Appendix A WATER SUPPLY SERVICING

A.1 DOMESTIC WATER DEMAND ESTIMATE

29 Robinson Avenue - Domestic Water Demand Estimates
- Based on Rubin and Rotman Architectural Site Plan, September 1, 2018 (160401428)

Building ID	Units	Population	Daily Rate of Demand [1]	Avg Day Demand [2]		Max Day Demand [2]		Peak Hour Demand [2]	
				(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Residential	51	72.1	350	17.5	0.292	43.8	0.730	96.4	1.606
Total Site :	**51**			**17.5**	**0.29**	**43.8**	**0.73**	**96.4**	**1.61**

1 Average day water demand for residential areas equal to 350 L/cap/d

2 City of Ottawa water demand criteria used to estimate peak demand rates for residential areas are as follows:
maximum day demand rate = 2.5 x average day demand rate
peak hour demand rate = 2.2 x maximum day demand rate

A.2 FIRE FLOW REQUIREMENTS PER FUS

FUS Fire Flow Calculation Sheet

Stantec Project #: 160401428
Project Name: FUS Protocol Test Drive
Date: 10/26/2018
Fire Flow Calculation #: 1
Description: 29 Robinson Avenue

Notes: Floors 3 and 4

<table>
<tr><th>Step</th><th>Task</th><th colspan="6">Notes</th><th>Value Used</th><th>Req'd Fire Flow (L/min)</th></tr>
<tr><td>1</td><td>Determine Type of Construction</td><td colspan="6">Wood Frame</td><td>1.5</td><td>-</td></tr>
<tr><td rowspan="2">2</td><td>Determine Ground Floor Area of One Unit</td><td colspan="6">-</td><td>595</td><td>-</td></tr>
<tr><td>Determine Number of Adjoining Units</td><td colspan="6">Includes adjacent wood frame structures separated by 3m or less</td><td>1</td><td>-</td></tr>
<tr><td>3</td><td>Determine Height in Storeys</td><td colspan="6">Does not include floors >50% below grade or open attic space</td><td>2</td><td>-</td></tr>
<tr><td>4</td><td>Determine Required Fire Flow</td><td colspan="6">(F = 220 x C x $A^{1/2}$). Round to nearest 1000 L/min</td><td>-</td><td>11000</td></tr>
<tr><td>5</td><td>Determine Occupancy Charge</td><td colspan="6">Limited Combustible</td><td>-15%</td><td>9350</td></tr>
<tr><td rowspan="4">6</td><td rowspan="4">Determine Sprinkler Reduction</td><td colspan="6">Conforms to NFPA 13</td><td>-30%</td><td rowspan="4">0</td></tr>
<tr><td colspan="6">Standard Water Supply</td><td>-10%</td></tr>
<tr><td colspan="6">Not Fully Supervised or N/A</td><td>0%</td></tr>
<tr><td colspan="6">% Coverage of Sprinkler System</td><td>0%</td></tr>
<tr><td rowspan="5">7</td><td rowspan="5">Determine Increase for Exposures (Max. 75%)</td><td>Direction</td><td>Exposure Distance (m)</td><td>Exposed Length (m)</td><td>Exposed Height (Stories)</td><td>Length-Height Factor (m x stories)</td><td>Construction of Adjacent Wall</td><td>-</td><td>-</td></tr>
<tr><td>North</td><td>> 45</td><td>27</td><td>0</td><td>0-30</td><td>Wood Frame or Non-Combustible</td><td>0%</td><td rowspan="4">3834</td></tr>
<tr><td>East</td><td>3.1 to 10</td><td>21</td><td>2</td><td>31-60</td><td>Ordinary or Fire-Resistive with Unprotected Openings</td><td>16%</td></tr>
<tr><td>South</td><td>20.1 to 30</td><td>27</td><td>2</td><td>31-60</td><td>Ordinary or Fire-Resistive with Unprotected Openings</td><td>7%</td></tr>
<tr><td>West</td><td>3.1 to 10</td><td>20</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>18%</td></tr>
<tr><td rowspan="4">8</td><td rowspan="4">Determine Final Required Fire Flow</td><td colspan="7">Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min</td><td>13000</td></tr>
<tr><td colspan="7">Total Required Fire Flow in L/s</td><td>216.7</td></tr>
<tr><td colspan="7">Required Duration of Fire Flow (hrs)</td><td>2.50</td></tr>
<tr><td colspan="7">Required Volume of Fire Flow (m^3)</td><td>1950</td></tr>
</table>

A.3 BOUNDARY CONDITIONS

Odam, Cameron

From: Wessel, Shawn <shawn.wessel@ottawa.ca>
Sent: Friday, June 22, 2018 8:52 AM
To: Odam, Cameron
Cc: Kilborn, Kris; Deiaco, Simon
Subject: FW: Boundary Conditions Request - 27-31 Robinson Avenue Project
Attachments: 27-31 Robinson June 2018.pdf

Good morning Mr. Odam.

Please find requested boundary conditions below.

******The following information may be passed on to the consultant, but do NOT forward this e-mail directly.******

The following are boundary conditions, HGL, for hydraulic analysis at 27-31 Robinson (zone 1W) assumed to be connected to the 203mm on Robinson (see attached PDF for location).

Minimum HGL = 105.2m

Maximum HGL = 114.7m

Available Flow @ 20psi = 220 L/s assuming a ground elevation of 59.6m

These are for current conditions and are based on computer model simulation.

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.

If you require additional information or clarification, please do not hesitate to contact me anytime.

Thank you

Regards,

Shawn Wessel, A.Sc.T.,rcji
Project Manager - Infrastructure Approvals
Gestionnaire de projet – Approbation des demandes d’infrastructures

Development Review Central Branch | Direction de l’examen des projets d’aménagement, Centrale
Planning, Infrastructure and Economic Development Department | Direction générale de la planification

de l’infrastructure et du développement économique
City of Ottawa | Ville d'Ottawa
110 Laurier Ave. W. | 110, avenue Laurier Ouest, Ottawa ON K1P 1J1
(613) 580 2424 Ext. | Poste 33017
shawn.wessel@ottawa.ca

From: Odam, Cameron <Cameron.Odam@stantec.com>
Sent: Tuesday, June 19, 2018 12:11 PM
To: Wessel, Shawn <shawn.wessel@ottawa.ca>
Cc: Deiaco, Simon <Simon.Deiaco@ottawa.ca>; Kilborn, Kris <kris.kilborn@stantec.com>
Subject: Boundary Conditions Request - 27-31 Robinson Avenue Project

Hi Shawn,

We are working with TC United on the 27-31 Robinson Avenue Project and looking for the watermain hydraulic boundary conditions for the proposed site located at 27-31 Robinson Avenue. The site consists of a proposed 3 storey residential apartment building that is to take up lots 27, 29 and 31. The water servicing will connect to the existing 200mm watermain on Robinson Avenue adjacent to the site.

Attached are the FUS calculations for the proposed building.

Estimated domestic demands and fire flow requirements for the site are as follows:
Average Day Demand – 0.28L/s
Max Day Demand - 0.71L/s
Peak Hour Demand - 1.6L/s

Fire Flow Requirement per FUS – 266.7L/s

Thanks,

Cameron Odam

Direct: +16137244353
Fax: +16137222799
Cameron.Odam@stantec.com

Stantec
400 - 1331 Clyde Avenue
Ottawa ON K2C 3G4 CA


Boundary Conditions for 21-31 Robinson
N
ROBINSON AVE
LEES AVE
RD
203mm
203mm
152mm
152mm
102mm
Legend
Pipe Ownership
Private
Public

Boundary Condition Request for 27-31 Robinson Avenue

Information Provided:
Date provided: June 2018

Scenario	Demand	
	L/min	L/s
Average Daily Demand	17.0	0.28
Maximum Daily Demand	42.5	0.71
Peak Hour	93.6	1.6
Fire Flow Demand	16 000	266.7

Location: 27-31 Robinson Avenue

Appendix B WASTEWATER SERVICING

B.1 SANITARY SEWER DESIGN SHEET

SUBDIVISION:

29 ROBINSON AVE

DATE: 10/24/2018
REVISION: 1
DESIGNED BY: TR
CHECKED BY: -

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401428

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 l/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 l/ha/day	MANNINGS n	0.013
PEAKING FACTOR (ICI >20%):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / BACHELOR	1.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 1 BEDROOM	1.4	INFILTRATION	0.33 l/s/Ha	HARMON CORRECTION FACTOR	0.8
PERSONS / 2 BEDROOM	2.1				

LOCATION			RESIDENTIAL AREA AND POPULATION								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	BACHELOR	1 BEDROOM	2 BEDROOM	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)
BLDG	BLDG	TEE	0.114	37	13	1	72	0.11	72	4.00	0.93

AREA ID NUMBER	COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I
	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)
BLDG	0.000	0.000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

AREA ID NUMBER	INFILTRATION			TOTAL	PIPE								
	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
BLDG	0.114	0.11	0.04	0.97	15.0	150	PVC	DR 28	1.00	15.3	**6.34%**	0.86	0.41

Appendix C STORMWATER MANAGEMENT

C.1 STORM SEWER DESIGN SHEET

Stantec	29 Robinson Avenue	
DATE:	24-Oct-2018	
REVISION:	1	
DESIGNED BY:	TR	FILE NUMBER: 1604-01428
CHECKED BY:		

STORM SEWER DESIGN SHEET (City of Ottawa)

DESIGN PARAMETERS

$I = a / (t+b)^c$ (As per City of Ottawa Guidelines, 2012)

	1:2 yr	1:100 yr			
a =	732.951	1735.688	MANNING'S n =	0.013	BEDDING CLASS = B
b =	6.199	6.014	MINIMUM COVER:	2.00 m	
c =	0.810	0.820	TIME OF ENTRY	10 min	

LOCATION			DRAINAGE AREA																PIPE SELECTION											
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (ROOF) (ha)	C (-)	ACCUM. AREA (2YR) (ha)	A x C (2-YEAR) (ha)	ACCUM. AxC (2YR) (ha)	ACCUM. AREA (100YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)	T of C (min)	$I_{5\text{-}YEAR}$ (mm/h)	$I_{10\text{-}YEAR}$ (mm/h)	$Q_{CONTROL}$ (NOTE 1) (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)	LENGTH (m)	PIPE WIDTH OR DIAMETEI (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE %	Q_{CAP} (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	VEL. (ACT) (m/s)	TIME OF FLOW (min)
SITE	MH 100	MAIN	0.11	0.00	0.00	0.79	0.11	0.084	0.084	0.00	0.000	0.000	10.00	76.81	178.56	0.0	0.0	17.8	11.8	200	200	CIRCULAR	PVC	-	1.00	33.3	**53.51%**	1.05	0.91	0.22
													10.22							225	225									

C.2 RATIONAL METHOD CALCULATIONS

Roof Drain Design Calculation Sheet

File No: **160401405**
Project: **29 Robinson Avenue**
Date: **24-Oct-18**

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**
Uncontrolled - Tributary	UNC-2	Hard	0.001		0.9	0.000		
		Soft	0.005		0.2	0.001		
	Subtotal			0.005			0.00135	0.270
Uncontrolled - Tributary	UNC-1	Hard	0.002		0.9	0.002		
		Soft	0.000		0.2	0.000		
	Subtotal			0.002			0.00207	0.90
Roof	BLDG	Hard	0.060		0.9	0.054		
		Soft	0.000		0.2	0.000		
	Subtotal			0.060			0.05409	0.90
Controlled - Tributary	MH 100, CB 201	Hard	0.029		0.9	0.02632		
		Soft	0.017		0.2	0.00341		
	Subtotal			0.046			0.029729	0.64
Total				**0.114**			**0.087**	
Overall Runoff Coefficient= C:								**0.77**

Total Roof Areas	**0.060 ha**
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.054 ha**
Total Tributary Area to Outlet	**0.114 ha**
Total Uncontrolled Areas (Non-Tributary)	**0.000 ha**
Total Site	**0.114 ha**

Roof Drain Design Calculation Sheet

Project #160401405, 29 Robinson Avenue
Modified Rational Method Calculatons for Storage

2 yr Intensity
City of Ottawa

$I = a/(t + b)^c$

a =	732.951
b =	6.199
c =	0.81

t (min)	I (mm/hr)
5	103.57
10	76.81
15	61.77
20	52.03
25	45.17
30	40.04
35	36.06
40	32.86
45	30.24
50	28.04
55	26.17
60	24.56

2 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 0.114
C: 0.40

Typical Time of Concentration

tc (min)	I (2 yr) (mm/hr)	Qtarget (L/s)
10	76.81	**9.69**

2 YEAR Modified Rational Method for Entire Site

Subdrainage Area: UNC-2 — Uncontrolled - Tributary
Area (ha): 0.01
C: 0.27

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	103.57	0.39	0.39		
10	76.81	0.29	0.29		
15	61.77	0.23	0.23		
20	52.03	0.20	0.20		
25	45.17	0.17	0.17		
30	40.04	0.15	0.15		
35	36.06	0.14	0.14		
40	32.86	0.12	0.12		
45	30.24	0.11	0.11		
50	28.04	0.11	0.11		
55	26.17	0.10	0.10		
60	24.56	0.09	0.09		

Subdrainage Area: UNC-1 — Uncontrolled - Tributary
Area (ha): 0.002
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	103.57	0.60	0.60		
10	76.81	0.44	0.44		
15	61.77	0.36	0.36		
20	52.03	0.30	0.30		
25	45.17	0.26	0.26		
30	40.04	0.23	0.23		
35	36.06	0.21	0.21		
40	32.86	0.19	0.19		
45	30.24	0.17	0.17		
50	28.04	0.16	0.16		
55	26.17	0.15	0.15		
60	24.56	0.14	0.14		

Subdrainage Area: BLDG — Roof
Area (ha): 0.060 — Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.81	11.55	2.04	9.51	5.70	91.3	0.00
20	52.03	7.82	2.17	5.66	6.79	97.9	0.00
30	40.04	6.02	2.18	3.84	6.91	98.6	0.00
40	32.86	4.94	2.16	2.79	6.68	97.3	0.00
50	28.04	4.22	2.11	2.10	6.31	95.0	0.00
60	24.56	3.69	2.06	1.63	5.87	92.3	0.00
70	21.91	3.30	2.01	1.29	5.40	89.5	0.00
80	19.83	2.98	1.96	1.03	4.93	86.6	0.00
90	18.14	2.73	1.90	0.83	4.46	83.8	0.00
100	16.75	2.52	1.85	0.67	4.01	81.1	0.00
110	15.57	2.34	1.80	0.54	3.57	78.4	0.00
120	14.56	2.19	1.75	0.44	3.15	75.9	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level	98.6	0.10	2.18	6.91	24.08	OK

Subdrainage Area: MH 100, CB 201 — Controlled - Tributary
Area (ha): 0.046
C: 0.64

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	8.39	3.32	5.07	3.04
20	52.03	6.47	3.57	2.90	3.48
30	40.04	5.49	3.57	1.93	3.47
40	32.86	4.87	3.49	1.39	3.33
50	28.04	4.43	3.38	1.05	3.15
60	24.56	4.09	3.27	0.82	2.95
70	21.91	3.82	3.16	0.66	2.77
80	19.83	3.59	3.05	0.54	2.59
90	18.14	3.40	2.95	0.45	2.42
100	16.75	3.23	2.86	0.38	2.27
110	15.57	3.09	2.76	0.32	2.12
120	14.56	2.96	2.68	0.28	1.99

Storage: : Above CB

Orifice Size:	LMF 95	
Invert Elevation	57.32	m
Obvert Elevation:	57.52	m
Max Storage Depth	0.20	m
Downstream W/L	57.20	m

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	57.52	0.20	3.57	3.48	12.45	OK

Project #160401405, 29 Robinson Avenue
Modified Rational Method Calculatons for Storage

100 yr Intensity
City of Ottawa

$I = a/(t + b)^c$

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): 0.0410
C: 0.40

	Qtarget	
2-Year Pre Development Discharge	9.69	L/s
Less Peak Sanitary Discharge of	0.97	L/s
Target Release Rate	**8.72**	**L/s**

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: UNC-2 — Uncontrolled - Tributary
Area (ha): 0.01
C: 0.34

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	0.84	0.84		
20	119.95	0.56	0.56		
30	91.87	0.43	0.43		
40	75.15	0.35	0.35		
50	63.95	0.30	0.30		
60	55.89	0.26	0.26		
70	49.79	0.23	0.23		
80	44.99	0.21	0.21		
90	41.11	0.19	0.19		
100	37.90	0.18	0.18		
110	35.20	0.17	0.17		
120	32.89	0.15	0.15		

Subdrainage Area: UNC-1 — Uncontrolled - Tributary
Area (ha): 0.002
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	1.14	1.14		
20	119.95	0.77	0.77		
30	91.87	0.59	0.59		
40	75.15	0.48	0.48		
50	63.95	0.41	0.41		
60	55.89	0.36	0.36		
70	49.79	0.32	0.32		
80	44.99	0.29	0.29		
90	41.11	0.26	0.26		
100	37.90	0.24	0.24		
110	35.20	0.23	0.23		
120	32.89	0.21	0.21		

Subdrainage Area: BLDG — Roof
Area (ha): 0.060 — 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	29.83	2.79	27.05	16.23	130.6	0.00
20	119.95	20.04	2.99	17.06	20.47	141.1	0.00
30	91.87	15.35	3.06	12.29	22.12	145.2	0.00
40	75.15	12.56	3.09	9.46	22.71	146.6	0.00
50	63.95	10.69	3.09	7.59	22.78	146.8	0.00
60	55.89	9.34	3.08	6.26	22.52	146.2	0.00
70	49.79	8.32	3.06	5.26	22.08	145.1	0.00
80	44.99	7.52	3.03	4.48	21.51	143.7	0.00
90	41.11	6.87	3.00	3.86	20.87	142.1	0.00
100	37.90	6.33	2.97	3.36	20.17	140.4	0.00
110	35.20	5.88	2.94	2.94	19.43	138.5	0.00
120	32.89	5.50	2.90	2.59	18.67	136.7	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	146.8	0.15	3.09	22.78	24.08	OK

Subdrainage Area: MH 100, CB 201 — Controlled - Tributary
Area (ha): 0.046
C: 0.80

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	21.23	5.92	15.32	9.19
20	119.95	15.38	6.45	8.93	10.71
30	91.87	12.55	6.52	6.04	10.87
40	75.15	10.85	6.42	4.43	10.63
50	63.95	9.70	6.28	3.42	10.26
60	55.89	8.86	6.13	2.73	9.82
70	49.79	8.20	5.98	2.23	9.37
80	44.99	7.68	5.83	1.86	8.91
90	41.11	7.25	5.68	1.57	8.47
100	37.90	6.89	5.55	1.34	8.05
110	35.20	6.57	5.42	1.16	7.65
120	32.89	6.30	5.29	1.01	7.27

Storage: Surface Storage Above CB

Orifice Size:	LMF 95		Volume in MHCB300 and MH100 when head = 0.75	0.66
Invert Elevation	57.32	m		
Obvert Elevation:	57.52	m	Max available volume in MH's	1.59
Max Storage Depth	0.67	m		
Downstream W/L	57.20	m		

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	57.99	0.67	6.52	10.87	12.45	OK
					1.59	

Roof Drain Design Calculation Sheet

Project #160401405, 29 Robinson Avenue
Modified Rational Method Calculatons for Storage

SUMMARY TO OUTLET

		Vrequired	Vavailable*	
Tributary Area	0.106 ha			
Total 2yr Flow to Sewer	3.6 L/s	3.48	12.45 m^3	Ok
Non-Tributary Area	0.007 ha			
Total 2yr Flow Uncontrolled	1.0 L/s			
Total Area	0.114 ha			
Total 2yr Flow	4.6 L/s			
Target	8.7 L/s			

Project #160401405, 29 Robinson Avenue
Modified Rational Method Calculatons for Storage

SUMMARY TO OUTLET

		Vrequired	Vavailable*
Tributary Area	0.106 ha		
Total 100yr Flow to Sewer	6.5 L/s	10.87	12.45 m^3
Non-Tributary Area	0.007 ha		
Total 100yr Flow Uncontrolled	2.0 L/s		
Total Area	0.114 ha		
Total 100yr Flow	8.4 L/s		
Target	8.7 L/s		

Project #160401405, 29 Robinson Avenue
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	0.0006	0	0.025	13	0	0	0.025
0.050	0.0006	0.0013	1	0.050	54	1	1	0.050
0.075	0.0009	0.0017	3	0.075	120	2	3	0.075
0.100	0.0011	0.0022	7	0.100	214	4	7	0.100
0.125	0.0013	0.0027	14	0.125	334	7	14	0.125
0.150	0.0016	0.0032	24	0.150	482	10	24	0.150

Drawdown Estimate

Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.8	618.5	0.8	0.17179
2.9	1220.8	2.1	0.51092
7.0	1868.0	4.1	1.0298
13.8	2536.2	6.8	1.7343
24.0	3216.0	10.1	2.62763

Rooftop Storage Summary

Total Building Area (sq.m)		602
Assume Available Roof Area (sq	80%	481.6
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		2
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		24
Estimated 100 Year Drawdown Time (h)		2.5

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

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

From Watts Drain Catalogue

Head (m)	L/s Open	**75%**	50%	25%	Closed
0.025	0.3155	**0.31545**	0.31545	0.31545	0.31545
0.050	0.6309	**0.6309**	0.6309	0.6309	0.31545
0.075	0.9464	**0.86749**	0.78863	0.70976	0.31545
0.100	1.2618	**1.10408**	0.94635	0.78863	0.31545
0.125	1.5773	**1.34067**	1.10408	0.86749	0.31545
0.150	1.8927	**1.57726**	1.2618	0.94635	0.31545

Calculation Results

	2yr	100yr	Available
Qresult (cu.m/s)	0.002	0.003	-
Depth (m)	0.099	0.147	0.150
Volume (cu.m)	6.9	22.8	24.1
Draintime (hrs)	1.0	2.5	

Appendix D GEOTECHNICAL INVESTIGATION

Geotechnical Investigation Report
Proposed Residential Development

27-31 Robinson Avenue, Ottawa, ON

Project No.: 121622041

Prepared for:
TC United Group
800 Industrial Ave, Unit 9
Ottawa, ON K1G 4B8

Prepared by:
Stantec Consulting Ltd.
1331 Clyde Avenue
Ottawa, Ontario

27 July 2018

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by TC United Group to carry out a geotechnical investigation for a proposed residential development located on the properties at 27-31 Robinson Avenue in Ottawa, Ontario as shown on the Key Plan (refer to Drawing No. 1).

The geotechnical investigation was completed in order to determine the subsurface conditions at the site and to provide recommendations on the geotechnical design aspects of the project.

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

2.1 SITE AND PROJECT DESCRIPTIONS

The proposed development site contains three properties (27, 29 and 31 Robinson Avenue) that each currently contain an individual residential structure located in the south (front) portion of the property. The existing building at 27 Robinson Avenue contains a basement level while the buildings at 29 and 31 Robinson Avenue contain basements under parts of the structures.

Based on the information provided by TC United Group, Stantec understands that it is planned to combine the lots into a single property and construct a three-storey apartment building with a basement level. The building will encompass a plan area of approximately 630 m^2 and is planned to be constructed at the location shown on Drawing No. 1 in Appendix B. The Final Floor Elevations (FFEs) (tops of slab) are understood to be 58.85 m for the below-grade/basement level and 62.38 m for the first floor.

Based on a topographical plan of the site prepared by Stantec dated April 12, 2018, the ground surface within the site is generally flat with existing ground surface elevations varying between about 59.1 m and 60.5 m.

2.2 GEOLOGY

Available geological maps and previous nearby borehole records indicate that the subsurface conditions at the site consist of glacial till overlying shale bedrock of the Billings formation. Based on available subsurface information in the vicinity of the site including records of boreholes drilled by Stantec on properties on Robinson Avenue located approximately 30 m to the west of the site, the depth to bedrock is anticipated to be approximately 9 m to 10 m below ground surface.

3.0 INVESTIGATION METHODS

3.1 FIELD 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 five (5) boreholes designated as BH18-1, BH18-2, BH18-3A, BH18-3B and BH18-4, was carried out on July 12, 2018. The approximate borehole locations are shown on the Borehole Location Plan (Drawing No. 1) in Appendix B.

The boreholes were drilled using a truck-mounted drill rig equipped with 200 mm diameter, hollow-stem augers with soil sampling 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 a member of Stantec's geotechnical staff.

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.

One of the boreholes (Borehole BH18-3A) encountered effective auger refusal on an inferred boulder at a depth of about 1.9 m below ground surface. Another borehole (BH18-3B) was, therefore, drilled approximately 1 m northwest of Borehole BH18-3A.

Dynamic Cone Penetration Testing (DCPT) was completed in BH18-4 in order to provide information on depth to bedrock. The DCPT encountered refusal at a depth of about 7.5 m below ground surface.

The locations and ground surface elevations at the boreholes were surveyed by Stantec field personnel and referenced to selected objects of known geodetic elevations interpolated from the topographical drawing of the site referenced earlier. The ground surface elevations at the borehole locations should be considered approximate only.

Details on the ground surface elevation and depth of drilling at each borehole are summarized in Table 3.1 below.

Table 3.1: Summary of Borehole Details

Borehole No.	Approximate Ground Elevation (m)	Total Depth Drilled (m)
BH18-1	59.5	8.2
BH18-2	59.4	5.9
BH18-3A	60.1	1.9
BH18-3B	60.1	7.5
BH18-4	60.1	7.5

A monitoring well was installed in Borehole BH18-3B. The monitoring well consisted of a 50 mm diameter PVC pipe with a 3.0 m long slotted pipe section. The well was backfilled with silica sand to approximately 0.3 m above the top of the screen and a bentonite plug was installed above the sand. The monitoring well installation details are provided on the Borehole Record for BH18-3B in Appendix D. All remaining boreholes were backfilled with drill cuttings mixed with bentonite.

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

3.2 LABORATORY TESTING

The following geotechnical laboratory testing was performed on selected samples:

- Moisture contents;
- Grain size distribution/hydrometer analyses; and
- Atterberg Limits.

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 one sample by Paracel Laboratories Inc.

The results of the laboratory tests are discussed in the text of this report and are provided on the Borehole Records in Appendix C. The results of the grain size distribution tests are also included in Appendix D.

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.

4.0 SUBSURFACE CONDITIONS

4.1 GENERAL

Detailed descriptions of the subsurface soil and groundwater conditions are presented on the Borehole Records 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 shown on the borehole records as well as Figures D1 and D2 in Appendix D.

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 will vary between boreholes and/or at locations away from the boreholes.

The 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.

In general, the subsurface stratigraphy encountered at the borehole locations consists of surficial materials including topsoil, asphalt and fill materials underlain by a native glacial till. A summary of the subsurface conditions encountered in the boreholes are provided in the following sections.

4.2 ASPHALT

Asphalt layers, measured to be about 50 mm and 75 mm in thickness, were encountered at the ground surface of Boreholes BH18-1 and BH18-2, respectively.

4.3 TOPSOIL

Topsoil was encountered in Boreholes BH18-3A and BH18-4. The thickness of the topsoil was determined to be approximately 0.2 m in both boreholes.

4.4 FILL

Fill materials of variable composition were encountered in Boreholes BH18-1 and BH18-2 (beneath the asphalt) and in Boreholes BH18-3A and BH18-4 beneath the topsoil.

The fill materials are comprised predominantly of silty sand with varying amounts of gravel and clay and contained rootlets. The fill was measured to extend to depths of approximately 0.5 m to 0.8 m below ground surface corresponding to elevations of approximately 58.7 m to 59.6 m.

Standard Penetration Test (SPT) penetration resistances of 3 to 15 blows per 0.3 m of penetration were measured within the fill materials indicating that these materials are in a very loose to compact state.

Laboratory testing conducted on samples of the fill measured natural moisture contents of between approximately 10% and 22%, expressed as a percentage of the dry weight of the soil.

4.5 GLACIAL TILL

A glacial till deposit was encountered below the fill materials and extended to the bottom of all boreholes. Based on manual/tactile examination of the till, the till consists predominantly of a sandy clay of low plasticity containing gravel. Zones of till comprised of silty sand with gravel were present between 6 m and 7m in Borehole BH18-3B and zones of sandy silt till were also encountered sporadically in the boreholes.

Borehole BH18-3A encountered refusal to auger penetration at a depth of approximately 1.9 m below ground surface on an inferred boulder and cobbles and boulders were inferred during drilling at other borehole locations. The glacial till in Ottawa is typically comprised of cobbles and boulders set in a matrix of finer-grained material (i.e. gravel, sand, silt and clay); larger boulders (e.g. in excess of 1.0 m) are common. The till is typically unsorted and without stratification, but in places contains discontinuous layers or irregular shaped masses of sand and silt. In this regard, where glacial till deposits are identified, cobbles and boulders will be present throughout the deposits and permeable layers of sand and/or silt may also randomly be present due to the unsorted and unstratified nature of the glacial till.

Standard Penetration Test (SPT) 'N' values measured in the glacial till ranged between 5 blows per 0.3 m penetration to spoon refusal of 50 blows per 0.1 m penetration. SPT 'N' values measured in the till within 3 m of ground surface were typically 15 or greater. An in situ shear vane test carried out at a depth of 4.3 m in BH18-3B measured an undrained shear strength of more than 110 kPa and a remoulded shear strength of 25 kPa. A vane test was attempted at a depth of about 4.4 m in BH18-1 but was not be completed as the vane could not be pushed into the soil.

Based on the results of the field testing and manual/tactile examination of the samples, the upper portion of the till typically has a very stiff to hard consistency within 3 m of ground surface and the till becomes stiff to very stiff with localized firm zones at greater depths.

Laboratory testing conducted on samples of the till measured natural moisture contents of between approximately 6 % and 18 %.

Grain size distribution tests were completed on four (4) samples of the glacial till. The results of the tests are included in Appendix D and summarized in Table 4.1.

Table 4.1: Grain Size Distribution Results – TILL

Borehole	Sample	Depth (m)	Unified Soil Classification System	% Gravel	% Sand	% Silt	% Clay
BH18-1	SS6	4.1	SANDY CLAY (CL)	8	28	54	10
BH18-2	SS7	4.9	SANDY SILT (ML)	6	36	53	5
BH18-3B	SS9	6.4	SILTY SAND (SM) with gravel	26	41	25	8
BH18-4	SS3	1.8	SANDY CLAY (CL)	8	41	40	11

In accordance with the Unified Soil Classification System, the samples tested can be typically be classified as sandy clay (CL) to sandy silt (ML). A sample of the more granular till encountered at a depth of about 6 m in BH18-3B can be classified as silty sand with gravel. It is noted that the gradation results do not represent materials larger than the split spoon diameter. Cobbles and boulders were noted throughout the till deposits.

4.6 GROUNDWATER CONDITIONS

Groundwater levels measured within the open boreholes were between 3.8 m and 4.6 m below ground surface upon completion of drilling, corresponding to elevations of 54.8 m to 55.7 m.

A groundwater monitoring well, with a well screen located at a depth of about 4.6 m to 7.6 m below ground surface, was installed in BH18-3B. The groundwater level in this well was recorded to be approximately 4.2 m below ground surface corresponding to an elevation of 55.9 m on July 20, 2018.

Groundwater levels are subject to fluctuations due to seasonal changes and precipitation events. The water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation or snow melt.

4.7 CHEMICAL ANALYSIS

Chemical testing related to the potential for corrosivity and sulphate attack was completed on one a selected soil sample from BH18-4. Table 4.2 below summarizes the test results. The laboratory test report is provided in Appendix E.

Table 4.2: 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)
BH18-4	SS3/1.8m	88.7	7.83	34.3	110	31

5.0 DISCUSSION AND RECOMMENDATIONS

This section provides engineering input related to the geotechnical design aspects of the proposed development based on our interpretation of the available subsurface information described herein and our understanding of the project requirements.

The discussion and recommendations presented in the following sections of this report are intended to provide the designers with preliminary information for planning and design purposes only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

5.1 SEISMIC DESIGN CONSIDERATIONS

5.1.1 Seismic Site Class

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 average 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 current field investigation, it is appropriate to classify the existing ground conditions at the subject site as a Site Class C.

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

5.1.2 Liquefaction Potential

The glacial till deposits that overlie the bedrock at this site consist predominantly of stiff to very stiff clayey soils that are not considered to be susceptible to liquefaction.

5.2 FROST PENETRATION

The design frost penetration depth for the Ottawa area is 1.8 m. All foundations founded on frost-susceptible materials should be provided with a minimum of 1.8 metres of earth cover or equivalent insulation for frost protection purposes.

It is to be 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 frost protection of watermains and services is 2.4 m below ground surface in the City of Ottawa.

Exterior slabs-on-grade or slabs-on-grade within unheated areas will also be subject to the risk of heave and deformation/cracking due to frost. Consideration could be given to the use rigid insulation to protect structures against frost action; however appropriate frost tapers would need to be incorporated at the ends of the insulation.

5.3 SITE PREPARATION

5.3.1 Grade Raise Restrictions

The final site grades in the area of the proposed building are understood to be approximately 59.8 m. The native subsurface materials present at the site consist predominantly of stiff to very stiff, sandy clay till overlying shale bedrock. These materials are not considered to be highly compressible when subjected to light to moderate loads. Therefore, grade raises of less than 1 m, if required, are not anticipated to result in settlements of the underlying soil/bedrock that would adversely affect the performance of the proposed development.

5.3.2 Site Preparation and Floor Slab Construction

In preparation for construction of the building foundations and floor slab, all vegetation and tree stumps/roots, organic soil (including topsoil), existing fill materials, existing infrastructure (e.g. foundations, floor slabs and services for the existing buildings) and any loose, wet, and/or otherwise disturbed native material should be removed from within the footprint of the proposed building and any other settlement sensitive areas. To provide consistent subgrade conditions, all below-grade portions of the existing buildings as well as basement wall backfill materials should be removed to expose the native glacial till.

Following removal of the above noted materials, the prepared subgrade will require inspection by geotechnical personnel to verify all unsuitable material has been removed.

The existing basements and foundations of the existing structures at the site are anticipated to extend below the basement floor slab level for the proposed building. Where removal of existing structures and/or unsuitable materials extends below the floor slab subgrade level, the grade beneath the new building floor slab should be raised/reinstated to the design subgrade level 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 floor slab for the lowest level of the proposed building is understood to be located below the final exterior grades. This level should either be designed to be waterproof/watertight or an underslab drainage system should be provided to prevent hydrostatic pressure build-up beneath the floor due to fluctuations in the water table and/or infiltration of surface water. At least 300 mm of free draining material, such as 16 mm clear crushed stone, should be provided beneath the base of the slab. These materials should be lightly-compacted to provide a level surface and improve trafficability during construction. Subdrains consisting of geotextile encapsulated, 100 mm diameter perforated pipes should be provided at approximately 6 m spacings within the floor slab bedding and should be connected to a frost-free gravity outlet or a sump from which the water is pumped. The requirements for a underslab vapour barrier should be determined in accordance with the requirements of the Ontario Building Code.

If existing fill materials or structures are present beneath the proposed founding elevations, all such fill materials and structures should be removed from beneath the footprint of the building, the footings and the zone of influence of all footings, to expose the native glacial till surface. The zone of influence is defined by a line drawn at 1 horizontal to 1 vertical, outward and downward from the edge of the footings. The grade should be raised back up to the founding level using Structural Fill as discussed above. Inspection and testing services will be critical to ensure that all fill, existing structures and unsuitable materials are removed beneath the proposed building, and that new engineered fill and concrete used is suitable and is placed competently.

5.4 FOUNDATION DESIGN INPUT

Based on the subsurface conditions encountered at the site and the proposed finished floor slab level of the proposed building, the preferred foundation option for this site is the use of shallow strip and/or spread footings bearing on either the undisturbed native till deposits or compacted Structural Fill placed above the undisturbed till.

5.4.1 Foundation Design Parameters - Shallow Footings

Shallow foundations bearing directly on undisturbed native till or on Structural Fill placed above the native till can be designed using factored geotechnical resistance values presented in Table 5.1 below.

Table 5.1: Geotechnical Resistance for Shallow Footings

Footing Type and Width (m)	Minimum Footing Embedment (m) Below Basement Floor Slab Surface	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Square Footings			
1	0.8	275	200
2			180
2.5			150
Strip Footings			
0.5 to 1.5	0.8	200	150

Notes:
The geotechnical resistances in the above table are provided for the range of footing widths and the minimum footing embedment depths (below the basement floor slab surface) listed in the above table.
Additional input should be provided by the geotechnical engineer if the foundation sizes or embedment depths are outside of the ranges outlined above.

The factored geotechnical bearing resistances at ULS incorporate resistance factors of 0.5. The post-construction total and differential settlements of footings sized using the above SLS bearing pressure should be less than about 25 and 20 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction.

The native soils are highly susceptible to disturbance by construction activity especially during wet or freezing weather. Care should be taken to preserve the integrity of the materials as bearing strata. It is essential that the founding level for the footings be inspected by the geotechnical engineer prior to placing concrete. If the concrete for the footings on the native soil cannot be placed immediately after excavation and inspection, it is recommended that a working mat of lean concrete be placed in the excavation to protect the integrity of the bearing stratum.

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

0.55 between OPSS Granular A or B Type II materials and cast-in-place concrete
0.4 between sandy clay till and cast-in-place concrete

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

5.4.2 Foundation Wall Backfill

The soils/fill materials encountered at the site are susceptible to frost heave and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of frost penetration. To avoid problems with frost adhesion and heaving, foundation walls in these areas should be backfilled with non-frost susceptible granular fill meeting the gradation requirements of OPSS Granular B Type I materials. The fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material's SPMDD using suitable vibratory compaction equipment.

In areas where hard surfacing (e.g., concrete slabs, sidewalks) surround the building, differential frost heaving will occur between the granular fill backfill zone and other areas. To reduce this differential heaving, a frost taper of the granular backfill is recommended. The frost taper should extend up from 1.5 metres below finished exterior grade (at the foundation wall) at a slope of 3 horizontal to 1 vertical, or flatter, to the surface level.

Exterior grades should be sloped away from the building to prevent ponding of water around the buildings. As the lowest floor slab level is understood to be below the final exterior grades, the basement wall backfill should be drained using a perimeter drainage system (e.g. perforated subdrain) which is provided positive drainage to storm sewer or to a sump from which water is pumped similar to the underslab drainage system discussed in Section 5.3.2.

5.5 EARTH PRESSURES

Earth pressures will need to be considered in the design of the basement walls. The total active (P_A), passive (P_P) and at-rest (P_O) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$
$$P_P = \frac{1}{2} K_p \gamma H^2$$
$$P_O = \frac{1}{2} K_o \gamma H^2$$

where;
H = height of the wall
γ = unit weight of the backfill soil

Values for K_a, K_p, K_o and γ are provided in the table below. These values are based on the assumption that a horizontal back slope is present behind and adjacent to the wall system. The earth pressure coefficients need to be adjusted (i.e. increased) where sloping backfill will be present behind the walls. At-rest earth pressures should be used in the design of walls that are restrained from movement. The thrust acts at a point one third up the height of the wall.

Table 5.2: Non-Seismic Lateral Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I
Bulk Unit Weight, γ (kN/m^3)	22
Effective Friction Angle	32°
Coefficient of Earth Pressure at Rest (K_o)	0.47
Coefficient of Active Earth Pressure (K_a)	0.31
Coefficient of Passive Earth Pressure (K_p)	3.25

Total active and passive thrusts under earthquake conditions can be calculated using the following equations:

$$P_{AE} = ½\ K_{AE}\ \gamma\ H^2$$

$$P_{PE} = ½\ K_{PE}\ \gamma\ H^2$$

where;

K_{AE} = active earth pressure coefficient (combined static and seismic)
K_{PE} = passive earth pressure coefficient (combined static and seismic)
H = height of wall
γ = total unit weight

The recommended seismic earth pressure parameters are provided in Table 4.3 below. The angle of friction between the soil and the wall has been assumed to be 0° to provide a conservative estimate.

Table 5.3: Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I
Bulk Unit Weight, γ (kN/m^3)	22
Effective Friction Angle	32°
K_{AE} (Non-Yielding Wall)	0.51
Height of Application of P_{AE} from base as a ratio of wall height, (H) – Non Yielding Wall	0.44
Active Earth Pressure (K_{AE}) – Yielding Wall	0.4
Height of Application of P_{AE} from base as a ratio of wall height, (H) - Yielding Wall	0.39
Passive Earth Pressure, (K_{PE})	2.99
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.31

In order to use the coefficients of pressure for the granular materials presented in the tables above, the granular backfill must be provided within a wedge extending out from the base of the wall at 45 degrees (or smaller) to the horizontal.

5.6 EXCAVATIONS AND BACKFILL

5.6.1 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 the open excavations.

The excavation side slopes should be protected from precipitation or surface runoff to prevent further softening that could lead to additional sloughing and caving. If sloughing and/or cave-ins are encountered in the excavation, the slopes should be further flattened to achieve a stable configuration.

Excavations required for the building construction are expected to typically be less than 2 m in depth although localized, deeper excavations could be required (e.g. for service connection tie-ins).

Shallow excavations within the overburden at the site are anticipated to extend through fill materials and the native glacial till deposit. Conventional hydraulic excavating equipment is considered suitable for developing excavations in these materials. recognizing that additional effort will be required to remove cobbles and boulders within the glacial till. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes.

The existing fill materials and the native glacial till deposit that are above the water table would be classified as Type 3 soils as defined by Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. Within Type 3 soils, temporary open cut excavations must be sloped at 1 horizontal to 1 vertical from the base of the excavation per the requirements of OHSA.

The excavation sideslopes would need to be flattened and/or appropriate groundwater control measures implemented if excavations are carried out in overburden materials below the water table.

The excavations must be developed in a manner to ensure that adequate support is provided for any existing structures, utilities or underground services located adjacent to the excavations. Where there is insufficient space to develop open cuts without resultant loss of support for existing features or encroaching into adjacent properties, the installation of a shoring system meeting the requirements of the OHSA would be required. All shoring systems should be designed and approved by a qualified Professional Engineer. The excavation support system should be designed to resist loads from traffic and adjacent building foundations.

5.6.2 Temporary Dewatering Considerations

The groundwater level measured in the piezometer installed in Borehole BH18-03B was measured to be at a depth of about 4.2 m below ground surface.

Control of groundwater into shallow excavations into the glacial till is expected to be able to be handled by filtered sumps within the excavation areas. More significant groundwater inflows should be expected for deeper excavations that extend below the ground water level and penetrate the more granular zones within the till.. More extensive dewatering systems (e.g. external dewatering system using well points or other dewatering wells) could be required for such conditions. Depending on the depth of excavations, dewatering activities may require either registration in the Ministry of the Environment and Climate Change (MOECC) Environmental Activity and Sector Registry (EASR) or obtaining a Permit to Take Water (PTTW) from the MOECC depending on the anticipated groundwater removal rates. If deep excavations extending below the water table are required, a separate hydrogeological assessment, should be completed to confirm these requirements before such excavations are undertaken. This assessment should include measurement of the in situ hydraulic conductivity of the site soils.

5.7 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 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 thickness of 300 mm of these materials should be provided as vertical and side cover beside and over top of the pipes. 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 or cover 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 the top of the pipe cover materials) should match the soil exposed on the trench walls for frost heave compatibility. A 3H:1V frost taper is recommended in order to minimize the effects of differential frost heaving if materials different than those present in excavation sidewalls are used as backfill.

Trench backfill above the pipe cover materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 98 % of the material's SPMDD using suitable vibratory compaction equipment.

The existing fill materials and the native glacial till that are free of organic matter and other deleterious materials, may be considered suitable for reuse as trench backfill or as general site grade fill (i.e. materials used to raise the site grade to the design elevations). The ability to compact these materials to the 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. In addition, any boulders or cobbles with dimensions greater than 150 mm should be removed from these materials prior to placement.

Any imported fill materials proposed for use as bedding or trench backfill should be tested and approved by a geotechnical engineering firm prior to delivery to the site.

Materials testing and inspection should be carried out during construction to ensure the materials meet the project specifications and required level of compaction.

5.8 ADVERSE WEATHER CONSTRUCTION

Additional precautions, effort, and measures may be required, when and where construction is undertaken during late fall, winter, and/or early spring (i.e. when the temperature and climatic conditions can have an adverse influence on the standard construction practices) or during periods of inclement weather. With respect to all earthworks activities undertaken during the late fall through to late spring, when less-than-ideal weather and construction conditions may prevail, the following comments are provided:

1. 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.
2. Similarly, concrete for floor slabs should not be placed or above frozen ground. Test pits or other measures should be undertaken to confirm that the soils beneath the slab(s) are frost-free prior to slab construction.

3. Following construction of footings, protection measures must be provided to prevent freezing of the foundation subgrade/bearing soils and for protection of the concrete during curing.
4. Engineered fills including pipe bedding and cover, are recommended to consist of imported granular materials, including OPSS Granular A or B materials. The use of non-granular fill materials may be considered for use as trench backfill but obtaining suitable compaction of such materials could be extremely problematic, and these materials should only be used if large, post-construction settlement of the trench backfill is deemed acceptable.
5. Fill placement should be inspected by qualified field personnel on a full-time basis 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 unfavourable.
6. Backfill materials, including imported materials, that contain ice, snow, or any frozen material should not be accepted for use.
7. Overnight frost penetration may occur, even in granular fill materials, where precipitation and ground surface runoff pools and accumulates, and freezing temperatures exist. The on-site clayey soils are prone to frost heave due to ice lensing. Any frozen materials should be removed prior to placing subsequent lifts of engineered fill. Breaking the frost in-situ is not considered acceptable.
8. It may be necessary to stop the placement of engineered fill during periods of cold, where ambient temperatures are -5° C or less exist.

Appropriate scheduling of the work may also require specific consideration and revision from that typically adopted. The scope of work intended may have to be reduced or adjusted, and/or only select construction activities be undertaken during specific climatic conditions. The areas of planned fill placement may have to be reduced on a daily basis, and the extent of excavations may have to be limited.

5.9 CEMENT TYPE AND CORROSION POTENTIAL

One (1) test was conducted on a selected soil sample to determine the water soluble sulphate content of the site soils. The sulphate concentration in the sample was 31 ug/g as shown in Table 4.2.

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 for concrete in contact with the soil and groundwater. General Use (GU) Portland cement is appropriate for use at the site.

The test results provided in Table 4.2 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 result of 7.83 is within what is considered the normal range for soil pH of 5.5 to 9.0. The pH level of the tested soil does not indicate a highly corrosive environment. The reported resistivity of 34.3 (ohm-m) suggests a moderate degree of corrosiveness for steel.

6.0 CLOSURE

Not all details related to the proposed development are known at this time. In this regard, all geotechnical comments provided in this report should be reviewed and, if necessary, revised once the final plans become available. Stantec should be retained to review the final drawings and specifications to confirm that the geotechnical input provided herein has been adequately addressed.

This report has been prepared for the sole benefit of TC United Group and their agents, and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and TC United Group. Any use, which a third party makes of this report, is the responsibility of such third party. Use of this report is subject to the Statement of General Conditions provided in Appendix A. It is the responsibility of TC United Group, 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 or unexpected site conditions
- Planning, design or construction

We trust the above information meets with your present requirements. Should you have any questions or require further information, please contact us. This report has been prepared by Ramy Saadeldin, Ph.D., P.Eng. and reviewed by Kevin Nelson, P.Eng.

Thank you for the opportunity to be of service to you.

Respectfully submitted,

STANTEC CONSULTING LTD.

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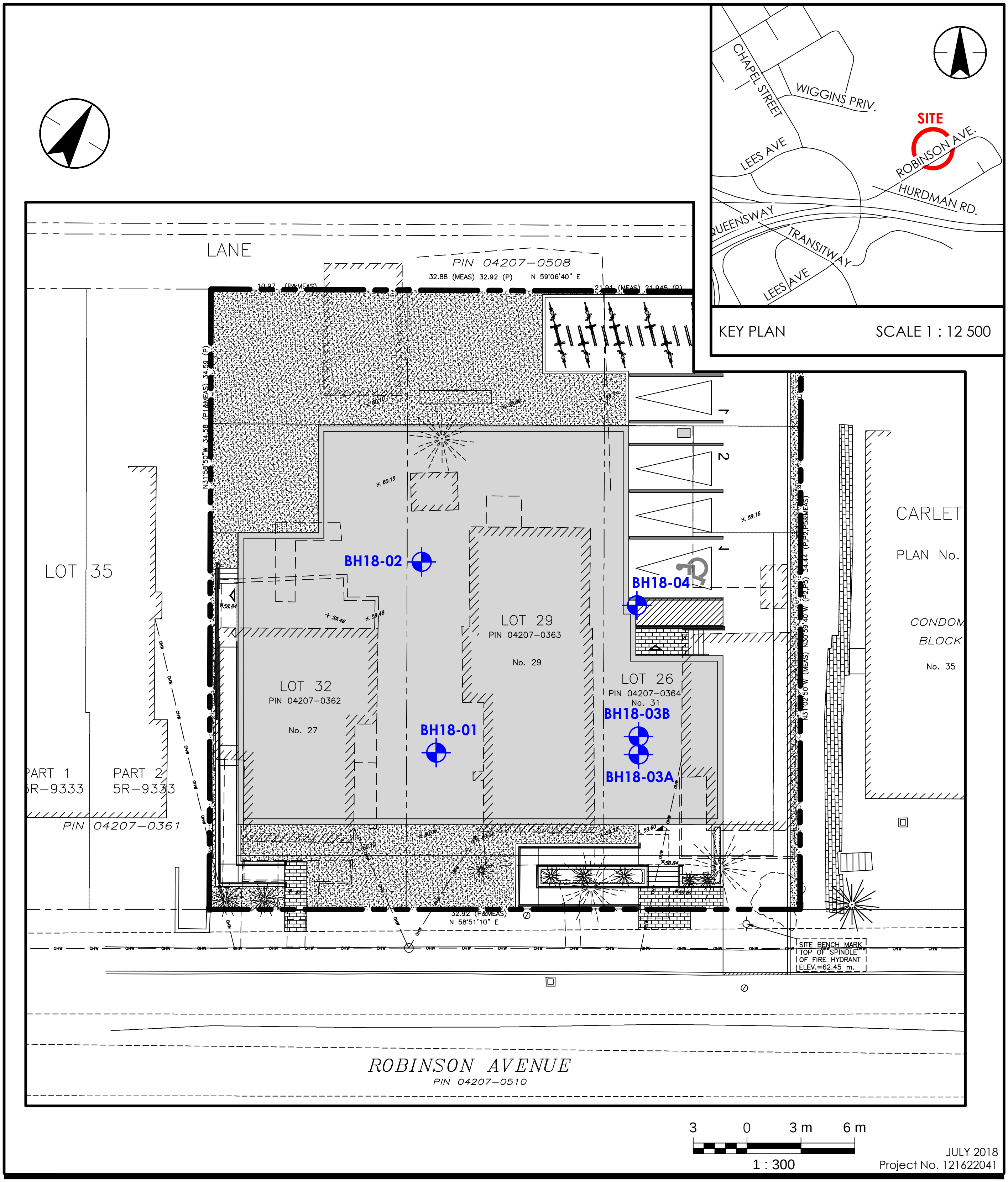

JULY 2018
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LEGEND

PROPERTY BOUNDARY

APPROXIMATE BOREHOLE LOCATION

NOTES

1. COORDINATE SYTEM: NAD 1983 MTM ZONE 9.
2. BASEPLAN PROVIDED BY TC UNITED GROUP (FILENAME: 1821 X - SitePlan.dwg)

Client/Project

TC UNITED GROUP

GEOTECHNICAL INVESTIGATION - PROPOSED BUILDING, 27-31 ROBINSON AVENUE, OTTAWA, ON

Drawing No.

1

Title

BOREHOLE LOCATION PLAN

Appendix E DRAWINGS