outlet	**1.056 ha**
Total Uncontrolled Areas (Non-Tributary)	**0.067 ha**
Total Site	**1.123 ha**

Stormwater Management Calculations

Project #160410200, Navan Coptic Church
Modified Rational Method Calculations for Storage

5 yr Intensity I = a/(t + b)^c
City of Ottawa

a =	998.071
b =	6.053
c =	0.814

t (min)	I (mm/hr)
5	141.18
10	104.19
15	83.56
20	70.25
25	60.90
30	53.93
35	48.52
40	44.18
45	40.63
50	37.65
55	35.12
60	32.94

5 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 1.12
C: 0.20

Typical Time of Concentration

tc (min)	I (5 yr) (mm/hr)	Qtarget (L/s)
19.7	70.92	**44.27**

5 YEAR Modified Rational Method for Entire Site

Subdrainage Area: BLDG — Roof
Area (ha): 0.173 — Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	104.19	45.02	7.96	37.06	22.24	102.3	0.00
20	70.25	30.36	8.14	22.22	26.66	107.9	0.00
30	53.93	23.30	8.16	15.14	**27.26**	108.7	0.00
40	44.18	19.09	8.12	10.97	26.33	107.5	0.00
50	37.65	16.27	8.05	8.22	24.65	105.3	0.00
60	32.94	14.23	7.97	6.26	22.55	102.7	0.00
70	29.37	12.69	7.87	4.82	20.25	99.5	0.00
80	26.56	11.48	7.72	3.76	18.02	94.8	0.00
90	24.29	10.49	7.57	2.92	15.78	90.1	0.00
100	22.41	9.68	7.42	2.26	13.55	85.4	0.00
110	20.82	9.00	7.28	1.72	11.35	80.7	0.00
120	19.47	8.41	7.13	1.28	9.20	76.2	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	108.7	0.11	8.16	27.26	69.12	OK

Subdrainage Area: POND (Orifice #1) — Controlled - Tributary
Area (ha): 0.883
C: 0.84

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.19	222.90	37.75	185.15	111.09
20	70.25	153.06	37.75	115.31	138.37
30	53.93	119.41	37.75	81.66	146.99
40	44.18	99.27	37.75	61.52	**147.66**
50	37.65	85.73	37.75	47.98	143.95
60	32.94	75.93	37.75	38.18	137.45
70	29.37	68.46	37.75	30.71	129.00
80	26.56	62.52	37.75	24.77	118.89
90	24.29	57.68	37.75	19.93	107.62
100	22.41	53.65	37.75	15.90	95.40
110	20.82	50.23	37.75	12.49	82.40
120	19.47	47.29	37.75	9.55	68.74

Storage: ɛ Above CB — pond elev 85.50
Orifice Equation: = CdA(2gh)^0.5 — Where C = 0.61

	ICD #1	
Orifice Diameter:	150.00	mm
Invert Elevation	84.80	m
ottom of Pond Elevation	85.10	m
Max Ponding Depth	0.40	m
Downstream W/L	84.50	m

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	85.50	0.63	37.75	147.66	309.00	OK

Subdrainage Area: UNC-1, UNC-2 — Uncontrolled - Tributary
Area (ha): 0.067
C: 0.20

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.19	3.86	3.86		
20	70.25	2.60	2.60		
30	53.93	2.00	2.00		
40	44.18	1.64	1.64		
50	37.65	1.39	1.39		
60	32.94	1.22	1.22		
70	29.37	1.09	1.09		
80	26.56	0.98	0.98		
90	24.29	0.90	0.90		

Project #160410200, Navan Coptic Church
Modified Rational Method Calculations for Storage

100 yr Intensity I = a/(t + b)
City of Ottawa

a =	1735.688
b =	6.014
c =	0.820

t (min)	I (mm/hr)
5	242.70
10	178.56
15	142.89
20	119.95
25	103.85
30	91.87
35	82.58
40	75.15
45	69.05
50	63.95
55	59.62
60	55.89

100 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 1.12
C: 0.20

tc (min)	I (100 yr) (mm/hr)	Qtarget (L/s)
19.7	121.10	**75.59**

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: BLDG — Roof
Area (ha): 0.173 — Maximum Storage Depth: 150 mm
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.56	85.73	8.84	76.89	46.13	130.3	0.00
20	119.95	57.59	9.17	48.42	58.11	140.5	0.00
30	91.87	44.11	9.29	34.82	62.67	144.5	0.00
40	75.15	36.08	9.33	26.75	**64.19**	145.8	0.00
50	63.95	30.70	9.33	21.38	64.13	145.7	0.00
60	55.89	26.84	9.30	17.53	63.12	144.9	0.00
70	49.79	23.90	9.26	14.65	61.52	143.5	0.00
80	44.99	21.60	9.20	12.40	59.51	141.7	0.00
90	41.11	19.74	9.14	10.60	57.22	139.8	0.00
100	37.90	18.20	9.07	9.12	54.74	137.7	0.00
110	35.20	16.90	9.00	7.90	52.12	135.4	0.00
120	32.89	15.79	8.93	6.86	49.41	133.1	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	145.8	0.15	9.33	64.19	69.12	OK

Subdrainage Area: POND (Orifice #1 & #2)
Area (ha): 0.883
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	447.36	66.66	380.70	228.42
20	119.95	303.75	66.66	237.09	284.50
30	91.87	234.90	66.66	168.24	302.84
40	75.15	193.88	66.66	127.22	**305.32**
50	63.95	166.39	66.66	99.73	299.19
60	55.89	146.57	66.66	79.91	287.68
70	49.79	131.53	66.66	64.87	272.47
80	44.99	119.69	66.66	53.03	254.56
90	41.11	110.10	66.66	43.44	234.60
100	37.90	102.16	66.66	35.50	212.99
110	35.20	95.46	66.66	28.80	190.05
120	32.89	89.71	66.66	23.05	166.00

Storage: Surface Storage Above CB — pond elev 85.70
Orifice Equation: Q = CdA(2gh)^0.5 — Where C = 0.61

	ICD #2		ICD #1	
Orifice Diameter:	145.00	mm	150.00	mm
ICD 2 Invert Elevation	85.50	m	84.80	m
Bottom of Pond Elevation	85.10	m	85.10	m
Max Ponding Elev	0.60	m	0.60	m
Downstream W/L	84.50	m	84.50	m

ICD #2	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	85.70	0.27	23.29	305.32	309.00	OK

ICD #1	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	85.70	0.82	43.37	305.32	309.00	OK

Subdrainage Area: UNC-1, UNC-2 — Uncontrolled - Tributary
Area (ha): 0.067
C: 0.25

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	8.26	8.26		
20	119.95	5.55	5.55		
30	91.87	4.25	4.25		
40	75.15	3.48	3.48		
50	63.95	2.96	2.96		
60	55.89	2.59	2.59		
70	49.79	2.30	2.30		
80	44.99	2.08	2.08		
90	41.11	1.90	1.90		

Stormwater Management Calculations

Project #160410200, Navan Coptic Church
Modified Rational Method Calculations for Storage

100	22.41	0.83	0.83
110	20.82	0.77	0.77
120	19.47	0.72	0.72

SUMMARY TO OUTLET

			Vrequired	Vavailable*	
Tributary Area	1.12	ha			
Total 5yr Flow to Sewer	41.61	L/s	147.66	309.00 m^3	Ok
Non-Tributary Area	0.00	ha			
Total 5yr Flow Uncontrolled	0.00	L/s			
Total Area	1.12	ha			
Total 5yr Flow	41.61	L/s			
Target	44.27	L/s			

Project #160410200, Navan Coptic Church
Modified Rational Method Calculations for Storage

100	37.90	1.75	1.75
110	35.20	1.63	1.63
120	32.89	1.52	1.52

SUMMARY TO OUTLET

			Vrequired	Vavailable*
Tributary Area	1.12	ha		
Total 100yr Flow to Sewer	74.92	L/s	305.32	309.00 m^3
Non-Tributary Area	0.00	ha		
Total 100yr Flow Uncontrolled	0.00	L/s		
Total Area	1.12	ha		
Total 100yr Flow	74.90	L/s		
Target	75.60	L/s		

Project #160410200, Navan Coptic Church
Roof Drain Design Sheet, Area BLDG
Standard Watts Model R1100 Accutrol Roof Drain

Rating Curve				Volume Estimation				
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m) Increment	Volume (cu. m) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0032	0	0.025	38	0	0	0.025
0.050	0.0006	0.0063	3	0.050	154	2	3	0.050
0.075	0.0007	0.0071	9	0.075	346	6	9	0.075
0.100	0.0008	0.0079	20	0.100	614	12	20	0.100
0.125	0.0009	0.0087	40	0.125	960	20	40	0.125
0.150	0.0009	0.0095	69	0.150	1382	29	69	0.150

Drawdown Estimate

Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
2.2	355.0	2.2	0.09862
8.3	856.6	6.1	0.33657
20.2	1501.3	11.8	0.75361
39.7	2250.2	19.5	1.37866
68.8	3077.1	29.1	2.2334

Rooftop Storage Summary

Total Building Area (sq.m)		1728
Assume Available Roof Area (sq.	80%	1382.4
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		10
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		69
Estimated 100 Year Drawdown Time (h)		2.1

* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).

From Watts Drain Catalogue

Head (m) L/s

Head (m)	Open	75%	50%	**25%**	Closed
0.025	0.3155	0.3155	0.3155	**0.3155**	0.3155
0.050	0.6309	0.6309	0.6309	**0.6309**	0.6309
0.075	0.9464	0.8675	0.7886	**0.7098**	0.6309
0.100	1.2618	1.1041	0.9464	**0.7886**	0.6309
0.125	1.5773	1.3407	1.1041	**0.8675**	0.6309
0.150	1.8927	1.5773	1.2618	**0.9464**	0.6309

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	2yr	100yr	Available
Qresult (cu.m/s)	0.008	0.009	-
Depth (m)	0.109	0.146	0.150
Volume (cu.m)	27.3	64.2	69.1
Draintime (hrs)	1.0	2.1	

C.2 EXCERPTS FROM MUNICIPAL DRAIN REPORT

Certified Copy

BY-LAW NUMBER 1447

OF THE TOWNSHIP OF CUMBERLAND

IN THE COUNTY OF RUSSELL

A BY-LAW TO PROVIDE FOR DRAINAGE WORK IN THE TOWNSHIP OF CUMBERLAND IN THE COUNTY OF RUSSELL and for borrowing on the credit of the Municipality, the sum of $1,290.00, for completing the same.

Provisionally adopted this 21st day of September, 1961.

WHEREAS the majority in number of the resident and non-resident owners (exclusive of farmer's sons not actual owners), as shown by the last revised assessment roll, of the property hereinafter set forth to be benefited by drainage work have petitioned the Council of the said Township of Cumberland praying that Lots numbered 7 and 8 Concession 11 of the Township of Cumberland be drained by means of a drain or drains.

AND WHEREAS, thereupon the said Council has procured an examination, to be made by Mr. L.P.Stidwill, O.L.S. being a person competent for such purpose, of the said area proposed to be drained and the means suggested for the drainage thereof, and of other lands and roads liable to assessment under the Municipal Drainage Act, and has also procured plans, specifications and estimates of the drainage work to be made by the said Mr. L.P.Stidwill and an estimate to be made by him of the lands and roads to be benefited by such drainage work, and of other lands and roads liable for contribution thereto, stating as nearly as he can the proportion of benefit, outlet liability and injuring liability, which, in his opinion, will be derived or incurred in consequence of such drainage work by every road and lot, or portion of lot, the said assessment so made being the assessment hereinafter by this by-law enacted to be assessed and levied upon the roads and lots or parts of lots hereinafter in that behalf specially set forth and described; and the report of the said Mr. L.P.Stidwill in respect of thereof, and of the said drainage work being as follows,-

To Roll 1962-63-64
1st 1962 $259.05

ENGINEER'S REPORT

ANTOINE CLEROUX DRAIN

TOWNSHIP OF CUMBERLAND
1961

TO THE REEVE AND MEMBERS OF COUNCIL

TOWNSHIP OF CUMBERLAND

Gentlemen:

The following report is respectfully submitted under Section 2 of The Municipal Drainage Act, R.S.O. 1960, Chapter 252, in answer to your instructions regarding the proposed construction of a drain.

This drain starts approximately 100 feet north of the south limit of the Southeast ¼ of Lot 7, Concession 11, at the foot of a rather steep ridge. The drain follows the foot of this ridge in a southwesterly direction for some 500 feet, and then turns south across the Canadian Pacific Railway right-of-way, and continues southerly and westerly until it empties into a rather deep drain running south through a large bog. The total length of drain to be dug is 1,559 feet, affecting an area of approximately 95 acres, part of which is used for market gardening.

PLAN

A plan has been prepared showing the names of the present owners. The boundary limit of the watershed is shown outlined in yellow, and the benefit area is shown in red. The general course and extent of the work is shown in a solid blue line, and turns and intersections with property lines have been referenced to the hub line.

PROFILE

Wooden stakes and numbered markers were driven in average ground bordering the drain at hundred foot intervals, called "Stations". From the results of levels and soundings then taken, a profile of the work has been prepared and a new grade designed.

The hub line and the present ditch bottom are shown by means of the solid and broken irregular lines respectively, while the proposed new grade is indicated by the solid red regular line. The depth of earth to be removed from the present ditch is shown coloured yellow on the profile. The cuts from the hub line to reach the new bottom are shown in feet and decimals of a foot along the top of the profile. A combined reference to the plan and profile should then show the amount of deepening recommended over any particular portion of the scheme.

It is intended that the accompanying plan, profile and specifications form a part of this report, and that they together govern the performance of the work.

FARM CROSSINGS

Allowances have been made for the construction of two farm bridges. The bridges are to be open bottomed structures with a free span equal in feet to one-tenth of the allowance made in dollars for each particular crossing. They should be built as high as the neighbouring banks and constructed and maintained by their respective owners.

In the case of future maintenance, for each ten year interval between the date of construction and a subsequent maintenance scheme, the owner shall be allowed the equivalentof one-third of the farm bridge allowance as allowed by the Engineer at that time. This allowance shall not, however, be increased beyond one hundred percent of the normal payment, and shall not be paid unless the owner did construct a bridge of a sufficient size for which an allowance is herein made, and has kept it in a proper state of repair. This is to be determined by the Engineer. This will apply only when a large enough maintenance program is being carried out to require the services of an Engineer, and when the costs thereof are spread over the entire scheme.

RAILROAD CULVERT

The present railroad culvert, consisting of a triangular shaped concrete pipe with a cross-sectional area of approximately two feet, is too small and too high, and has to be replaced, preferably by a corrugated steel pipe-arch culvert with a cross-sectional area of not less than eleven square feet, laid to grade. This new culvert is to be located approximately 135 feet west of the present culvert, thus bringing it in line with the drain which will be deepened considerably at this point.

In order to qualify for the Provincial Grant of one-third of the costof work done on municipal drains, the Railway Company should prepare a detailed estimate of what this work will cost. The cost of work on this culvert must be shown in the Municipal Treasurer's books in the same way as the general contractor's payments are recorded. The Treasurer should pay the Railway for the cost of the work and then assess them for this cost, less the grant. The Government grant, under the provisions of The Provincial Aid to Drainage Act would then be paid to the Municipal Treasurer as part of the whole scheme.

FUTURE MAINTENANCE

The drain shall be maintained by the Township of Cumberland and the costs spread over the owners in the same proportion as laid down in the attached Schedule of assessment.

ESTIMATE OF COST

900 cu. yds Earth Excavation @ $0.80 per cu. yd.	$ 720.00
Allowances for farm bridges	120.00
Clerk's Fees	75.00
Preparing and Printing By-laws	50.00
Court of Revision	25.00
Engineering Fees, Plans, Reports, Specifications	250.00
Supervision and Inspection	50.00
Total Estimated Cost	$1,290.00

The total cost of $1,290.00 has been apportioned in the following manner;

Outlet Liability-Real Property	$ 605.30
Benefit Liability-Real Property	549.79
Outlet Liability-United Counties	35.20
Outlet Liability-Canadian Pacific Railway	26.70
Benefit Liability-Canadian Pacific Railway	73.01
Total	$1,290.00

Under the provisions of The Provincial Aid to Drainage Act, R.S.O. 1960, the Province of Ontario pays a grant of one-third of the cost of municipal drains. In this case, with an estimated cost of $1,290.00. this grant would amount tp $430.00. leaving a total of $860 to be paid by the lands and roads herein assessed.

The attached schedule of assessments has been prepared to show the cost which would be borne by the individual ratepayers were there no such grant. Each owner assessed herein will, therefore, be entitled to a one-third reduction in his assessment upon receipt of this grant.

Trusting that this report will meet with your approval, Gentlemen,

We have the honour to be
Your obedient Servant

STIDWILL & ASSOCIATES LIMITED

Cornwall, Ontario August 3, 1961. Per L.P.Stidwill C.E., O.L.S.

SCHEDULE "A"

SCHEDULE OF ASSESSMENT

Name of Owner	Con.	Lot or Part		Acres Owned	Acres Drained	Outlet Liability	Benefit
Antoine Cleroux 301 473	11	N.E.Pt.N.W.$\frac{1}{4}$	8	11	9	$68.63	$305.65
Remi Cleroux 301 467	11	Pt. S.W $\frac{1}{4}$	7	39.33	18	137.27	105.73
Antonio 301 471	11	Pt. S W $\frac{1}{4}$	8	34.97	4	31.53	138.41
Martial Cleroux 301 466	11	Pt. NW $\frac{1}{4}$	7	0.4	0.4	2.93	
Edward Cleroux 301 465	11	Pt. S$\frac{1}{2}$-N$\frac{1}{2}$-W$\frac{1}{2}$	7	20.64	5	34.46	
Lucien Cleroux 301 436	11	S E Pt.NW$\frac{1}{4}$	7	0.16	0.16	1.10	
Lucien Cleroux 301 435	11	W. Pt.S$\frac{1}{2}$NE$\frac{1}{4}$	7	10.54	10	69.59	
Leon Cleroux 301 437	11	E Pt. NW$\frac{1}{4}$	7	5.42	5.42	37.76	
Ferdinald Cleroux 438	11	Pt.N.W. $\frac{1}{4}$	7	4.52	4.52	31.16	
Felix Robinson 301 464	11	W Pt. N $\frac{1}{4}$	7	4.70	1	6.45	
Lucien Dumas 301 440	11	Pt. N $\frac{1}{4}$	7	8.50	6	41.36	
Gaetan Rochon 301 443	11	S Pt.	6	34.16	3.5	12.61	
Alex Robinson 301 412	11	N $\frac{1}{2}$-NE $\frac{1}{4}$	7	24.8	4	14.45	
R. & A. Prest 301 413	11	E Pt. N$\frac{1}{2}$-NE$\frac{1}{4}$	7	14.46	1.5	5.57	
Philias Robinson 301 430	11	E $\frac{1}{2}$ of E $\frac{1}{2}$ of W $\frac{1}{2}$ of SE $\frac{1}{4}$	7	6.25	2	12.61	
Gerard Cleroux 301 432	11	W $\frac{1}{2}$ of E $\frac{1}{2}$ of W $\frac{1}{2}$ SE $\frac{1}{4}$	7	6.25	4.5	29.70	
Gerard Cleroux 301 433	11	E Pt. W $\frac{1}{4}$ SE $\frac{1}{4}$	7	0.22	0.07	.44	
Aldema Cleroux 301 4330	11	Pt. W $\frac{1}{4}$ SE$\frac{1}{4}$	7		7.5	52.28	
Adrien Lavergne 301 410	11	W Pt.W$\frac{1}{4}$-SE$\frac{1}{4}$	7	0.91	0.91	6.38	
Antonio Cleroux 301 472	11	Pt.W Pt.NE$\frac{1}{4}$	7	8.7	1.5	9.02	
United Counties of Prescott & Russell Rg.	11	County Road #1			2.5	35.20	
Canadian Pacific Railway	11	Pt. Lot 7			2	26.70	73.01
						$667.20	$622.80

SCHEDULE "B"

FARM BRIDGES

Name of Owner	Lot or Part	Allowance
Antoine Cleroux	N E Pt. NW $\frac{1}{4}$ Lot 8, Con. 11	$60.00
Antonio Cleroux	S Pt. NW $\frac{1}{4}$ Lot 8, Con. 11	$60.00

AND WHEREAS the said Council is of the opinion that the drainage of the area described is desireable;

THEREFORE the Council of the said Township of Cumberland pursuant to the provisions of The Municipal Drainage Act, enacts as follows;-

1. The said report, plans, specifications, assessments and estimates are hereby adopted, and the drainage work as therein indicated and set forth shall be made and constructed in accordance therewith.

2. The Reeve of the said Township may borrow on the credit of the Corporation of the said Township of Cumberland the sum of ONE THOUSAND TWO HUNDRED AND NINETY......00/100 DOLLARS ($1,290.00), being the funds necessary for the work not otherwise provided for, and may issue debentures of the Corporation to that amount in sums of not less than $50.00 each, payable within three years from the date of the said debentures with interest at the rate of six per cent per annum, that is to say, in THREE consecutive equal, annual payments to be made of such amounts that the aggregate amount payable for principal and interest in any year shall be equal as nearly as possible to what is payable for principal and interest during each of the other years of the said period of THREE years, such debentures to be payable at the Royal Bank of Canada, Navan, ONTARIO, or at such other banking office as may be agreed upon between the purchaser or purchasers of the said debentures and the said Reeve

3. For paying the sum of SIX HUNDRED AND TWENTY-TWO......80/100 DOLLARS ($622.80) the amount charged against the said lands and roads for benefit, and the sum of SIX HUNDRED AND SIXTY-SEVEN .....20/100 DOLLARS ($667.20), the amount charged against said lands and roads for outlet liability, there being no assessments charged against the lands and roads belonging to or controlled by the Municipality, and for covering interest thereon for THREE years at the rate of SIX (6) per centum per annum, and the following total special rates over and above all other rates shall be assessed, levied and collected (in the same manner and at the same time as other taxes are levied and collected) upon and from the undermentioned lots and parts of lots and roads, and the amount of the said total special rates and interest against each lot or part of lot respectively shall be divided into three equal parts, and one such part shall be assessed, levied and collected as aforesaid, in each year of three years, after the final passing of this By-law, during which the said debentures have to run.

SCHEDULE "A" SCHEDULE OF ASSESSMENT

Name of Owner	Con.	Lot or Part		Acres Owned	Acres Drained	Outlet Liability	Benefit	Annual Assessment for 3 years @ 6%
Antoine Cleroux	11	N E Pt.N W $\frac{1}{4}$	8	11	9	$68.63	$305.65	$140.02
Remi Cleroux	11	Pt. S W $\frac{1}{4}$	7	39.33	18	137.27	105.73	90.91
Antonio Cleroux	11	Pt. S W $\frac{1}{4}$	8	34.97	4	31.53	138.41	63.58
Martial Cleroux	11	Pt.N W $\frac{1}{4}$	7	0.4	0.4	2.93		1.10
Edward Cleroux	11	Pt. S$\frac{1}{2}$N$\frac{1}{2}$W$\frac{1}{2}$	7	20.64	5	34.46		12.89
Lucien Cleroux	11	S E Pt NW$\frac{1}{4}$	7	0.16	0.16	1.10		.41
Lucien Cleroux	11	W Pt.S$\frac{1}{2}$NE$\frac{1}{4}$	7	10.54	10	69.59		26.03
Leon Cleroux	11	EPt.NW$\frac{1}{4}$	7	5.42	5.42	37.76		14.13
Ferdinand Cleroux	11	Pt.NW$\frac{1}{4}$	7	4.52	4.52	31.16		11.66
Felix Robinson	11	W Pt.N $\frac{1}{4}$	7	4.70	1	6.45		2.41
Lucien Dumas	11	Pt. N $\frac{1}{4}$	7	8.50	6	41.36		15.47
Gaetan Rochon	11	S Pt.	6	34.16	3.5	12.61		4.72
Alex Robinson	11	N$\frac{1}{2}$-NE$\frac{1}{4}$	7	24.8	4	14.45		5.41
R & A Prest	11	E Pt.N$\frac{1}{2}$NE $\frac{1}{4}$	7	14.46	1.5	5.57		2.08
Phileas Robinson Jr.	11	E$\frac{1}{2}$ of E$\frac{1}{2}$ of W$\frac{1}{2}$ of SE$\frac{1}{4}$	7	6.25	2	12.61		4.72
Gerard Cleroux	11	W$\frac{1}{2}$of E$\frac{1}{2}$ of W$\frac{1}{2}$ SE$\frac{1}{4}$	7	6.25	4.5	29.70		11.11
Gerard Cleroux	11	E Pt W$\frac{1}{4}$SE$\frac{1}{4}$	7	0.22	0.07	.44		.16
Aldema Cleroux	11	Pt. W $\frac{1}{4}$ SE$\frac{1}{4}$	7		7.5	52.28		19.56
Adrien Lavergne	11	W PtW$\frac{1}{4}$SE$\frac{1}{4}$	7	0.91	0.91	6.38		2.39
Antonio Cleroux	11	Pt.WPt.NE$\frac{1}{4}$	7	8.7	1.5	9.02		3.37
United Counties of Prescott & Russell	11	County Road #1			2.5	35.20		13.17
Canadian Pacific Railway	11	Pt. Lot	7		2.	26.70	73.01	37.30
						$667.20	$622.80	$ 482.60

4. Instead of being published in a newspaper, this By-law, together with a Notice of the Sitting of the Court of Revision thereunder, shall be printed and served or mailed in accordance with the provisions of Section 25 of The Municipal Drainage Act.

5. This By-law may be cited as The Antoine Cleroux Construction By-law and shall come into force and effect upon and after the final passing thereof.

BY-LAW READ A FIRST AND SECOND TIME AND PROVISIONALLY ADOPTED THIS 21st day of September, A.D. 1961.

R.J.Kennedy
Clerk.

Nelson Charlebois
Reeve.

BY-LAW READ A THIRD TIME AND PASSED, SIGNED AND SEALED IN OPEN COUNCIL THIS ________

day of ______________ A.D. 1961

____
Clerk

____
Reeve.

NOTICE

NOTICE is hereby given that a COURT OF REVISION will be held at the Township Hall at Leonard, Ontario, in the Township, of Cumberland, in the County of Russell, on Monday the 23rd day of October, 1961, commencing at 8 o'clock in the evening, for the hearing and trial of appeals made against the foregoing assessment, or any part thereof in the manner provided by the Municipal Drainage Act.

NOTICE of such appeals must be served on the Clerk of the Municipality at least TEN days prior to the first sittings of the said Court.

AND FURTHER NOTICE IS HEREBY GIVEN that anyone intending to have the said By-law or any part thereof, quashed, must not later than ten days after the final passing thereof serve notice in writing upon the Reeve or other Head Officer, and upon the Clerk of the said Municipality of his intention to make application for that purpose to the Drainage Referee during the six weeks next ensuing the final passing of this By-law.

Dated this 21st day of September, A.D. 1961.

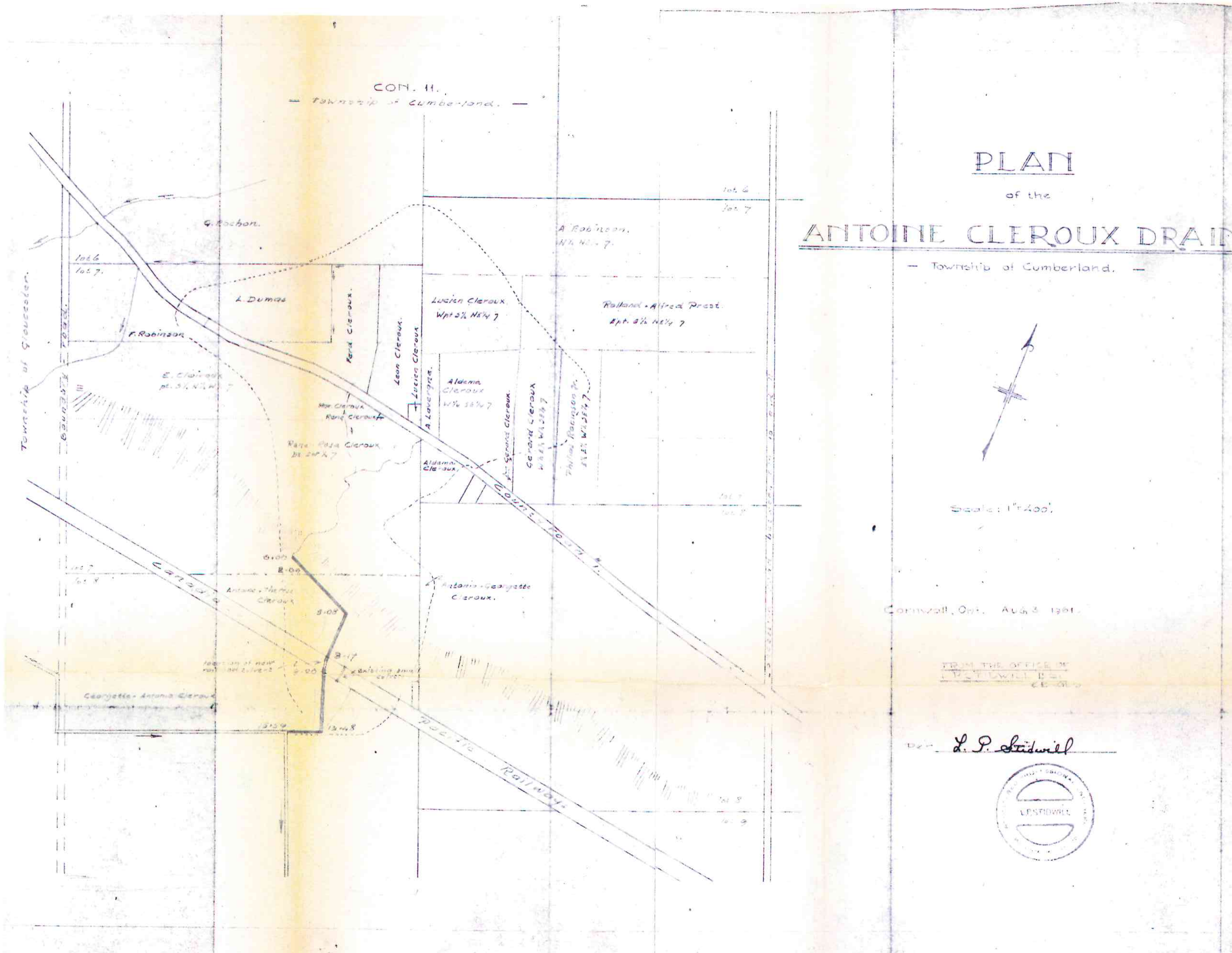
R.J.Kennedy,
Clerk of the Township of Cumberland
Cumberland, Ontario.

I, R.J.Kennedy, Clerk of the Corporation of the Township of Cumberland do hereby certify that the foregoing is a true copy of Township of Cumberland By-law No. 1447.

Dated at Cumberland, Ontario,

this 10th day of October, 1961.

R J Kennedy
Clerk.



CON. 11.
— Township of Cumberland. —
PLAN
of the
ANTOINE CLEROUX DRAIN
— Township of Cumberland. —
Scale: 1"=400'
Cornwall, Ont. Aug 3 1961.
FROM THE OFFICE OF
L. P. STIDWILL
L. P. Stidwill
L.P.STIDWILL
G. Rochon.
lot 6
lot 7
L. Dumas
F. Robinson
E. Cleroux
Ferd. Cleroux
Leon Cleroux
Lucien Cleroux
A. Lavergne
Aldama Cleroux
Lucien Cleroux
A. Robinson.
Rolland - Alfred Prest
Gerard Cleroux
Rena - Rosa Cleroux
Aldama Cleroux
Antoine - Georgette Cleroux.
Georgette - Antonio Cleroux
Township of Gloucester
County Road
Canadian Pacific Railway
lot 8
lot 9

REPORT ON

THE EDOUARD CLEROUX MUNICIPAL DRAIN

FOR

THE TOWNSHIP OF CUMBERLAND

BY

McNEELY, LECOMPTE & ASSOCIATES LTD.
Consulting Civil Engineers

ROCKLAND JUNE, 1970.

TEL. 763-5486
236-1891

P. O. BOX 640
ROCKLAND, ONTARIO

McNEELY, LECOMPTE & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS
INGENIEURS CIVILS - CONSEILS

ENGINEER'S REPORT

THE EDOUARD CLEROUX MUNICIPAL DRAIN

THE TOWNSHIP OF CUMBERLAND

1970

TO THE REEVE AND MEMBERS OF COUNCIL,
THE TOWNSHIP OF CUMBERLAND

Gentlemen:

In answer to a petition of the majority of the property owners concerned, requesting drainage of lands in Lot 7 of Concession XI of the Township of Cumberland, under Section 3 of the Drainage Act 1962-63 and by reason of a resolution passed in Council, we have made a survey of the drain, prepared plan, profile and schedule of assessment for the above requested drainage and report as follows.

This drain shall be called the Edouard Cleroux Municipal Drain.

PRESENT CONDITION OF DRAIN

In its present condition this drain does not provide sufficient outlet of lands within Lot 7 of Concession XI, Township of Cumberland.

Inadequate drainage has resulted from the filling in of the existing ditch by loose fine sandy soils. It is recommended that the owners of the properties involved seed the new ditch side slopes as soon as possible after construction is completed to minimize future maintenance.

EXTENT OF DRAIN

It is proposed that this Municipal Drain commence at a point 565'$\pm$ southeast and 60'$\pm$ north of the intersection of the North boundary lines of the Regional Municipality of Ottawa-Carleton Road No. 28 road allowance and Lot 7 of Concession XI, Township of Cumberland, run southerly for 60' then southeasterly outside the north limits of said Regional road allowance for 170' then southerly for 70' crossing Regional Road No. 28 then westerly for 700' terminating with sufficient outlet at Station 10 + 00 in Lot 7 of Concession XI, Township of Cumberland.

This Municipal Drain is 1000 ft. long and drains approximately 13 acres.

PLAN

A plan has been prepared showing the boundary limits of the watershed in yellow and the benefit in red. The general course and extent of the proposed work is shown by means of a heavy line. The names of the owners are written in their respective properties and have been obtained with the assistance of the Township Clerk. Turns and intersections with the property lines have been referenced to the hub line.

PROFILE

Wooden hubs and numbered markers were driven in average ground bordering the drain at one hundred foot intervals called "stations". From the results of levels and soundings taken, a profile was plotted and new gradelines were designed. The hub line, the existing ditch bottom and the new design gradelines are shown on the profile. The depth of earth to be removed is shown coloured yellow.

DITCH RELOCATION

The existing ditch located from Station 0 + 60 to Station 2 + 30 along the Regional Municipality of Ottawa-Carleton road allowance through Lot 7 of Concession XI, Township of Cumberland, shall be relocated to the property side of the NW¼ Lot 7 of Concession XI as part of this drainage works.

ALLOWANCE FOR FENCES

An allowance is made for the construction of a fence along the private property side of the ditch from Station 0 + 60 to Station 2 + 30 of this Municipal Drain.

For this allowance the owner shall construct the fence a minimum of 2' outside the top edge of the drain.

ALLOWANCE FOR LAND TAKEN

An allowance is made to the owner of Pt. NW¼ Lot 7 of Concession XI, Township of Cumberland, for land taken from Station 0 + 60 to Station 2 + 30 as part of this drainage works.

REGIONAL MUNICIPALITY OF OTTAWA-CARLETON ROAD CULVERT

The existing 24"Ø C.S.P. crossing the Regional Municipality of Ottawa-Carleton Road No. 28 appears to be in poor condition. It is recommended that this culvert be replaced with one having an equivalent effective size opening or better at an elevation 2'-6" lower than its present inverts, if proper drainage of surrounding lands is to be achieved.

No allowance has been made for this work as it can be more readily done under the Highways Improvement Act, under which Act the Municipality obtains an eighty percent subsidy on the cost of the work approved by the Department of Highways of Ontario.

FUTURE MAINTENANCE

This drain shall be maintained by the Township of Cumberland and the cost of such future maintenance shall be charged against the owners of the lands and roads herein assessed for its construction, and in the same relative proportions as shown in the attached schedule of assessment.

ESTIMATE OF COST

868 cu. yds. of Earth Excavation @ $0.75/cu. yd.	$ 651.00
Allowance for Fencing	51.00
Allowance for Land Taken	39.00
Contingencies	110.00
Preparing and Printing By-Laws	100.00
Council Meetings	100.00
Court of Revision	85.00
Advertising Contract	65.00
Clerk's Fees, Township of Cumberland	150.00
Engineer's Fees - report, plan, specifications	606.68
Supervision and Inspection	150.00
TOTAL ESTIMATED COST	$2,107.68

This total estimated cost of $2,107.68 has been apportioned in the following manner.

Outlet Liability - Real Property - Township of Cumberland	$952.04
Benefit Liability - Real Property - Township of Cumberland	831.41
Outlet Liability - Roads - Regional Municipality of Ottawa-Carleton	101.80
Benefit Liability - Roads - Regional Municipality of Ottawa-Carleton	222.43

Under the provisions of the Drainage Act 1962-63, the Province pays a grant of one-third the cost of Municipal Drains. For certain areas including the Township of Cumberland a special grant of another one-third is given for drainage works.

In this case, with an estimated cost of $2,107.68 these grants will amount to $1,405.12, leaving a total amount of $702.56 to be paid by the lands and roads herein assessed. The attached schedule of assessments has been prepared to show the costs which would have been borne by the individual ratepayers, were there no such grant. Each owner herein assessed will therefore be entitled to a two-thirds reduction in his assessments upon receipt of these grants.

Trusting that this will meet with your approval, gentlemen, we respectfully submit this report.

REGISTERED PROFESSIONAL ENGINEER
P. A. McNEELY
PROVINCE OF ONTARIO

June 30th, 1970.

Philip McNeely

Philip McNeely, P.Eng.

SCHEDULE OF ASSESSMENT

SCHEDULE "A"

THE TOWNSHIP OF CUMBERLAND

Name of Owner	Conc.	Lot or Part	Acres Drained	Outlet Liability	Benefit
Felix Robinson	XI	W.Pt. N½W½ Lot 7	1.76	137.00	152.00
Lucien Dumas	XI	Pt. NW¼ Lot 7	2.62	272.00	332.14
Edouard Cleroux	XI	Pt. S½N½W½ Lot 7	6.97	543.04	347.27
Regional Municipality of Ottawa-Carleton					
Regional Road No. 28 through Lot 7 of Concession XI, Township of Cumberland			1.03	101.80	222.43

SCHEDULE OF ASSESSMENT

SCHEDULE "B"

ALLOWANCE FOR FENCING

Name of Owner	Conc.	Lot or Part	Allowance
Lucien Dumas	XI	Pt. N.W.¼ Lot 7	$51.00

SCHEDULE OF ASSESSMENT

SCHEDULE "C"

ALLOWANCE FOR LAND TAKEN

Name of Owner	Conc.	Lot or Part	Allowance
Lucien Dumas	XI	Pt. N.W.¼ Lot 7	$39.00

McNEELY, LECOMPTE & ASSOCIATES LTD.

GENERAL SPECIFICATIONS

FOR

THE EDOUARD CLEROUX

MUNICIPAL DRAIN

THE TOWNSHIP OF CUMBERLAND

1. These specifications are drawn up to cover the work as outlined in the Engineer's report on the drain, and the Engineer's report forms part of these specifications.

Where there is any doubt as to the meaning or intention of the specifications, it shall be the Contractor's duty to obtain a written ruling from the Engineer before proceeding with the work.

2. EXTENT OF WORK: The work shall consist of the necessary excavation to the line, grades and dimensions as shown on the plan, profile, report and specifications for the drain. The cutting of brush, grubbing of roots and removal and disposal of excavated materials as herein specified must be carried out. Straightening where indicated must be done. The supply and installation of culverts, the construction of fences and the supply and/or construction of other items as shown in the Tender also form part of this Contract.

3. CONSTRUCTION: In all cases construction of the drainage project shall start at the outlet of the drain project and continue progressively up the specified grades.

4. SUPPLY OF LABOUR AND MATERIALS: The Contractor shall supply all materials, labour, equipment, tools, machinery etc., for the full and proper completion of this work in accordance with the specifications, plan and profile. All work must be done in a neat and workmanlike manner to the Engineer's satisfaction.

5. ROADS TO BE KEPT OPEN: All roads, public and private, are to be kept open and in a passable condition during the continuance of this work.

6. RELIEF DITCHES: Should the Contractor deem it necessary to dig relief ditches on any part of this work, he shall do so and re-fill same entirely at his own expense.

7. DAMAGES: In the event that buried utilities or buried tile drain outlets are encountered in this project it shall be the Contractor's responsibility to contact the Utility Company or the property owners concerned for further information in regard to the exact location of these utilities and tile drain outlets and to exercise necessary care in construction operations.

The Contractor shall be held liable for all damages to any farm, other property or utility which results from blasting or other operation carried out under this contract.

The Contractor must give owners 10 days written notice in order that they may remove any produce growing on lands adjacent to the proposed work otherwise he shall be held responsible for any damage caused.

8. CLEARING AND GRUBBING: The Contractor shall remove all brush growing in the drain or along the banks, where the excavated material is to be spread. Small brush removed shall be piled in a neat and workmanlike manner so that it may be burned by the farmer.

The Contractor shall also carry out grubbing in a clean and workmanlike manner and shall not spread spoil directly on small brush without previously grubbing unless it is in those areas shown in the specifications in swamp or muck lands.

Payment for this work is deemed to be included in the bid price per cubic yard for earth excavation unless otherwise specified in the contract documents.

Large trees are not to be cut without the written permission of the owner. When it is necessary to cut large trees, stumps shall be removed and piled in a corner of the field from which they were taken, adjacent to, but not closer than 4 feet to the edge of the drain. Stumps are not to exceed one foot in height.

Trees having a stump diameter of 6" or more are to be trimmed and cut into log or cord wood lengths and piled clear of the spread materials so that they might be salvaged by the farmer.

At locations where the drain passes through bush or wood lots, a strip of land will be cleared along one side of the drain only. The locations where such clearings will be required and the width of the proposed clearing will be determined by the Engineer. Payment for this work will be made under the Tender Item "Clearing".

9. DISPOSAL OF MATERIALS: The excavated materials shall be disposed of in such a way that the minimum damage is caused to lands and crops. It shall be taken back from the edge of the finished ditch at least 10 feet, then spread evenly over the adjoining lands, a depth not to exceed the elevation of the adjoining lands by more than six (6) inches in cultivated land and by more than twelve (12) inches in other areas. The completed work is to have a neat, smooth appearance. No natural or man made drainage shall be interrupted by this operation.

Hardpan excavated from the drain is to be taken back four (4) feet from the edge of the drain and piled so that the property owners may dispose of it.

The Contractor shall dispose of all boulders with a volume of 1 cubic foot or greater by piling them in a corner of the field from which they were taken, adjacent to, but not closer than four (4) feet from the edge of the drain so that they might be disposed of by the farmer.

The unit price bid in the Tender for each material will be deemed to include payment for the work as aforementioned.

10. STRAIGHTENING: When straightening occurs, the Contractor shall fill in the upper end of the old course from shoulder to shoulder for a distance of twenty (20) feet back from the new course. The remainder of the excavated material is to be distributed in the old ditch when the intervening distance does not exceed 100 feet. Where the distance does exceed 100 feet the shoulders of the banks of the old course shall be pushed into the ditch and the former course so filled and graded that no low spots remain, thereby affording good drainage towards the lower end of the straightening and leaving a ditch section through which the farmer may drive and gradually reclaim the land.

Payment for excavation will be deemed to include payment for all spreading and material disposal as aforementioned.

11. RELOCATION: At locations where the drain is to be removed from a road allowance to the property side of the fenceline, excavated materials from this construction are to be used to fill the present road drain in such a manner as to allow the water from the road to enter the new drain. The road ditch will be connected to the new drain at culvert locations and at the upper end of the relocation. Excess materials are to be disposed of on the adjoining lands as above.

Payment for this work is deemed to be included in the bid price per cubic yard for earth excavation.

12. DESCRIPTION OF DITCH: The bottom of the drain shall be brought to an even grade between the stations so that no water may lie stagnant therein.

The Physical Dimensions of the drain shall be as follows:

Station to Station	Bottom Width	Side Slopes Vertical	Side Slopes Horizontal

See Profile for this information

Side slopes specified must be maintained. The Contractor shall not be allowed to increase the bottom width without maintaining the specified side slopes.

Rock cuts, should they occur, shall have vertical sides and a width equal to the specified bottom width of the drain.

13. GRADES: Grades will be given to the Contractor upon receiving written notice five days previous that the same are required.

14. COMPLETION: At the date fixed for the completion of the work there shall be a continuous ditch or watercourse of the dimensions set forth in the drawings, specifications, etc., and any material which may be accumulated or fallen into the ditch shall be removed by the Contractor except when it can be clearly shown that such accumulation was not caused by any action or want of action on the part of the Contractor or his employees.

15. CLASSIFICATION: Excavation shall be paid for under the following headings: Earth, Hardpan and Rock.

"Earth" shall mean clay, silt, sand, gravel, small stones or muck.

"Hardpan" shall mean such materials other than rock that cannot be removed by the ordinary means but requires the use of picks, bars or explosives in its removal. Shale rock or layer rock that can be excavated without the use of dynamite or ripper will be classified as Hardpan.

"Rock" shall mean strata rock or boulders measuring 14 cubic feet or more.

The Engineer is to be notified that rock has been encountered and no blasting will take place until the necessary measurements have been taken.

No claim for reclassification of material will be allowed without positive evidence as to the exact location where reclassification is necessary, and in the case of boulders, the exact size (height, width and circumference measured in two directions) of each boulder encountered.

In the case where hardpan and rock have not been classified and quantities are included in the Tender form based on insufficient information in the eventuality that hardpan and rock are encountered, no claim for classification of said materials will be allowed without positive evidence as to the exact location where classification is necessary.

The unit price bid in the tender for each material shall include the removal of any brush from the ditch or banks, the removal and replacing of fences and any other work required in order to execute the contract.

16. CENTRE LINE: The Centre Line for construction shall be the Centre Line of the present ditches except:

(1) At locations where the distance from fence to ditch shoulder is not sufficient to allow the proper width of ditch bottom and side slope, in which case the Centre line will be moved away from the fence a sufficient distance to allow for the proper sloping of the banks.

(2) At locations where a row of large trees is growing close to one bank of the drain the Centre line will be moved away from these trees a sufficient distance to allow for the proper sloping of the banks.

(3) At locations where the drain is to be removed from a Road Allowance the Centre line will be staked by the Engineer when required to allow for a clear berm of 4 feet in width between the fenceline and the edge of the drain.

(4) At locations where there is not a present water course or the present water course is to be straightened, the Centre line will be as staked in the field by the Engineer.

(5) At locations where the present drain meanders, the course of the new construction will follow straight lines to radius turns.

17. FARM BRIDGES AND CULVERTS:

Crossings to be Retained

(a) The Contractor shall clean out and excavate to grade under farm crossings and bridges that are considered to be large enough for their locations. Where the farm bridges are shown to be retained, the Contractor may remove the flooring for the purpose of cleaning but must replace same in as good condition as before removal.

Payment for this work is deemed to be included in the bid price per cubic yard for earth excavation.

New Crossings to be Supplied and Constructed by the Owner.

(b) In locations where the replacement of bridges is necessary or in the case of first construction, Property Owners will be given an allowance towards the construction of farm bridges and it will be the responsibility of the property owners to construct suitable bridges. Where a new corrugated metal structure is to be supplied by the Owner, the Contractor shall assist in the installation of same when the culvert is at the site at the time the ditch at the culvert location is being excavated by the Contractor.

New Crossings to be Supplied and Constructed by the Contractor.

(c) Where as directed in the report it becomes the Contractor's responsibility to supply and place bridges or culverts in locations where the replacement of bridges is necessary or in the case of first construction the contractor shall supply and install the new culvert and construct the roadway over the culvert, placing a minimum depth of 12" of crushed stone or crushed gravel over the pipe for a distance of 10' on each side of the pipe centerline for the full width of the roadway. Payment for this work will be made in the Tender Item "Installation of Culverts".

The crossing must be maintained at all times during construction or an alternative crossing must be provided to the satisfaction of the owner.

18. ROAD BRIDGES AND CULVERTS: The Contractor shall clean out under any existing open bottomed road culverts or bridges which are of sufficient size, and shall clean out any pipe culvert which has been accepted by the Engineer as being of a satisfactory size and at a proper depth. All excavated materials will be removed from under the open bottomed road culvert or bridge and disposed of as other materials excavated from the drain are disposed of.

Payment for the above work is deemed to be included in the bid price per cubic yard for earth excavation.

Any work other than the above which may be required at the site of the road culvert shall be charged against the road costs and shall be the responsibility of the Road Superintendent concerned. In order to make his equipment available for any excavation required by the lowering or replacing of a pipe culvert, however, the Contractor shall give the Road Superintendent concerned ten clear days notice in writing informing him when he will reach the site of any given culvert requiring work. Should the Road Superintendent prefer to carry out the work using his own equipment and personnel, then the Contractor shall only be required to excavate to the grade up to and away from any such culvert.

19. FENCES:

(a) Fences crossing the drainage works may be opened to allow construction equipment to pass and are to be closed immediately after that piece of equipment has passed. The Contractor shall be held liable for damage to live stock or any inconvenience to the property owner due to the Contractor's neglect to properly close the fences.

(b) Fences which are parallel to the drain and are required to be removed to permit construction must be replaced immediately after the construction is completed on that section of the drain.

(c) All fences removed or opened in order to execute the work must be rebuilt or spliced by the Contractor in such a manner that the fence is returned to its original condition prior to the construction.

Payment for work under sub-sections (a), (b) and (c) as above is deemed to be included in the bid price per cubic yard for earth excavation.

(d) New fences as may be required will be constructed as follows:

1. Wire shall be Page Wire #842, Tight Lock, Canadian made.
2. Fence posts shall be cedar, 8' long, minimum 5" dia. top.
3. Post spacing shall be maximum 20 feet.
4. Posts shall be set at least 4 feet into the ground.
5. Brace panels shall be constructed as required.
6. Corner and end posts, minimum diameter 10" shall be placed as required.

Payment for the supply and installation of fences will be made under the Tender Item "Fencing".

(e) The Centre Line for new fences will be staked by the Engineer where required prior to construction of the new fence.

20. RIGHT TO INCREASE OR DECREASE THE WORK: The Municipality reserves the right when it is deemed advisable by the Engineer to increase or decrease the amount of excavation or to deviate from the line or grade of the proposed work as shown by the drawings, but no variation shall be made from the price per cubic yard named in the contract, in consequence of any such change.

21. SUB-LETTING: No portion of the work is to be sub-let without the consent of the Municipal Council and the Engineer.

22. DATE FOR COMPLETION: The whole work shall be completed on or before the day of 19 . When the Contractor considers that the work is completed he must notify the Engineer in writing that he requires a final inspection thereof.

Upon receiving written notification from the Contractor that the work is ready for its final inspection, the Engineer will carry out such inspection and the costs thereof shall be charged to the Municipality, whether or not the work passes. Should the work not pass this final inspection, however, the Contractor shall then be responsible for any subsequent costs brought about by all extra work necessary for the ultimate final approval of the work, e.g. replacing the hubs, re-inspection, etc., and the final payment to him shall not be approved until such extra costs have been paid by him.

The Contractor shall forfeit the amount of Ten Dollars ($10.00) per day from his contract price for every calendar day's delay in completing the work after the date of completion unless he has been granted an extension of time by the Council of the Municipality.

23. PAYMENT: The Municipality shall make monthly cash payments to the Contractor equal to ninety per cent of the value of the work done according to the certificate of the Engineer if such payment exceeds Three Hundred Dollars ($300.00). The remaining 10% will be retained until thirty (30) days after the whole work has been accepted as complete.

24. CALCULATION OF QUANTITIES: The calculation of quantities will be carried out by the Engineer and will be based on original cross sections taken at intervals determined by the Engineer and theoretical or as constructed cross sections, whichever gives the lesser quantity.

25. RIGHT TO REJECT TENDERS: The Municipal Council reserves the right to reject any or all tenders.

26. WORKMEN'S COMPENSATION: The Contractor shall comply with the regulations of the Workmen's Compensation Board of Ontario.

27. MEANING OF TERMS:

Municipal Council shall mean the Municipal Council of

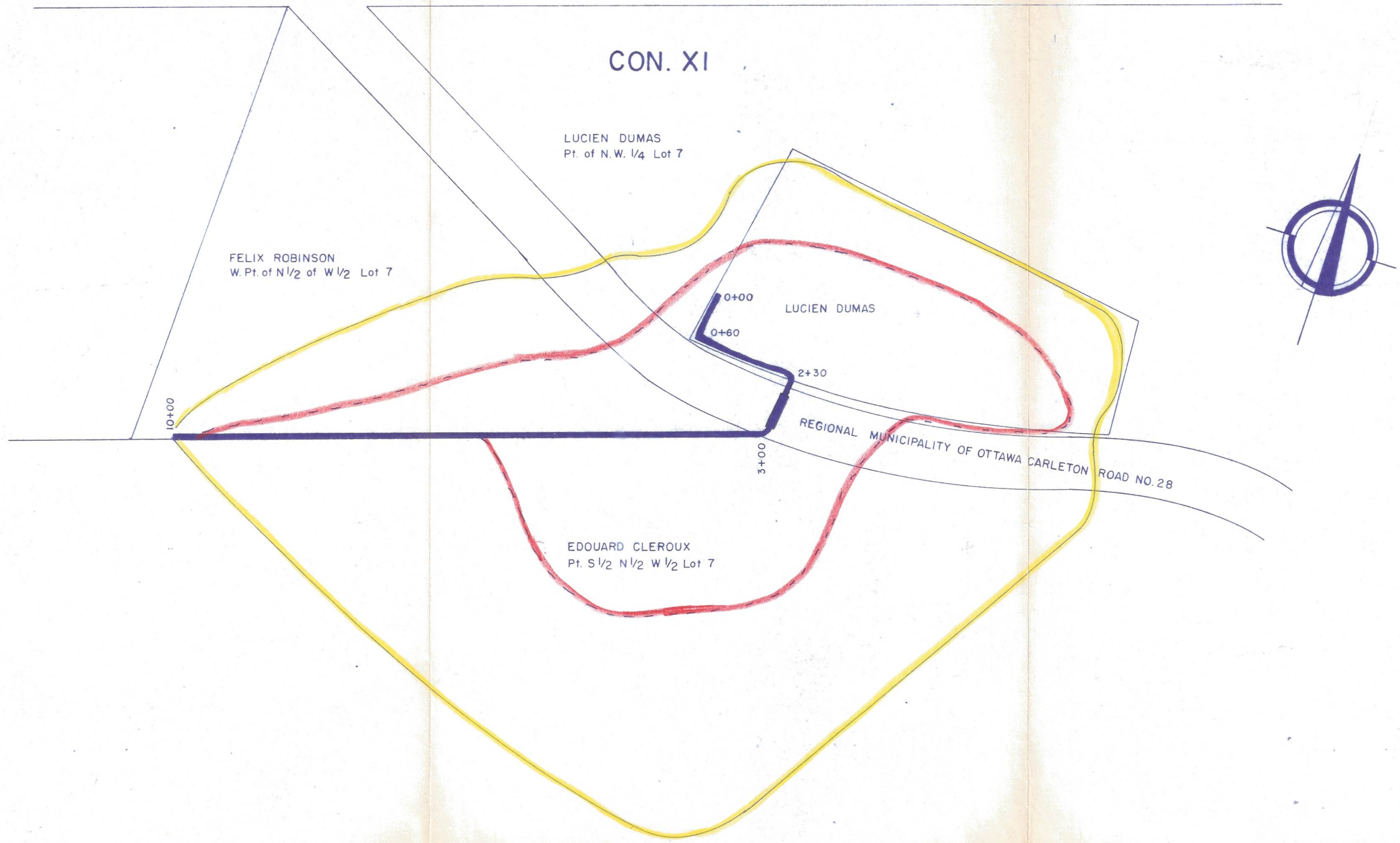
The Township of Cumberland

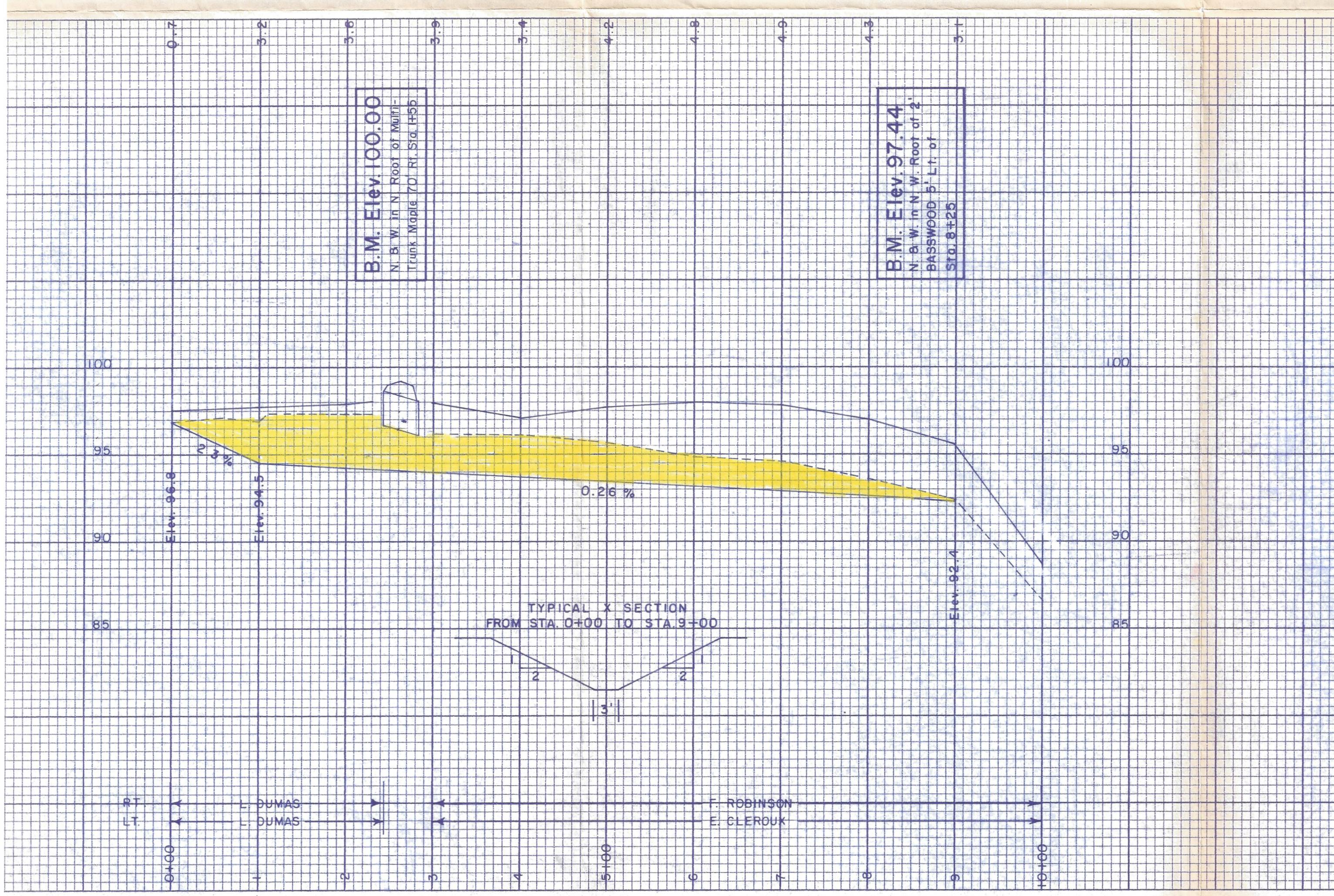
Reeve shall mean the Reeve of the Township of

Cumberland

Engineer shall mean the Consulting Engineers, McNeely, Lecompte & Associates Ltd. or their representative.

Contractor shall mean the person, partnership or corporation undertaking the execution of the work under the terms of the contract.





PLAN & PROFILE OF

EDOUARD CLEROUX MUNICIPAL DRAIN

TOWNSHIP OF CUMBERLAND

McNEELY, LECOMPTE & ASSOC. LTD.
CONSULTING CIVIL ENGINEERS
ROCKLAND ONTARIO

SCALE: 1" = 100' Hor.
1" = 5' Vert.

DATE: JUNE 1970

REGISTERED PROFESSIONAL ENGINEER
P. A. McNEELY
PROVINCE OF ONTARIO

C.3 QUALITY CONTROL VOLUME AND POND DRAWDOWN CALCULATIONS

Minimum Hydraulic Conductivity	0.00029 cm/s		0.005
Maximum Hydraulic Conductivity	0.00174 cm/s		0.065

Relation between infiltration rate and hydraulic conductivity:
y = 529.09x^0.2646

Minimum infiltration rate	61	mm/hr
Maximum infiltration rate	98	mm/hr
Safety Correction Factor:	3.5	
Corrected minimum infiltration rate	18	mm/hr
Corrected maximum infiltration rate	28	mm/hr
Corrected average infiltration rate	23	mm/hr

Storage Volume (cu.m/ha) for Impervious Level

	35%	55%	70%	85%	91.4%
Enhanced (80% TSS removal)	25	30	35	40	42.1
Normal (70% TSS removal)	20	20	25	30	
Basic (60% TSS removal)	20	20	20	20	

per MECP Equation 4.3:

A = 1000V/Pnt Where:

A =	Bottom area of trench (m2)
V =	Runoff volume to be infiltrated (m3)
P =	Percolation Rate of soil (mm/hr)
n =	Porosity of storage media (1.0 for surface storage)
t =	retention time (24-48hrs)

Parcel	Area (ha)	Imp. %	V (m3)	Amin (m2)	A (m2)
Pond	1.10	91.4	46.3	110.3	255

Table 2-8 Soil Hydraulic Loading Rates for Residential Strength Wastewater

SOIL CHARACTERISTICS[1]			PERCOLATION RATES (MIN/2.54 CM)	FIELD SATURATED HYDRAULIC CONDUCTIVITY (KFS) MM/DAY	WASTEWATER LOADING RATES IMPERIAL GALLONS/FT^2/DAY (LITRES/M^2/DAY)		
TEXTURE (USDA)	STRUCTURE						
	SHAPE	GRADE			TYPE 1	TYPE 2	TYPE 3
Gravelly sand	—	Single grain	< 2	>3,500	0.7 (34)	1.4 (68)	2.1 (103)
Coarse to medium sand/loamy sand	—	Single grain	2 – 5	1,500 – 3,500	0.6 (29)	1.2 (59)	1.8 (88)
Fine sand/fine loamy sand	—	Single grain	5 – 15	250 – 1,500	0.5 (25)	1.0 (49)	1.5 (75)
Sandy loam	Massive	structureless	20 – 30	125 – 250	0.3 (15)	0.45 (22)	0.6 (29)
	Platy	weak			0.3 (15)	0.45 (22)	0.6 (29)
		moderate, strong			not recommended	not recommended	not recommended
	prismatic, blocky, granular	weak	10 – 20	250 – 500	0.4 (20)	0.7 (34)	1.0 (49)
		moderate, strong			0.5 (25)	1.0 (49)	1.5 (74)
Loam	massive	structureless	30 – 40	60 – 125	0.2 (10)	0.3 (15)	0.4 (20)
	Platy	weak			0.2 (10)	0.3 (15)	0.4 (20)
		moderate, strong			not recommended	not recommended	not recommended
	prismatic, blocky, granular	weak	20 – 30	125 – 250	0.3 (15)	0.5 (24)	0.7 (34)
		moderate, strong			0.4 (20)	0.8 (39)	1.2 (59)
Silt loam, silt	massive	structureless	40 – 60	30 – 60	0.2 (10)	0.3 (15)	0.4 (20)
	platy	weak			0.2 (10)	0.3 (15)	0.4 (20)
		moderate, strong			not recommended	not recommended	not recommended
	prismatic, blocky, granular	weak	20 – 40	60 – 250	0.3 (15)	0.5 (24)	0.7 (34)
		moderate, strong			0.4 (20)	0.8 (39)	1.2 (59)
Clay loam, sandy clay loam, silty clay loam	massive	structureless	60 – 90	15 – 30	not suitable	not suitable	not suitable
	platy	weak			not suitable	not recommended	not recommended
		moderate, strong			not suitable	not suitable	not suitable
	prismatic, blocky, granular	weak	40 – 60	30 – 60	0.2 (10)	0.3 (15)	0.4 (20)
		moderate, strong			0.3 (15)	0.45 (22)	0.6 (29)
Sandy clay, silty clay, clay	massive	structureless	90 – > 120	< 5.0 – 60	not suitable	not suitable	not suitable
	platy	weak				not recommended	not recommended
		moderate, strong				not suitable	not suitable
	prismatic, blocky, granular	weak				0.15 (7)	0.18 (9)
		moderate, strong				0.2 (10)	0.25 (13)

APPENDIX C – SITE EVALUATION AND SOIL TESTING PROTOCOL FOR STORMWATER INFILTRATION

Site Evaluation and Soil Testing Protocol for Stormwater Infiltration

C1.0 INTRODUCTION

C1.1 Purpose of the Protocol

The purpose of this protocol is to describe evaluation and field testing procedures to:

- Determine if stormwater infiltration best management practices (BMPs) are well suited to a site, and at what locations; and
- Obtain the required data for stormwater infiltration BMP design.

C1.2 When to Conduct Testing

Designers are encouraged to conduct site evaluation and soil testing early in the development planning and design process so that information gained can be incorporated into the design. Chapters 2 and 3 of this guide describe planning and design principles, processes and practices to better integrate stormwater management into the development planning process. It is recommended that site evaluation and soil testing be conducted following the development of a preliminary plan for the proposed development. The designer should possess an understanding of potential BMP types and locations prior to soil testing. On-site tests may be carried out in advance to identify potential BMP types and locations.

C1.3 Who Should Conduct Testing

Qualified professionals, who can substantiate by qualifications or experience their ability to carry out the evaluation, should conduct the soil testing. A professional, experienced in observing and evaluating soil conditions is necessary to ascertain conditions that might affect BMP performance that cannot be thoroughly assessed with testing procedures.

C2.0 SOIL INFILTRATION TESTING: A MULTI-STEP PROCESS

Soil infiltration testing is a four-step process to obtain the necessary information for stormwater management planning and design. The four steps include:

1. Background Evaluation
    - Based on available published and site specific data;
    - Includes consideration of proposed development plan;
    - Used to identify potential BMP types, locations and soil test locations;
    - Done prior to field work; and
    - On-site soil tests may be done to identify/screen potential BMP locations.

2. Test Pit or Soil Boring Observations
    - Includes multiple testing locations;
    - Provides an understanding of sub-surface conditions; and

- Identifies limiting conditions (e.g., aquitard, bedrock or water table elevations).

3. Infiltration Testing
   - Must be conducted on-site;
   - Various testing methods are available; and
   - Different testing methods for screening versus verification purposes.

4. Design Considerations
   - Determination of a suitable infiltration rate for design calculations; and
   - Consideration of desired BMP drawdown period.

C2.1 Step 1. Background Evaluation

Prior to performing testing and developing a detailed site plan, existing site conditions should be inventoried and mapped including, but not limited to:

- Surficial geology and underlying stratigraphy;
- Watercourses (perennial and intermittent), water bodies, wetlands and floodplains;
- Small headwater drainage features;
- Topography, slope, and drainage patterns;
- Existing land cover and land use;
- Natural heritage conservation areas; and
- Other man-made features or conditions that may impact design such as existing nearby structures (buildings, infrastructure, etc.).

A sketch plan or preliminary layout plan for the proposed development should be evaluated, including:

- The preliminary grading plan and areas of cut and fill;
- The location and water surface elevation of all existing, and location of proposed water supply sources and wells;
- The location of all existing and proposed on-site wastewater (septic) systems;
- The location of other features of note such as utility rights-of-way, water and sewer lines, etc.;
- Existing data from borehole, well and geophysical testing; and
- Proposed location of development features (buildings, roads, utilities, etc.).

In Step 1, the designer should determine the potential location of infiltration BMPs. The approximate location of these BMPs should be noted on the proposed development plan and should serve as the basis for the location and number of soil tests to be performed on-site.

Important: If the proposed development is located on areas that may otherwise be suitable for stormwater infiltration BMPs, or if the proposed grading plan is such that potential BMP locations are eliminated, the designer is strongly encouraged to revisit the proposed layout and grading plan and adjust the development plan as necessary.

Development of areas suitable for infiltration BMPs does not preclude the use of subsurface infiltration BMPs for runoff volume reduction and groundwater recharge benefits (e.g., soakaways, infiltration trenches and chambers, perforated pipe systems).

C2.2 Step 2. Test Pit or Soil Boring Observations

Test pits or soil borings provide information regarding the soil horizons and overall soil conditions both horizontally and vertically in that portion of the site. Multiple observations can be made across a site at a relatively low cost and in a short time period. The use of test pits is preferable to soil borings as visual observation is narrowly limited in a soil boring and the soil horizons cannot be observed *in-situ*, but must be observed from the extracted borings.

Test pit excavations or soil borings should extend to a depth of between 2.5 to 5 metres below ground surface or until bedrock or fully saturated conditions are encountered. It is important that the tests provide information related to conditions at least 1.5 metres below the proposed bottom elevation of the infiltration BMP. Test pit trenches should be benched at 1 metre depth intervals for access and infiltration testing. A test pit should never be accessed if soil conditions are unsuitable for safe entry, or if site constraints preclude entry or exit. Where excavation of a test pit to the required depth would create an undesirable or unsafe condition, two soil borings may be conducted instead.

At each test location, the following conditions should be noted and described:

- Soil horizons (upper and lower boundary);
- Soil texture and colour for each horizon;
- Color patterns (mottling) and observed depth;
- Depth to water table (if encountered);
- Depth to bedrock (if encountered);
- Observations of pores or roots (size, depth);
- Estimated type and percent coarse fragments;
- Hardpan or other limiting layers; and
- Strike and dip of soil horizons.

At the designer's discretion, soil samples may be collected at various horizons for additional analyses (e.g., grain size analysis).

The number of test pits or soil borings varies depending on site conditions and the proposed development plan. General guidelines are as follows:

- For infiltration BMPs with footprint surface areas from 50 to 900 square metres, a minimum of two test pits or one test pit and two soil borings are required at, or within 10 metres of the proposed location to determine the suitability and distribution of soil types present;
- For infiltration BMPs with footprint surface areas greater than 900 square metres, a minimum of one test should be conducted for each 450 square metres of

footprint surface area. Tests should be conducted equidistant from each other to provide adequate characterization of the area;

- For linear infiltration BMPs a minimum of one test should be conducted within each soil mapping unit present along the proposed BMP location. Soil borings should be conducted every 50 metres and a test pit should be conducted every 450 metres; and
- For sites with multiple infiltration BMPs, each with footprint surface areas less than 50 square metres, a minimum of one test pit is required and one soil boring per infiltration BMP location is recommended.

The recommendations above are guidelines. Additional tests should be conducted if local conditions indicate significant variability in soil type, geology, water table levels, bedrock or topography. Similarly, uniform site conditions may indicate that fewer tests are required.

C2.3 Step 3. Infiltration Testing

A variety of field tests exist for estimating the infiltration rate of the native soil that include the use of permeameter or infiltrometer devices, percolation tests and empirical relationships between grain size distribution and hydraulic conductivity. At least one test should be conducted at the proposed bottom elevation of the infiltration BMP, plus additional tests at every other soil horizon encountered within 1.5 metres below the proposed bottom elevation. A minimum of two tests per test pit are recommended. More tests are warranted if results from the first two tests are substantially different. The geometric mean value should be used to determine the average infiltration rate for each soil horizon following multiple tests.

Based on field observations, infiltration testing results and the desired drawdown period (typically 48 hours), the designer may elect to modify the proposed bottom elevation of a BMP (see Step 4). Therefore, personnel conducting infiltration tests should be prepared to adjust test locations and depths depending upon observed conditions.

Infiltration testing methods discussed in this protocol include:

- Guelph permeameter test;
- Double-ring infiltrometer test;
- Borehole permeameter test; and
- Percolation test.

There are differences between these methods. Guelph permeameter and double-ring infiltrometer tests estimate the vertical movement of water through the bottom of the test area. The outer ring helps to reduce the lateral movement of water in the soil. Borehole permeameter and percolation tests allow water movement through both the bottom and sides of the test area. For this reason, the measured rate of water level drop in these types of tests must be adjusted to represent the discharge that is occurring on both the bottom and sides of the test hole.

For initial screening of a site for potential BMP types and locations, percolation tests and grain size analyses of samples from soil borings are suitable methods for estimating the infiltration rate of the native soil. Tests should not be conducted in the rain or within 24 hours of significant rainfall events (>15 millimetres depth), or when the temperature is below freezing. The preferred testing period is during April and May. This is the period when infiltration is likely to be diminished by saturated conditions. Percolation tests conducted between June 1 and December 31 should be done following a 24 hour pre-soaking period to simulate field saturated conditions. Pre-soaking is not required for permeameter or infiltrometer test methods.

To verify native soil infiltration rates for design purposes, it is strongly recommended that infiltration tests be carried out with a permeameter or infiltrometer to determine the field saturated hydraulic conductivity (K_{fs}), rather than percolation tests or grain-size analyses. Alternatively, other permeability test procedures that yield a saturated hydraulic conductivity rate can be used, such as formulas developed by Elrick and Reynolds[1], or others for computation of hydraulic conductivity and saturated hydraulic conductivity.

Many *in-situ* methods have been developed for determining field saturated hydraulic conductivity within the unsaturated (vadose) zone of the soil. Detailed testing methods and standards that are available but not discussed in detail in this protocol include (but are not limited to):

- Constant head well permeameter method (i.e., Guelph Permeameter method)[2, 3];
- Constant head double-ring infiltrometer method[3, 4];
- Constant head pressure (single-ring) infiltrometer method[5];

A complete guide for comparing standard methods is presented in ASTM International Designation D5126-90 (2004)[6]. Further detailed discussion on standard methods can also be found in Amoozegar and Warrick (1986)[5].

[1] Elrick, D.E. and Reynolds, W.D. 1992. Infiltration from constant head well permeameters and infiltrometers. In, G.C. Topp, W.D. Reynolds and R.E. Green (Eds.). Advances in measurement of soil physical properties: Bringing theory into practice. Special Publication 30. Soil Society of America. Madison, WI.
[2] Reynolds, W.D., Elrick, D.E. 1986. A method for simultaneous *in-situ* measurement in the vadose zone of field-saturated hydraulic conductivity, sorptivity and the conductivity-pressure head relationship. *Ground Water Monitoring Review*. No. 9. pp. 184-193.
[3] Reynolds, W.D. 1993. Saturated Hydraulic Conductivity: Field Measurement. In, M.R. Carter (ed.). Soil Sampling and Methods of Analysis. Chapter 56. Canadian Society of Soil Science. Lewis Publishers. Ann Arbor, MA.
[4] ASTM International. 2003. Designation D 3385-03, Standard Test Method for Infiltration Rate of Soils in Field Using a Double-Ring Infiltrometer. West Conshohocken, PA.
[5] Amoozegar, A. and Warrick, A.W. 1986. Hydraulic conductivity of saturated soils: field methods. In, A. Klute (ed.) Methods of Soil Analysis. 2nd edition. No. 9 Agronomy. American Society of Agronomy, Madison, WI.
[6] ASTM International. 2004. Designation D5126-90 (2004), Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in the Vadose Zone. West Conshohocken, PA.

For the purpose of designing the infiltration BMP, hydraulic conductivity values (typically in centimetres per second) generated from permeameter or infiltrometer tests must be converted into infiltration rates (typically in millimetres per hour). **It is critical to note that hydraulic conductivity and infiltration rate are two different concepts and that conversion from one parameter to another cannot be done through unit conversion**. Particularly for fine grained soils, there is no consistent relationship due to the many factors involved. Table C1 and Figure C1 describes approximate relationships between hydraulic conductivity, percolation time and infiltration rate. Measured hydraulic conductivity values can be converted to infiltration rates using the approximate relationship described in Figure C1.

Table C1: Approximate relationships between hydraulic conductivity, percolation time and infiltration rate

Hydraulic Conductivity, K_{fs} (centimetres/second)	Percolation Time, T (minutes/centimetre)	Infiltration Rate, 1/T (millimetres/hour)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

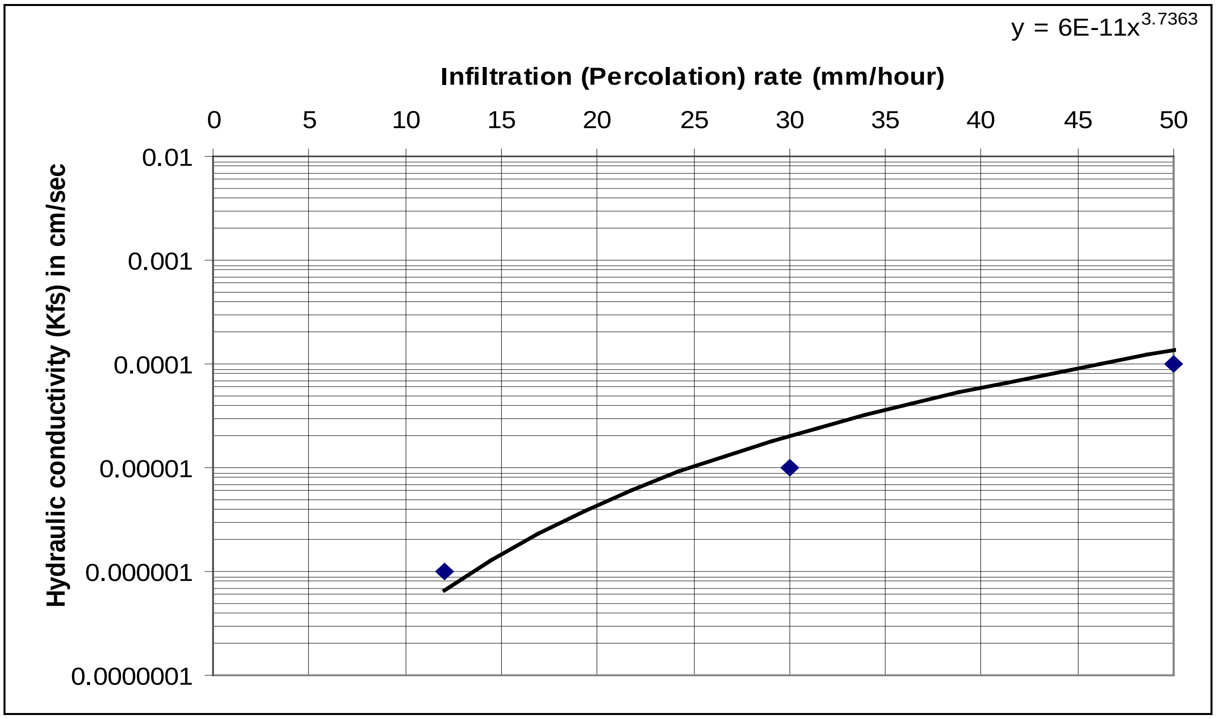
Following testing, the test pits should be refilled with the original soil and the surface replaced with the original topsoil.

The results and locations of all test pits, soil borings and infiltration tests should be included in documents submitted to commenting and approval agencies in support of the development proposal.

C2.4 Step 4. Design Considerations

The infiltration rate used to design an infiltration BMP must incorporate a safety correction factor that compensates for potential reductions in soil permeability due to compaction or smearing during construction, gradual accumulation of fine sediments over the lifespan of the BMP and uncertainty in measured values when less permeable soil horizons exist within 1.5 metres below the proposed bottom elevation of the BMP.

Figure C1: Approximate relationship between infiltration rate and hydraulic conductivity



Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.

The measured infiltration rate (in millimetres per hour) at the proposed bottom elevation of the BMP must be divided by a safety correction factor selected from Table C2 to calculate the design infiltration rate. To select a safety correction factor from Table C2, calculate the ratio of the mean (geometric) measured infiltration rate at the proposed bottom elevation of the BMP to the rate in the least permeable soil horizon within 1.5 metres below the bottom of the BMP. Based on this ratio, a safety correction factor is selected from Table C2. For example, where the mean infiltration rate measured at the proposed bottom elevation of the BMP is 30 mm/h, and the mean infiltration rate measured in an underlying soil horizon within 1.5 metres of the bottom is 12 mm/h, the ratio would be 2.5, the safety correction factor would be 3.5, and the design infiltration rate would be 8.6 mm/h. Where the soil horizon is continuous within 1.5 metres below the proposed bottom of the BMP, the mean infiltration rate measured at the bottom elevation of the BMP should be divided by a safety correction factor of 2.5 to calculate the design infiltration rate.

Table C2: Safety correction factors for calculating design infiltration rates

Ratio of Mean Measured Infiltration Rates[1]	Safety Correction Factor[2]
≤ 1	2.5
1.1 to 4.0	3.5
4.1 to 8.0	4.5
8.1 to 16.0	6.5
16.1 or greater	8.5

Source: Wisconsin Department of Natural Resources. 2004. Conservation Practice Standards. Site Evaluation for Stormwater Infiltration (1002). Madison, WI.

Notes:

1. Ratio is determined by dividing the geometric mean measured infiltration rate at the proposed bottom elevation of the BMP by the geometric mean measured infiltration rate of the least permeable soil horizon within 1.5 metres below the proposed bottom elevation of the BMP.
2. The design infiltration rate is calculated by dividing the geometric mean measured infiltration rate at the proposed bottom elevation of the BMP by the safety correction factor.

The design infiltration rate should be used to determine the maximum depth of the water storage component of the BMP, based on the desired drawdown period (typically 48 hours to fully drain the BMP; see Chapter 4 for guidance regarding the design of specific infiltration BMP types). Based on the calculated design infiltration rate, assumptions regarding the bottom elevation of the BMP may need to be reconsidered and further infiltration testing may be warranted.

Appendix D GEOTECHNICAL INVESTIGATION

D.1 EXCERPTS FROM HYDROGEOLOGICAL ASSESSMENT AND TERRAIN ANALYSIS

Stantec

Geotechnical Investigation Report

New Church
3856, 3866 and 3876 Navan Road
Ottawa, ON

Prepared for:
St. George and St. Anthony Church
1081 Cadboro Road
Ottawa, ON
K1J 7T8

Prepared by:
Stantec Consulting Ltd.
1331 Clyde Avenue
Ottawa, ON
K2C 3G4

Project No. 160410200

December 10, 2018

Table of Contents

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by St. George and St. Anthony Church to carry out a geotechnical investigation for a new church to be constructed in the City of Ottawa, Ontario. The geotechnical investigation was completed in order to determine the subsurface conditions at the site and to provide geotechnical recommendations and design parameters.

This report presents the results of the field investigation program and laboratory testing, as well as geotechnical design recommendations. Limitations associated with this report and its contents are provided in the Statement of General Conditions included in Appendix A.

2.0 SITE AND PROJECT DESCRIPTIONS

The site is located on 3856, 3866 and 3876 Navan Road in Ottawa, Ontario as shown on Drawing No. 1 in Appendix B. The property is a vacant, undeveloped land. Based on a topographical plan of the site prepared by Annis, O'Sullivan, Vollebekk Ltd. dated September 2016, the ground surface at the site is generally flat with existing ground surface elevations varying between about 85.2 m and 85.8 m.

It is noted that beyond the south edge of the property is a treed slope that is approximately 15 m in height extending down towards the Prescott-Russell recreational trail and the Mer Bleue Bog located approximately 700 m away from the site. The slope appears to be locally as steep as 4H:1V.

Based on the information provided by Eternal Engineering Corp, Stantec understands that the proposed development will consist of a one-storey church building with an approximate plan area of 846 m^2, a one-storey service building with an approximate plan area of 1,585 m^2, paved parking areas and driving lanes. The proposed buildings will not include underground levels.

3.0 GEOLOGY

Available geological maps indicate that the surficial geology at the site is anticipated to consist of surficial sand deposit underlain by a thick deposit of Champlain Sea clay over shale bedrock of Billings formation. Nearby borehole and water well records suggest that the clay may extend to depths of about 25 to 30 m below ground surface.

The Champlain Sea clays are typically highly compressible and can undergo large settlements when subjected to new loads associated with site grade fills or foundations. In accordance with the Interactive Vs30 Google Map for the City of Ottawa, the site is located in a seismic class E zone.

4.0 INVESTIGATION METHODS

4.1 BOREHOLE INVESTIGATION

Prior to commencing the field investigation, Stantec arranged for utility clearances to be completed by a private utility locating contractor, USL-1.

A geotechnical field investigation consisting of advancing six (6) boreholes, designated as BH18-1 to BH18-6, was carried out on February 2 to 6, 2018. The approximate borehole locations are shown on Drawing No. 2.

The boreholes were drilled using a track-mounted drill rig equipped with 200 mm diameter, hollow-stem augers and rock coring capabilities that was supplied and operated by George Downing Estate Drilling Ltd.

The subsurface stratigraphy encountered in each borehole was recorded in the field by Stantec field personnel. Soil samples were recovered at regular intervals using a 50-mm (outside diameter) split-tube sampler by conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in ASTM specification D1586. In-situ shear vane measurements were carried out at selected depths within the cohesive soil deposit. A series of Shelby Tube samples were collected within the clayey soils. Dynamic Cone Penetration Testing (DCPT) was completed in BH18-4, BH18-5 and BH18-6 to confirm the inferred depth to bedrock. Coring was carried out in BH18-6 to confirm the type and engineering characteristics of the bedrock.

All soil samples recovered from the boreholes were placed in moisture-proof bags. Soil and bedrock samples collected during the investigation were returned to Stantec's Ottawa laboratory for detailed classification and testing.

A standpipe piezometer was installed in BH18-5 to facilitate the measurement of the groundwater level at the site. The remaining boreholes were backfilled with drill cuttings mixed with bentonite.

Borehole location information is presented on the Borehole Records in Appendix C and summarized in Table 4.1 below.

Table 4.1: Summary of Borehole Details

Borehole No.	Approximate UTM Coordinates (Zone 18T)		Approximate Ground Elevation (m)
	Northing	Easting	
BH18-1	5030115	462323	85.3
BH18-2	5030037	462357	85.4
BH18-3	5030054	462401	85.5
BH18-4	5030054	462360	85.3
BH18-5	5030105	462398	85.8
BH18-6	5030087	462377	85.6

4.2 LABORATORY TESTING

The following geotechnical laboratory testing was performed on selected samples:

- Moisture contents;
- Grain size distribution/hydrometer analyses; and
- One oedometer (consolidation) test.

The results of the laboratory tests are discussed in the text of this report and are provided on the Borehole Records and Bedrock Core Log in Appendix C. Figures illustrating the results of the grain size distribution tests, and Atterberg Limits tests are included in Appendix D.

Chemical analyses related to parameters associated with the potential for corrosion or sulphate attack (i.e. pH, resistivity, and chloride and sulphate content) were completed on two (2) samples by Paracel Laboratories Inc.

Samples remaining after testing will be stored for a period of three (3) months after issuance of the final report. Samples will then be discarded after this period unless otherwise directed.

5.0 SUBSURFACE CONDITIONS

5.1 GENERAL

Detailed descriptions of the subsurface soil and bedrock conditions are presented on the Borehole Records, Bedrock Core Log, and Rock Core Photographs provided in Appendix C. Documents providing explanations of the symbols and terms used on the borehole records are also provided in Appendix C. Laboratory test results are presented in Appendix D as well as on the borehole records.

The stratigraphic boundaries on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units. The borehole records depict conditions at the particular locations and at the particular times indicated. The subsurface soil and groundwater conditions between boreholes and/or at locations away from the borehole locations will vary from those indicated on the borehole records.

It is noted that information provided in the following sections is intended to summarize the conditions encountered; however, the borehole records provided in Appendix C should be used as the primary source of the subsurface information for the site.

A summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

5.2 OVERBURDEN

In general, the subsurface stratigraphy encountered at the site consists of a surficial layer of topsoil followed by silty sand that is underlain by a thick Champlain Sea clay deposit followed by shale bedrock.

5.2.1 Topsoil

The thickness of the topsoil was measured to be approximately 100 to 190 mm at the surface of all borehole locations. The topsoil was typically comprised of silty clay that was black/grey in colour.

5.2.2 Silty Sand

A layer of silty sand was encountered beneath the topsoil and extended to depths of approximately 3 to 4 m below ground surface. Standard Penetration Test (SPT) penetration resistances of 3 to 14 per 0.3 m of penetration were measured within this layer indicating these materials are in a loose to compact state.

Laboratory testing conducted on samples of the silty sand measured natural moisture contents of between 24 and 36%, expressed as a percentage of the dry weight of the soil.

Grain size distribution tests were completed on four (4) samples of the silty sand. The results of the tests are presented on Figure D1 in Appendix D and summarized in Table 5.1.

Table 5.1: Grain Size Distribution – Silty Sand (SM)

Borehole	Sample	Depth (m)	Description	% Gravel	% Sand	% Silt and Clay
BH18-1	SS2	1.1	SILTY SAND (SM)	0	86.4	13.6
BH18-1	SS3	1.8	SILTY SAND (SM)	2.1	77.1	20.7
BH18-2	SS3	1.8	POORLY GRADED SAND with silt (SP-SM)	0	89	11
BH18-5	SS3	1.8	SILTY SAND (SM)	0	86.1	13.9

In accordance with the Unified Soil Classification System, the sample tested can be generally classified as SILTY SAND (SM).

5.2.3 Champlain Sea Clay

The silty sand deposit was underlain by a deposit of sensitive to extra-sensitive Champlain Sea clay. This deposit extended to depths of approximately 28 to 29 m below ground surface.

In-situ vane shear tests conducted on the Champlain Sea clay measured undrained shear strength values of about 27 to 60 kPa. The estimated sensitivity values of the Champlain Sea clay are presented on Figure B2 in Appendix B. The sensitivity of the clay ranged from 4 to 14 and the clay is classified as sensitive to extra-sensitive in accordance with the errata to the 4th (2006) Edition of the Canadian Foundation Engineering Manual (CFEM).

Laboratory testing conducted on samples of the Champlain Sea clay measured natural moisture contents of between 51 and 84%.

The results of grain size distribution tests completed on two (2) samples of the Champlain Sea clay are displayed on Figure D2 in Appendix D and are summarized in Table 5.2 below.

Table 5.2: Grain Size Distribution – Champlain Sea Clay

Borehole	Sample	Depth (m)	% Gravel	% Sand	% Silt	% Clay
BH18-4	SS7	5.6	0	16	46	38
BH18-6	ST8	7.9	0	1	69	30

The results of Atterberg limits testing carried out on representative samples of this material are summarized in Table 5.3 below. The results of this testing are also shown on the Borehole Records included in Appendix C and on Figure D3 in Appendix D, indicate that the Champlain Sea clay samples tested can be classified as Clay of high plasticity (CH).

In addition, the calculated Liquidity Index for the Champlain Sea clay samples were 1.1 and 1.3 as presented in Table 5.3 below. A liquidity index greater than about 1 corresponds to sensitive to extra sensitive clay and values of approximately 1.4 or greater are indicative of quick clay conditions.

Table 5.3: Atterberg Limits Test Results – Champlain Sea Clay (CH)

Borehole	Sample	Depth (m)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI = (LL-PL)	Liquidity Index, LI LI = (Wn-PL)/(LL-PL)
BH18-4	SS7	5.6	67	23	44	1.1
BH18-6	ST8	7.9	63	22	41	1.3

The results of a consolidation test carried out on a Shelby tube sample collected from a depth of 7.9 m at Borehole BH18-6 are summarized in Table 5.4 below. The results of the consolidation testing are also presented graphically in Appendix D.

Table 5.4: Summary of Consolidation Test Results - Champlain Sea Clay (CH)

Borehole No/Sample No	Sample Depth/Elevation (m)	Moisture Content (%)	Initial Void Ratio	Specific Gravity, G_s	C_r	C_c	P'_c (kPa)
BH18-6/ST 8	7.9/77.7	77	2.2	2.8	0.03	2.2	98

where:

C_r = Recompression Index.
C_c = Compression Index
P'_c = Estimated Preconsolidation Pressure

5.3 BEDROCK

Bedrock was encountered in Borehole BH18-6 at a depth of 27.8 m (corresponding to elevation of 57.8m). DCPT refusal was encountered on inferred bedrock at depths of 28.5 and 29 m in BH18-4 and BH18-5, respectively (corresponding to an elevation of 56.8 m). The bedrock core obtained from the borehole consisted predominantly of good quality, black, slightly weathered shale. A detailed description of the rock core is provided on the Bedrock Core Log in Appendix C. Rock core photographs are also provided in Appendix C.

A compressive strength test conducted on one (1) rock core sample collected from a depth of about 28 m in BH18-6 showed that the compressive strength of the sample tested was 35 MPa. The test result indicates that the bedrock is medium strong.

5.4 GROUNDWATER CONDITIONS

A groundwater monitoring well, with a screen from 3.4 m to 6.4 m below ground surface, was installed in BH18-5. The groundwater level in this well was recorded at approximately 0.2 m below ground surface on February 27th, 2018 (corresponding to an approximate elevation of 85.6 m). An additional measurement, taken on July 4, 2018, encountered water at a depth of 1.8 m below grade (approximate elevation of 84.0 m). Groundwater levels at the site will be subject to fluctuations due to seasonal changes and precipitation events.

5.5 CHEMICAL ANALYSIS

Chemical testing was completed on selected soil samples from BH18-5 and BH18-6. Table 5.5 below summarizes the results.

Table 5.5: Summary of Chemical Testing Results

Borehole No.	Sample No./Depth	Physical Characteristics				
		% Solids (by Wt.)	pH	Resistivity (Ohm-m)	Chloride (ug/g)	Sulphate (ug/g)
BH18-5	SS4/2.6m	79.2	7.6	49.6	109	27
BH18-6	SS1/1.1m	80.6	7.2	150	9	29

6.0 DISCUSSION AND RECOMMENDATIONS

The geotechnical investigation was carried out based on an early proposed site layout. It is understood that due to the geotechnical and site grading constraints, the site plan has been modified resulting in portions of the proposed buildings falling outside of the area enclosed by the boreholes. The recommendations provided in this report include a light weight fill design option which is applicable to the proposed new layout locations, however, prior to construction, additional boreholes will be required in order to confirm that no other issues are present and to provide a final confirmation of the slope stability analysis for the natural slope immediately south of the property and within approximately 7 m of the south building limit beyond which the ground slopes down for an approximate height of 15 m towards the Prescott-Russell recreation trail and the Mer Bleue Bog.

Although it is not anticipated that soil conditions will differ significantly from the conditions encountered at the boreholes, it is recommended that prior to construction at least a total of three additional boreholes be drilled and sampled within the final building footprints and near the natural slope.

6.1 KEY GEOTECHNICAL ISSUES

Key geotechnical issues that require consideration for this project include the following:

- The site includes a 3 m to 4 m silty sand cap that generally in a loose to compact state and would be liquefiable under the standard applicable earthquake design loading. Therefore, as part of the site preparation works, the sands at the site will need to be densified.
- All topsoil and/or organic soils should be removed.
- The site is underlain by an approximately 25 m thick, compressible deposit of Champlain Sea clay. The clay deposit has a firm consistency and has a limited capacity to support new loads (e.g. from site grade fill placement, foundation and floor loads and/or potential groundwater level lowering, etc.).
- The in-situ and laboratory shear vane test results suggest that the Champlain Sea clay deposit is highly sensitive to strength loss when disturbed. In addition, laboratory test results indicate the natural moisture content of the clay is higher than the measured liquid limit. Therefore, the clay can behave like a fluid when excavated and/or disturbed. This material is not considered suitable for re-use and could require specialized handling procedures (e.g. drying) prior to transport off-site.
- Grade raises of more than 0.6 m in depth shall be achieved using light weight fill (i.e., Styrofoam blocks) within the building footprints and shall be extended to at least 6 m away from the building footprints.
- As discussed before, the sand will have to be densified before the installation of the shallow foundations. Otherwise, piled foundations driven to the bedrock surface may be used.
- The Champlain Sea clay deposit is typically expected to be highly frost susceptible. It is typically prone to large amounts of heaving for the first few years; magnitudes of over 150 mm should be expected. It is generally not recommended to cut significantly within this type of soil unless large frost heave movements can be tolerated or unless insulation is applied below pavement structures. In this regard, the site is covered with 3 to 4 m of sand, and therefore, it is not anticipated that the clay will be exposed to freezing conditions.
- The Champlain Sea clay is typically sensitive to settlement from the water demand from trees. The selection and planting of trees should follow the City of Ottawa guidelines for tree planting in sensitive marine clay. The overgrowth of tree roots, as well as the phenomenon of tree root removing moisture from surrounding soils, may modify the soils properties. Therefore, species of tree whose characteristics are known to match these concerns should not be proposed in the landscape areas. In general, the planting of trees should be offset from foundations by a distance equal to at least the theoretical mature tree height.

The following sections incorporate the above-mentioned key geotechnical issues.

6.2 GRADE RAISE RESTRICTION

The site is underlain by a highly compressible Champlain Sea clay deposit that is approximately 25 m thick. The results of a consolidation test carried out on a sample of the clay indicates that the material has a pre-consolidation pressure of approximately 98 kPa at an approximate depth of 7.9 m, which is slightly higher than current loading conditions due to self-weight of the sand and clay and considering a design anticipated ground water level of being at elevation 83.0 m.

Large consolidation settlements may occur when the application of new loads such as site grade fills and building loads result in final loads exceeding the maximum past loading conditions (i.e. the preconsolidation pressure) of the Champlain Sea clays.

Calculation of the potential settlement of the compressible clay beneath this site due to the placement of the proposed site grade fill materials was performed using the computer program Settle 3D (Rocscience, 2009) which is a three-dimensional program for the analysis of the consolidation/vertical settlement of soil under surface loads. The geotechnical design parameters used for the Settle 3D analysis are summarized on Figure B1 in Appendix B.

The results of the settlement analyses indicate that the placement of granular fill to raise the site grades 0.6 m would result in settlements of approximately 25 mm. A maximum grade raise restriction of 0.6 m is, therefore, recommended for the development due to the compressible soils encountered at the site.

Grade raises of more than 0.6 m in depth shall be achieved using light weight fill (i.e., Styrofoam blocks) within the building footprints and shall be extended to at least 6 m away from the building footprints.

6.3 FROST PENETRATION

The frost penetration depth for foundation design at this site is 1.8 m.

It is noted that the above frost penetration depth is applicable only to foundation design. Short period deeper frost penetrations, which would have little impacts on foundations, may occur. The typical soil cover for water mains is 2.4 m below ground surface in the City of Ottawa.

6.4 SITE PREPARATION

An approximately 100 to 190 mm thick layer of topsoil containing organic matters was encountered at the surface of the boreholes. Prior to carrying out soil densification works, all existing surficial topsoil, vegetation and/or other deleterious materials (e.g. any loose, wet, and/or otherwise disturbed native materials) should be completely removed from within the footprint of the new development.

As discussed in Section 6.10.1 (Seismic Design Considerations), the 3 to 4 m thick silty sand layer which overlies the deep clay layer is in a loose to compact state and has the potential of liquefying under the earthquake design conditions defined by the Ontario Building Code. Therefore, as part of the site preparation works, soil densification will be required and should incorporate the following:

- Densification methods, such as dynamic compaction or rapid impact methods should be considered.
- Construction vibrations should be monitored during the construction works to meet the vibration constraints at nearby buildings that are imposed by the City of Ottawa.
- The in-situ densification program should be designed to achieve an in-situ density corresponding to an SLS bearing pressure of 150 kPa for the sand. The in-situ densification should be designed to achieve a minimum factor of safety against liquefaction of 1 throughout the densified pad.
- The densified sand should extend to 6 m beyond the limits of the proposed buildings.

The prepared subgrade soils will require inspection by geotechnical personnel prior to structural fill placement to verify all unsuitable material has been removed.

The site grades should then be raised/reinstated using Structural Fill consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type I or II materials that are placed in lifts no thicker than 300 mm and compacted to at least 100% of the material's Standard Proctor Maximum Dry Density (SPMDD). The final layer of fill should consist of OPSS Granular A materials with a minimum thickness of 300 mm beneath the floor slabs and 200 mm in other areas.

The placement of all engineered fill materials should be monitored on a full-time basis by qualified and experienced geotechnical personnel under the supervision of a geotechnical engineer, with the authority to stop the placement of fill at any time when conditions are considered to be unacceptable.

All fill materials imported to the site must meet all applicable municipal, provincial, and federal guidelines and requirements associated with environmental characterization of the materials.

The contractor should be responsible for protecting the subgrade soils from disturbance due to construction traffic. This may require that construction access routes are temporarily overbuilt (i.e. provided with increased granular fill) and/or geotextiles are provided between the granular fill and the subgrade surface.

Imported fill materials should be tested and approved by a geotechnical engineering firm prior to delivery/use. Monitoring of fill placement and in situ compaction testing should be carried out to confirm that all fill is placed and compacted to the required degree.

6.5 SLOPE STABILITY CONSIDERATION

The current proposed building footprint is located within 7 m of the south limit of the property beyond which the ground surface slopes down about 15 m towards the recreational Prescott-Russell recreational trail and the Mer Bleue Bog. Based on a cursory review of the slope, it appears to be relatively gentle with slopes ranging from 7H:1V to 10H:1V, with local portions possibly as steep as 4H:1V. As a result of the current proposed building location, it is recommended that the following be incorporated in the final geotechnical investigation phase:

- Confirmation of the nearby slope geometry by either an elevation survey or by a lidar survey.
- Geotechnical soil stability analysis incorporating the nearby slope geometry and future borehole information to be obtained near the slope.

Generally, the surface drainage within the site should be collected and directed towards a storm water management system. Drainage should not be directed towards the sloping ground to the south in order to avoid surface erosion of any natural clay slopes. Surface erosion in some circumstances can destabilize stable clay slopes.

6.6 TEMPORARY EXCAVATIONS

All temporary excavations should be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. Care should be taken to direct surface water away from open excavations.

It is anticipated that shallow open cut excavations extending to depths of less than 2 to 3 m below existing ground surface. The potential for instability of excavations extending to greater depths should be reviewed by a geotechnical engineer.

Based on the boreholes advanced within the site, excavations within upper 3 m to 4 m of existing site grades are expected to be within the silty sand deposit. This material would be classified as Type 3 soils, as defined by the Occupational Health and Safety Act and Regulations for Construction Projects.

Provided that appropriate groundwater control is provided to maintain the water level below the base of the excavation, OHSA indicates that temporary excavations made within Type 3 soils should be developed with side slopes no steeper than 1H:1V.

Steeper side slopes would require shoring to meet the requirements of the OHSA. All shoring systems should be designed and approved by a qualified Professional Engineer.

The stability of the wall of the excavation may be affected by surcharge loads, stockpiles as well as groundwater seepage conditions. Therefore, soils excavated from the trenches and/or construction materials should not be stockpiled adjacent to excavations.

The base of excavations should not be exposed for extended periods of time.

6.7 DEWATERING

Groundwater inflows into small and shallow excavations of less than 2.0 to 3.0 m deep developed within the silty sand deposit that extends slightly below the water table could be handled by pumping from filtered sumps within the excavation areas.

More significant groundwater inflows should be expected for deeper excavations. Therefore, more extensive dewatering systems could be required for such conditions requiring Ministry of the Environment and Climate Change (MOECC) permitting.

6.8 REUSE OF ON-SITE MATERIALS

The surficial topsoil materials are unsuitable for reuse in any application except for general landscaping purposes.

The native silty sand soils are not considered to be suitable for reuse as engineered/structural fill below or adjacent to new foundations. These materials that are free of organic matter and other deleterious materials, may be considered suitable for reuse as trench backfill (outside of foundation areas) or as general site grade fill (i.e. materials used to raise the site grade to the design elevations outside building footprints).

The ability to compact these materials to required levels is dependent on the moisture content of the materials; thus, the amount of re-useable material will be dependent on the natural moisture content, weather conditions and the construction techniques at the time of excavation and placement. Although not expected for this site, any boulders or cobbles with dimensions greater than 150 mm should be removed from these materials prior to placement.

The Champlain Sea clay soils encountered at depths of approximately 3 to 4 m are not considered to be suitable for re-use due to the high natural water content(s) of these materials. As indicated previously, this clay material has natural water contents that are above their Liquid Limits. These materials can behave like a fluid once excavated/disturbed and could require drying of the soil prior to transport.

6.9 PIPE BEDDING AND BACKFILL

OPSS Granular A materials should be placed below sewer and water pipes as bedding material. The bedding should have a minimum thickness of 150 mm or more to meet City of Ottawa standards. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to thicken the bedding layer or provide a sub-bedding layer of compacted Granular B Type II materials. Pipe backfill and cover materials should also consist of OPSS Granular A material. A minimum of 300 mm vertical and side cover should be provided. These materials should be compacted to at least 95% of the material's SPMDD in lifts no greater than 300 mm. Clear crushed stone backfill should not be permitted as pipe bedding materials.

Where the pipe trenches will be covered with hard surfaced areas, the type of native material placed in the frost zone (i.e. between subgrade level and 1.8 meters depth or the top of the pipe cover materials) should match the soil exposed on the trench walls for frost heave compatibility.

Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 98 percent of the material's standard Proctor maximum dry density using suitable compaction equipment.

If there is insufficient reusable material at the site, any bulk fill required to raise the site grades should consist of imported granular fill meeting the requirements of OPSS Select Subgrade Material (SSM).

All imported fill materials should be tested and approved by a geotechnical engineering firm prior to delivery to the site.

6.10 FOUNDATION DESIGN

6.10.1 Seismic Design Considerations

The seismic Site Class value, as defined in Section 4.1.8.4 of the 2012 Ontario Building Code (OBC), contains a seismic analysis and design methodology which uses a seismic site response and site classification system defined by the shear stiffness of the upper 30 metres of the ground below the foundation level. There are six site classes (from A to F), decreasing in stiffness from A (hard rock) to E (soft soil); Site Class F denotes problematic soils for which a site-specific evaluation is required.

Based on the results of the field investigation conducted at the site, a Site Class designation of E is considered appropriate. If a higher site classification is required, a site-specific shear wave velocity test could be carried out to see if more favourable results are possible. However, given the depth to bedrock at this site, it should be anticipated that the results of further testing could leave the site class unmodified.

A copy of the NBC Seismic Hazard Calculation Data sheet is provided in Appendix F.

The potential liquefaction of the site soils under seismic loading conditions was assessed using the analysis methodology suggested by Idriss and Boulanger (2008). The evaluation was completed based on the SPT resistance values from the boreholes and based on the following:

- A Site Adjusted PGA of 0.349g.
- An earthquake magnitude, M_w of 6.47.

The assessment indicates that the Silty Sand soils extended to depths of approximately 3 to 4 m are considered susceptible to liquefaction as presented on Figure F1 in Appendix F. As a result of liquefaction, earthquake-induced settlements in the order of 40 mm to 70 mm should be anticipated. Soil strata susceptible to liquefaction will also undergo loss of strength, stiffness and support capacity. Soil improvement techniques should be implemented to improve the Silty Sand condition (e.g., dynamic compaction). The dynamic compaction program should be designed and implemented so that a factor of safety against liquefaction of 1 or more within the proposed building footprints is achieved.

6.10.2 Shallow Footings

The buildings could be supported on shallow footings bearing on the native silty sand deposit encountered above the clay deposit at the site. The following will need to be incorporated in the design:

- The sand is under a loose to compact condition and was determined to be liquefiable. Therefore, the sand will have to be densified before the installation of the footing foundations. This can be achieved using dynamic compaction techniques or rapid impact compaction methods. As stated earlier, the soil densification program should be designed and implemented so that a factor of safety against liquefaction of 1 or more within the proposed building footprints is achieved.
- Light weight fill materials (Styrofoam Blocks) shall be used to achieve the proposed grade raises of more than 0.6 m in depth within the building footprint and should be extended to at least 6 m away from the building perimeters.
- Subdrains shall be installed around the perimeters of the building and the Styrofoam blocks.

Assuming that the sand is treated as discussed above, shallow footing foundations may be considered for the proposed buildings and should be installed at a depth of 1.8 m below the final grade. Resistances at Ultimate Limits States (ULS) and at Serviceability Limits States (SLS) for new square and strip footings have been calculated and are provided in Table 6.1 below.

Table 6.1: Geotechnical Resistance for Shallow Footings

Footing Width (m)	Minimum Footing Embedment (m) Below Final Ground Surface	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Square Footings			
1	1.8	380	150
2		410	115
3		345	60
4		260	45
Strip Footings			
0.8	1.8	260	150
1		270	110
1.5		295	75
2		250	60

Notes:

The geotechnical resistances in the above table are provided for the range of footing widths and the minimum footing embedments listed in the above table. Additional input should be provided by the geotechnical engineer if the foundation sizes or depths are outside of the ranges outlined above.

The factored geotechnical bearing resistance at ULS incorporates a resistance factor of 0.5. The post-construction total settlements of footings sized using the above SLS bearing pressure should be less than about 25 mm,.

The subgrade surfaces beneath all footings must be inspected by qualified geotechnical personnel prior to placing concrete in order to confirm the above design pressures and to ensure there are no disturbances or deleterious materials at the bearing surface.

The unfactored horizontal resistance to sliding of the spread foundations may be calculated using the following unfactored coefficients of friction:

0.45	between native silty sand and cast-in-place concrete
0.55	between OPSS Granular A or B Type II materials and cast-in-place concrete

In accordance with Table 8.1 of the Canadian Foundation Engineering Manual 4th Edition (CFEM), a resistance factor (ϕ_{gu}) against sliding (for frictional materials) of 0.8 should be applied to obtain the resistance at ULS.

6.10.3 Piled Foundations

Deep foundation systems are considered technically feasible for the proposed development at this site. The buildings could be supported on deep foundations transferring the foundation loads to below the compressible Champlain Sea clay layer (i.e., down to the bedrock surface).

A suitable pile type would be concrete filled steel pipe piles (driven closed-ended) or H-piles, with the piles end-bearing on bedrock. For this site, the piles should be driven to practical refusal on the bedrock surface which appears to be at 28 m to 29 m below the existing ground surface. The piles should attain refusal at the surface of the weathered bedrock; it is likely that some limited penetration of the piles into the bedrock may occur.

For piles attaining refusal at or slightly below the bedrock surface, settlement at the toe will be negligible and the total pile head settlement will correspond to the elastic deformation of the piles.

The ultimate limit states (ULS) axial geotechnical resistance in compression of piles driven to refusal on bedrock (or slightly within) at this site should be considered to be the structural capacity of the pile.

Due to stresses imposed by the pile driving methods and to avoid damaging the steel during driving, it is recommended that the ULS geotechnical resistance be limited to 141 N/mm^2 of the steel cross-sectional area of the piles. In the case where pipe piles are to be filled with concrete and the pile driving contractor proposes higher capacities to incorporate the structural benefits of the concrete, the contractor would be required to demonstrate that the piles have achieved the proposed higher capacities by field testing.

Based on a limiting stress value of 141 N/mm^2 against steel cross-sectional area, the following ULS geotechnical resistances may be considered.

HP 310x110	1988 kN at ULS
Pipe 324 mm diameter, 11 mm thick wall	1540 kN at ULS

The actual piles selected will depend on the pile load requirements and the pile cap configurations.

Given that the potentially liquefiable layer is shallow and thin, drag loads normally associated with liquefaction is not required to be incorporated in the pile design.

It is anticipated that piles will be spaced more than three diameters apart and that pile groups will contain relative few piles. Therefore, group effects requiring reduction in pile capacities or resulting in significant ground heaving around the piles are not anticipated.

As discussed elsewhere in this report, measures (i.e. a grade raise restriction of 0.6 m) are to be undertaken at the site to prevent soil consolidation within the footprint of the proposed building. Therefore, it has been assumed that drag loads due to soil settlements may not be considered in the design.

For piles driven to bedrock, the geotechnical resistance at serviceability limit state (SLS) exceeds the ULS value and therefore is considered to be not applicable to the design.

The pile driving contractor should be required to submit the following information prior to mobilizing to the site.

- Outline of proposed pile driving equipment
- Pile driving refusal criteria to provide the ULS design value selected for the project

Pile caps/grade beams for unheated areas such as exterior structures should be provided with 1.8 m of soil cover.

10% of the driven piles should be subjected to dynamic pile testing to confirm that they are well seated on bedrock and that the pile driving strategy did damage the piles upon reaching bedrock. Dynamic testing should be carried out using a pile driving analyser (PDA).

6.10.4 Floor Slab

The recommendations provided herein are based on the assumption that the average net slab loads will not exceed 12 kPa. Should a greater average load be proposed, Stantec should review the recommendations presented herein.

The subgrade beneath the floor slab should be prepared in accordance with the recommendations included in Section 6.4. Generally, the floor slab should be supported by a minimum of 300 mm of OPSS Granular A material that is compacted to at least 100% of the material's Standard Proctor Maximum Dry Density (SPMDD).

Non-structural slab-on-grade units should 'float' independently of all load-bearing walls and columns and sawcut control joints should be provided at regular intervals and along column lines to minimize shrinkage cracking and to allow for normal differential settlement of the floor slabs.

A modulus of subgrade reaction of 30 MPa/m may be used for the floor structural design.

6.11 PARKING AREAS

Provided that subgrade preparation below pavements will comply with the requirements outlined in Section 6.4 of this report, the pavement structure provided in Table 6.2 below may be used for design.

Table 6.2: Recommended Pavement Structure

Location	Asphalt Thickness	Base Thickness OPSS Granular A (mm)	Subbase Thickness Granular B Type II (mm)
Standard Duty Parking Areas	60 mm SP12.5 mm	150	300
Heavy Duty Parking	40 mm SP12.5 mm 50 mm SP SP19.0 mm	150	400

Notes:

- The finished sub-grade surface must be compacted to achieve a minimum of 95% of the materials SPMDD immediately prior to placement of the granular materials.
- Asphalt performance grade PG 58-34 should be specified.
- The Superpave mix designs should use a Traffic Category of B.
- The compaction of the asphalt layers should be to at least 92% Maximum Theoretical Relative Density (MTRD) in accordance with OPSS 310.
- All granular materials should be in accordance with the requirements of OPSS Specification. These materials should be compacted to at least 100% of the material's Standard Proctor maximum dry density (SPMDD) in lifts no greater than 300 mm.
- A tack coat is recommended between asphalt layers and along the edges of any cuts in asphalt.
- In the event that the asphalt layer is not placed at the same time as the granular sub-base/base and the base is left exposed for a period of time, the top layer of granular material should be re-shaped, surface compacted and replaced with a fresh layer of Granular A prior to the placement of the asphalt surface.
- Control of surface water is a critical factor in achieving good performance over the pavement structure life. In this regard, the elevations of the surface of the parking areas should be designed to promote adequate surface drainage.

6.12 COLD WEATHER CONSTRUCTION

Placement of fill materials in cold weather requires a considerable increase in effort from that required in "better" weather conditions. Additional costs are typically incurred as a result, and general productivity can be expected to suffer. In addition to the prevailing weather conditions, the quantity of fill to be placed, the required lateral extent and thickness, the equipment used for placement and compaction, and the protection methods employed by the contractor, will all have an influence on the success of placing fill in adverse weather conditions.

Notwithstanding the comments provided in the previous sections of this report pertaining to backfilling and engineered fill, when construction is undertaken during periods of inclement weather or when freezing conditions exist, the placement of fill materials for any purpose should consider the comments provided below.

- Foundations shall be constructed on non-frozen ground only; where non-frozen ground includes the material at surface and all underlying soils. The non-frozen nature of the ground must be confirmed by a geotechnical inspection within 1 hour of concrete placement.

- Following construction of foundations, protection measures must be provided to prevent freezing of the foundation subgrade/bearing soils and for protection of the concrete during curing. The protective measures must also keep the subgrade soils beneath the foundations from freezing after the concrete has cured.
- Foundations shall be backfilled with free-draining granular material and drainage shall be provided to prevent lifting of the foundations due to adfreeze during the construction period.
- Structural fill shall not be placed on frozen ground and the structural fill materials shall be free of snow and frozen material.
- Overnight frost penetration into the existing sub-grade or the structural fill must be prevented. Alternatively, the frozen fill must be completely removed prior to placing subsequent lifts. Breaking the frost in-situ is not considered acceptable.
- Moisture adjustment of the fill materials (i.e. adding water or allowing fill to dry) is not practical in freezing conditions. Therefore, obtaining the required compaction levels of 98 percent of the materials Standard Proctor maximum dry density for Structural Fill will not be practical if the fill materials are not supplied to the site near their optimum water content for compaction.
- Regular checks of the temperature of the fill should be made. The soil temperature should be greater than +2C to allow for compaction to the specified degree.
- Imported fill should not be stockpiled on site in such a condition where freezing of the material in the stockpile can develop. Direct import, placement, and compaction is recommended.
- Full-time inspection and testing services is required during earthworks in winter conditions.

6.13 CEMENT TYPE AND CORROSION POTENTIAL

Two (2) tests were conducted on selected soil samples to determine the water soluble sulphate content of the site soils. The sulphate concentrations in the samples were 27 and 29 ug/g. Results of the sulphate analysis are shown in Table 5.5. The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected on concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The test results provided in Table 5.5 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects. The soil pH results were 7.2 and 7.6 which is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. The reported resistivity of 49.6 and 150 (ohm-m) suggests a low corrosive environment.

7.0 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of the St. George and St. Anthony Church, who is identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying of unexpected site conditions
- Planning, design or construction

This report has been prepared by Ramy Saadeldin, Ph.D., P.Eng. and reviewed by Raymond Haché, M.Sc., P.Eng., ing.

Respectfully submitted,

STANTEC CONSULTING LTD.

LICENSED PROFESSIONAL ENGINEER
R. SAADELDIN
100506710
2018/12/10
PROVINCE OF ONTARIO

Ramy Saadeldin, Ph.D., P.Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
17713504
2018/12/10
PROVINCE OF ONTARIO

Raymond Haché, M.Sc., P.Eng., ing.
Senior Principal, Geotechnical Engineering

Appendix E DRAWINGS