Appendix A WATER SUPPLY SERVICING

A.1 DOMESTIC WATER DEMAND ESTIMATE

689 Churchill Avenue - Domestic Water Demand Estimates

Phase 1

Building ID	Area (m2)	Population	Daily Rate of Demand [1]	Avg Day Demand [2]		Max Day Demand [3]		Peak Hour Demand [3]	
				(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Residential 1	203	22	350	5.4	0.09	13.6	0.23	29.9	0.50
Total Site :		**22**		**5.4**	**0.09**	**13.6**	**0.23**	**29.9**	**0.50**

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 for residential

maximum hour demand rate = 2.2 x maximum day demand rate for residential

A.2 FIRE FLOW REQUIREMENTS PER FUS

FUS Fire Flow Calculation Sheet

Stantec Project #: 160401400
Project Name: 689 Churchill Avenue
Date: 3/4/2019
Fire Flow Calculation #: 2
Description: Apartment Building

Notes:

<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>203</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>3</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>8000</td></tr>
<tr><td>5</td><td>Determine Occupancy Charge</td><td colspan="6">Limited Combustible</td><td>-15%</td><td>6800</td></tr>
<tr><td rowspan="4">6</td><td rowspan="4">Determine Sprinkler Reduction</td><td colspan="6">None</td><td>0%</td><td rowspan="4">0</td></tr>
<tr><td colspan="6">Non-Standard Water Supply or N/A</td><td>0%</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>3.1 to 10</td><td>10.7</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>18%</td><td rowspan="4">3536</td></tr>
<tr><td>East</td><td>20.1 to 30</td><td>12.3</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>8%</td></tr>
<tr><td>South</td><td>3.1 to 10</td><td>18.7</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>18%</td></tr>
<tr><td>West</td><td>20.1 to 30</td><td>12.3</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>8%</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>10000</td></tr>
<tr><td colspan="7">Total Required Fire Flow in L/s</td><td>166.7</td></tr>
<tr><td colspan="7">Required Duration of Fire Flow (hrs)</td><td>2.00</td></tr>
<tr><td colspan="7">Required Volume of Fire Flow (m^3)</td><td>1200</td></tr>
</table>

A.3 BOUNDARY CONDITIONS

Kilborn, Kris

From:	Buchanan, Richard <Richard.Buchanan@ottawa.ca>
Sent:	Wednesday, February 21, 2018 5:08 PM
To:	Kilborn, Kris
Subject:	FW: Boundary Conditions Request - 689 Churchill Avenue N
Attachments:	689 Churchill Feb 2018.pdf

Hi Kris

The following are boundary conditions, HGL, for hydraulic analysis at 689 Churchill (zone 1W) assumed to be connected to the 406mm on Churchill (see attached PDF for location).

Minimum HGL = 109.0m

Maximum HGL = 114.6m

Max Day + FireFlow (167 L/s) = 108.8m

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.

From: Kilborn, Kris [mailto:kris.kilborn@stantec.com]
Sent: Thursday, February 15, 2018 2:38 PM
To: Buchanan, Richard <Richard.Buchanan@ottawa.ca>
Cc: Odam, Cameron <Cameron.Odam@stantec.com>
Subject: FW: Boundary Conditions Request - 689 Churchill Avenue N

Hey Richard its Kris again

We have been retained by TC United for civil design for a project located at 689 Churchill Avenue N. Daniel Boulanger from TC United indicated that you attended the pre-consultation meeting for this development.

Would it be possible for you to request the watermain hydraulic boundary conditions for the proposed 689 Churchill Avenue N – site plan. We anticipate the watermain connection to the proposed site plan as shown in the attached figure. This connection is to the 406mm WM along Churchill Avenue N - adjacent to the site.

The intended land use is a 3 storey apartment building consisting of a ground floor with three 2-bedroom units and the second and third floor that both have two 2-bedroom units and 2 1-bedroom units.

Estimated domestic demands and fire flow requirements for the site are as follows:
Average Day Demand - 0.1 L/s

Max Day Demand - 0.18 L/s
Peak Hour Demand - 0.4 L/s
Fire Flow Demand - 167 L/s

The Fire Flow Requirement is based on see the information and calculations in the FUS sheet attached to the email.

Thanks in advance,

Kris Kilborn
Senior Associate, Community Development,
Business Center Sector Leader (BCSL)

Direct: (613) 724-4337
Mobile: (613) 297-0571
Fax: (613) 722-2799

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

Stantec

Boundary Condition for 689 Churchill

Appendix B WASTEWATER SERVICING

B.1 SANITARY SEWER DESIGN SHEET

SUBDIVISION:	689 Churchill Avenue North
DATE:	3/4/2019
REVISION:	2
DESIGNED BY:	WAJ
CHECKED BY:	

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401400

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 (COMM., INST.):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / BACHELOR APT	1.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 2 BED APT	2.1	INFILTRATION	0.33 l/s/Ha		
PERSONS / 3 BED APT	3.1				

LOCATION			RESIDENTIAL AREA AND POPULATION									COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)	
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	UNITS Bachelor	UNITS 2 BED	UNITS 3 BED	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)
BLDG	BLDG	TEE	0.050	13	2	0	22	0.050	22	4.00	0.29	0.00	0.00	0.00	0.00	0.00	0.00

LOCATION	INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	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	0.00	0.00	0.00	0.00	0.00	0.050	0.050	0.02	0.31	13.7	135	PVC	SDR 28	1.00	11.5	**2.66%**	0.80	0.29
											135							

Appendix C STORMWATER MANAGEMENT

C.1 STORM SEWER DESIGN SHEET

Stantec	689 Churchill Avenue North		STORM SEWER DESIGN SHEET (City of Ottawa)
	DATE:	4-Mar-2019	
	REVISION:	3	
	DESIGNED BY:	WAJ	FILE NUMBER: 1604-01400
	CHECKED BY:		

DESIGN PARAMETERS

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

	1:5 yr	1:100 yr				
a =	998.071	1735.688	MANNING'S n =	0.013	BEDDING CLASS =	B
b =	6.053	6.014	MINIMUM COVER:	2.00	m	
c =	0.814	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 (100-YEAR) (ha)	AREA (ROOF) (ha)	C (-)	ACCUM. AREA (5YR) (ha)	A x C (5-YEAR) (ha)	ACCUM. AxC (5YR) (ha)	ACCUM. AREA (100YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)	T of C (min)	$I_{2\text{-YEAR}}$ (mm/h)	$I_{100\text{-YEAR}}$ (mm/h)	$Q_{CONTROL}$ (NOTE 1) (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)	LENGTH (m)	PIPE WIDTH OR DIAMETER (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)
BLDG	BLDG	MAIN	0.05	0.00	0.00	0.69	0.05	0.035	0.03	0.00	0.000	0.000	10.00	104.19	178.56	0.0	0.0	9.99	15.6	200	200	CIRCULAR	PVC	-	1.00	33.3	**29.98%**	1.05	0.77	0.34
													10.34							675	675									

C.2 RATIONAL METHOD CALCULATIONS

Stormwater Management Calculations

File No: **160401400**
Project: **689 Churchill Avenue**
Date: **28-Jan-19**

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	0.019		0.9	0.017		
		Soft	0.000		0.2	0.000		
		Subtotal		0.0191			0.0172	0.90
Controlled - Tributary	CB100	Hard	0.0041		0.9	0.004		
		Soft	0.0199		0.2	0.004		
		Subtotal		0.024			0.0077	0.32
Uncontrolled - Tributary	UNC1	Hard	0.000		0.9	0.000		
		Soft	0.000		0.2	0.000		
		Subtotal		0.006			0.000	0.88
Total				**0.049**			**0.025**	
Overall Runoff Coefficient= C:								**0.51**

Total Roof Areas	**0.019 ha**
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.030 ha**
Total Tributary Area to Outlet	**0.049 ha**
Total Uncontrolled Areas (Non-Tributary)	**0.000 ha**
Total Site	**0.049 ha**

Stormwater Management Calculations

Project #160401400, 689 Churchill Avenue
Modified Rational Method Calculatons 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): 0.0491
C: 0.50

Typical Time of Concentration

tc (min)	I (5 yr) (mm/hr)	Qtarget (L/s)
10	104.19	**7.11**

5 YEAR Modified Rational Method for Entire Site

Subdrainage Area: BLDG — Roof
Area (ha): 0.02 — 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	4.98	1.55	3.43	2.06	96.0	0.00
20	70.25	3.36	1.56	1.79	**2.15**	97.9	0.00
30	53.93	2.58	1.53	1.05	1.88	92.7	0.00
40	44.18	2.11	1.49	0.63	1.50	85.5	0.00
50	37.65	1.80	1.44	0.36	1.09	77.6	0.00
60	32.94	1.57	1.37	0.21	0.74	67.0	0.00
70	29.37	1.40	1.30	0.10	0.44	55.9	0.00
80	26.56	1.27	1.21	0.06	0.26	48.1	0.00
90	24.29	1.16	1.12	0.04	0.23	44.3	0.00
100	22.41	1.07	1.04	0.03	0.20	41.1	0.00
110	20.82	1.00	0.97	0.03	0.17	38.4	0.00
120	19.47	0.93	0.91	0.02	0.14	36.1	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	97.87	0.10	1.56	2.15	7.64	0.00

Subdrainage Area: CB100 — Controlled - Tributary
Area (ha): 0.02
C: 0.32

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.19	3.78	3.76	0.02	**0.01**
20	70.25	3.06	3.76	0.00	0.00
30	53.93	2.68	3.76	0.00	0.00
40	44.18	2.43	3.76	0.00	0.00
50	37.65	2.24	3.76	0.00	0.00
60	32.94	2.07	3.76	0.00	0.00
70	29.37	1.93	3.76	0.00	0.00
80	26.56	1.78	3.76	0.00	0.00
90	24.29	1.64	3.76	0.00	0.00
100	22.41	1.52	3.76	0.00	0.00
110	20.82	1.41	3.76	0.00	0.00
120	19.47	1.33	3.76	0.00	0.00

Storage: Above CB

Orifice Diameter:	LMF 55	mm
Invert Elevation	76.03	m
T/G Elevation	77.96	m
Max Ponding Depth	0.00	m
Downstream W/L	75.26	m

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	77.96	1.93	3.76	0.01	3.17	OK

Subdrainage Area: UNC1 — Uncontrolled - Tributary
Area (ha): 0.01
C: 0.88

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.19	1.53	1.53		
20	70.25	1.03	1.03		
30	53.93	0.79	0.79		
40	44.18	0.65	0.65		
50	37.65	0.55	0.55		
60	32.94	0.48	0.48		
70	29.37	0.43	0.43		
80	26.56	0.39	0.39		
90	24.29	0.36	0.36		
100	22.41	0.33	0.33		
110	20.82	0.31	0.31		
120	19.47	0.29	0.29		

Project #160401400, 689 Churchill Avenue
Modified Rational Method Calculatons for Storage

100 yr Intensity | I = a/(t + b)^c
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 Modified Rational Method for Entire Site

Subdrainage Area: BLDG — Roof
Area (ha): 0.02 — 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	9.48	1.75	7.74	4.64	126.7	0.00
20	119.95	6.37	1.79	4.58	5.50	133.4	0.00
30	91.87	4.88	1.79	3.09	**5.56**	133.8	0.00
40	75.15	3.99	1.78	2.21	5.31	131.9	0.00
50	63.95	3.40	1.76	1.64	4.91	128.8	0.00
60	55.89	2.97	1.74	1.23	4.44	125.1	0.00
70	49.79	2.64	1.70	0.94	3.96	119.6	0.00
80	44.99	2.39	1.67	0.72	3.47	114.0	0.00
90	41.11	2.18	1.63	0.55	2.99	108.4	0.00
100	37.90	2.01	1.59	0.42	2.51	102.8	0.00
110	35.20	1.87	1.55	0.31	2.08	96.4	0.00
120	32.89	1.75	1.51	0.24	1.70	89.3	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	133.82	0.13	1.79	5.56	7.64	0.00

Subdrainage Area: CB100 — Controlled - Tributary
Area (ha): 0.02
C: 0.40

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	6.51	3.76	2.75	**1.65**
20	119.95	4.99	3.76	1.23	1.48
30	91.87	4.24	3.76	0.48	0.87
40	75.15	3.78	3.76	0.02	0.06
50	63.95	3.47	3.47	0.00	0.00
60	55.89	3.23	3.23	0.00	0.00
70	49.79	3.03	3.03	0.00	0.00
80	44.99	2.87	2.87	0.00	0.00
90	41.11	2.73	2.73	0.00	0.00
100	37.90	2.61	2.61	0.00	0.00
110	35.20	2.49	2.49	0.00	0.00
120	32.89	2.39	2.39	0.00	0.00

Storage: Surface Storage Above CB

Orifice Diameter:	LMF 55	mm
Invert Elevation	76.03	m
T/G Elevation	77.96	m
Max Ponding Depth	0.00	m
Downstream W/L	75.26	m

Volume Within CB's and HDPE Subdrain: 3.17
Total Storage: **3.17**

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	77.96	1.93	3.76	1.65	3.17	OK
					1.52	

Subdrainage Area: UNC1 — Uncontrolled - Tributary
Area (ha): 0.01
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	2.98	2.98		
20	119.95	2.00	2.00		
30	91.87	1.53	1.53		
40	75.15	1.25	1.25		
50	63.95	1.07	1.07		
60	55.89	0.93	0.93		
70	49.79	0.83	0.83		
80	44.99	0.75	0.75		
90	41.11	0.69	0.69		
100	37.90	0.63	0.63		
110	35.20	0.59	0.59		
120	32.89	0.55	0.55		

Stormwater Management Calculations

Project #160401400, 689 Churchill Avenue
Modified Rational Method Calculatons for Storage

SUMMARY TO OUTLET						
			Vrequired	Vavailable*		
Tributary Area	0.043	ha				
Total 5yr Flow to Sewer	3.78	L/s	0	0	m^3	Ok
Non-Tributary Area	0.006	ha				
Total 5yr Flow Uncontrolled	1.53	L/s				
Total Area	0.049	ha				
Total 5yr Flow	5.31	L/s				
Target	7.11	L/s				

Project #160401400, 689 Churchill Avenue
Modified Rational Method Calculatons for Storage

SUMMARY TO OUTLET						
			Vrequired	Vavailable*		
Tributary Area	0.043	ha				
Total 100yr Flow to Sewer	3.76	L/s	0	0	m^3	Ok
Non-Tributary Area	0.006	ha				
Total 100yr Flow Uncontrolled	2.98	L/s				
Total Area	0.049	ha				
Total 100yr Flow	6.74	L/s				
Target	7.11	L/s				

Project #160401400, 689 Churchill Avenue
Roof Drain Design Sheet, Area ROOF1
Standard Watts Model R1100 Accuflow 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	4	0	0	0.025
0.050	0.0006	0.0013	0	0.050	17	0	0	0.050
0.075	0.0007	0.0014	1	0.075	38	1	1	0.075
0.100	0.0008	0.0016	2	0.100	68	1	2	0.100
0.125	0.0009	0.0017	4	0.125	106	2	4	0.125
0.150	0.0009	0.0019	8	0.150	153	3	8	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.2	196.2	0.2	0.05451
0.9	473.4	0.7	0.18601
2.2	829.7	1.3	0.41649
4.4	1243.6	2.2	0.76193
7.6	1700.6	3.2	1.23432

Rooftop Storage Summary

Total Building Area (sq.m)		191
Assume Available Roof Area (sq.	80%	152.8
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)		8
Estimated 100 Year Drawdown Time (h)		0.9

* 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/min Open	L/s 75%	Notch Rating 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**	0.3155
0.075	0.9464	0.8675	0.7886	**0.7098**	0.3155
0.100	1.2618	1.1041	0.9464	**0.7886**	0.3155
0.125	1.5773	1.3407	1.1041	**0.8675**	0.3155
0.150	1.8927	1.5773	1.2618	**0.9464**	0.3155

Calculation Results

	5yr	**100yr**	**Available**
Qresult (cu.m/s)	0.002	0.002	-
Depth (m)	0.098	0.134	0.150
Volume (cu.m)	2.2	5.6	7.6
Draintime (hrs)	0.4	0.9	

WATTS	**Adjustable Accutrol Weir** **Tag:** ______	**Adjustable Flow Control for Roof Drains**

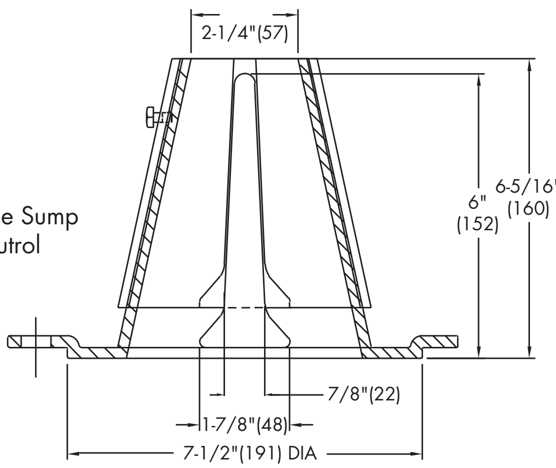
ADJUSTABLE ACCUTROL (for Large Sump Roof Drains only)

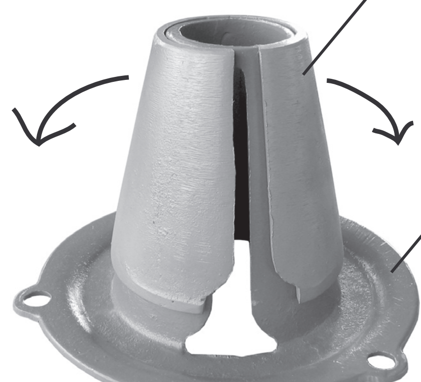
For more flexibility in controlling flow with heads deeper than 2", Watts Drainage offers the Adjustable Accutrol. The Adjustable Accutrol Weir is designed with a single parabolic opening that can be covered to restrict flow above 2" of head to less than 5 gpm per inch, up to 6" of head. To adjust the flow rate for depths over 2" of head, set the slot in the adjustable upper cone according to the flow rate required. Refer to Table 1 below.
Note: Flow rates are directly proportional to the amount of weir opening that is exposed.

EXAMPLE:

For example, if the adjustable upper cone is set to cover 1/2 of the weir opening, flow rates above 2"of head will be restricted to 2-1/2 gpm per inch of head.

Therefore, at 3"of head, the flow rate through the Accutrol Weir that has 1/2 the slot exposed will be:
[5 gpm (per inch of head) x 2 inches of head] + 2-1/2 gpm (for the third inch of head) = 12-1/2 gpm.





1/2 Weir Opening Exposed Shown Above

TABLE 1. Adjustable Accutrol Flow Rate Settings

Weir Opening Exposed	1"	2"	3"	4"	5"	6"
	Flow Rate (gallons per minute)					
Fully Exposed	5	10	15	20	25	30
3/4	5	10	13.75	17.5	21.25	25
1/2	5	10	12.5	15	17.5	20
1/4	5	10	11.25	12.5	13.75	15
Closed	5	5	5	5	5	5

Job Name ______ Contractor ______

Job Location ______ Contractor’s P.O. No. ______

Engineer ______ Representative ______

WATTS
A Watts Water Technologies Company

USA: Tel: (800) 338-2581 • Fax: (828) 248-3929 • Watts.com
Canada: Tel: (905) 332-4090 • Fax: (905) 332-7068 • Watts.ca
Latin America: Tel: (52) 81-1001-8600 • Fax: (52) 81-8000-7091 • Watts.com

Appendix D GEOTECHNICAL INVESTIGATION

Geotechnical Investigation Report
Proposed Residential Development

689 Churchill Ave North, Ottawa, ON

Project No.: 121621808

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

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

April 30, 2018

Table of Contents

LIST OF TABLES

LIST OF APPENDICES

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 property at 689 Churchill Avenue North in Ottawa, Ontario as shown on the Key Plan (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 property currently contains a single, two-storey residential building with a partial basement located on an approximate 525 m^2 lot. In addition to the building, the lot contains three sheds, landscaped areas, a paved driveway and several trees. Based on a topographical plan of the site prepared by Stantec dated April 17, 2018, the ground surface within the site is generally flat with existing ground surface elevations varying between about 77.7 m and 78.3 m.

Stantec understands that it is planned to redevelop the subject property with a new residential building. The proposed development will consist of a 3-storey building with the floor slab of the lowest level being slightly lower than final site grades. Each floor of the building will encompass a plan area of approximately 180 m^2. The Final Floor Elevations (FFEs) (tops of slab) are understood to be 77.1 m for the below-grade/basement level and 80.36 m for the first floor. A small retaining wall (i.e. supporting a retained soil height of approximately 1 m) is required adjacent to a stairwell on the north side of the building immediately adjacent to the north property line.

2.2 GEOLOGY

Available geological maps indicate that the subsurface conditions at the site consist of clay and/or till deposits over limestone bedrock. The depth to bedrock is expected to be within about 3 m below ground surface.

3.0 INVESTIGATION METHODS

3.1 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 two (2) boreholes, designated as BH18-1 and BH18-2, was carried out on April 6, 2018. The approximate borehole locations are shown on the Borehole Location Plan (Drawing No. 2) in Appendix B.

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. Boreholes were advanced in southern portion of the site; other areas of the site could not be accessed due to the existing building and attached deck, sheds, trees and/or powerlines.

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.

Both boreholes encountered effective refusal to auger drilling on bedrock at a depth of about 1.3 m below ground surface. Coring was carried out in BH18-2 to confirm the type and engineering characteristics of the bedrock.

A standpipe piezometer was installed in BH17-2 to facilitate the measurement of the groundwater level at the site. BH17-1 was backfilled with drill cuttings mixed with bentonite.

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.

Borehole location information is presented on the Borehole Records in Appendix C and summarized in Table 3.1 below.

Table 3.1: Summary of Borehole Details

Borehole No.	Approximate UTM Coordinates (Zone 18T)		Approximate Ground Elevation (m)	Total Depth Drilled (m)
	Northing	Easting		
BH18-1	5025726	441449	77.9	1.3
BH18-2	5025726	441458	78	5.1

3.2 LABORATORY TESTING

The following geotechnical laboratory testing was performed on selected samples:

- Moisture contents;
- Grain size distribution/hydrometer analyses on two soil samples; and
- Unconfined compressive strength tests on two bedrock samples.

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 and Bedrock Core Log 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 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 shown on the borehole records as well as Figure D1 in Appendix D.

The stratigraphic boundaries on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil and bedrock 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, bedrock 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 fill materials including asphalt overlying limestone bedrock at a depth of 1.3 m below ground surface. A summary of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2 ASPHALTIC CONCRETE

A surficial asphaltic concrete layer was encountered at ground surface in both BH18-1 and BH18-2. The thickness of the asphalt varied between 55 mm and 70 mm at the borehole locations.

4.3 FILL

Fill materials comprised of silty sand with gravel to clayey sand with gravel were encountered beneath the asphaltic concrete layer in BH18-1 and BH18-2. The fill materials extended to a depth of about 1.3 m below ground surface in both boreholes.

An approximately 100 mm thick layer of buried topsoil was encountered at a depth of 0.6 m below ground surface in BH18-1. The topsoil was underlain by a layer of gravel and/or cobbles that extended to a depth of 0.8 m. Brick pieces were noted within the fill in BH18-2.

Standard Penetration Test (SPT) penetration resistances of 7 and 8 per 0.3 m of penetration were measured within the upper zone of the fill to a depth of 0.6 m indicating that these materials are in a loose state. SPT penetration resistances of 50 blows per 0.1 m of penetration and 50 blows per 0.08 m of penetration were measured within the lower zone of granular fill indicating that these materials are in a very dense state.

Laboratory testing conducted on samples of the fill measured natural moisture contents of between 5 and 17%, expressed as a percentage of the dry weight of the soil.

Grain size distribution tests were completed on two (2) samples of the fill. The results of the tests are plotted on Figure D1 in Appendix D and summarized in Table 4.1.

Table 4.1: Grain Size Distribution Results – FILL

Borehole	Sample	Depth (m)	Unified Soil Classification System	% Gravel	% Sand	% Silt and Clay
BH18-1	SS1	0.07 to 0.6	SILTY SAND (SM) with gravel	22	46.5	31.5
BH18-2	SS2	0.8 to 1.4	CLAYEY SAND with gravel (SC)	26.5	24.5	49

4.4 BEDROCK AND AUGER REFUSAL

The bedrock surface was encountered at a depth of approximately 1.3 m below ground surface corresponding to an elevation of about 76.7 m in BH18-2. The presence of bedrock was confirmed by coring at this location. A detailed description of the rock core retrieved from this borehole is provided on the Bedrock Core Log in Appendix C. Rock core photographs are also provided in Appendix C.

The bedrock core obtained from BH18-2 consisted predominantly of grey, slightly weathered limestone with shale laminations to interbeds. An approximately 70 mm thick zone of fractured rock or void was encountered at a depth of about 3.1 m depth. Rock Quality Designation (RQD) values of between 70 % and 100 % were recorded within the core runs indicating the bedrock is of fair to excellent quality.

Compressive strength tests conducted on two (2) rock core samples collected from depths of about 1.5 m and 2.7 m indicated that the compressive strength of the samples tested were 207 MPa and 148 MPa, respectively. The test results indicate that the bedrock is very strong.

Auger refusal was also encountered at a depth of 1.3 m in BH18-1. As this is the same depth as the bedrock was encountered in BH18-2, the refusal in inferred to be a result of encountering bedrock.

4.5 GROUNDWATER CONDITIONS

No groundwater seepage was noted within the overburden materials during drilling and no accumulation of water was observed within the boreholes immediately after completion of drilling, in BH18-1 and BH18-2.

A groundwater monitoring well, with a well screen located at a depth of 2.1 m to 3.7 m below ground surface, was installed in BH18-2. The groundwater level in this well was recorded at approximately 3.1 m below ground surface on April 13, 2018.

Perched groundwater conditions may develop within and above the fill materials and bedrock. Therefore, it should be noted that groundwater levels at the site will be subject to fluctuations due to seasonal changes and precipitation events.

4.6 CHEMICAL ANALYSIS

Chemical testing related to the potential for corrosivity and sulphate attack was completed on one a selected soil sample from BH18-2. 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)
BH18-2	SS2/1.1m	100	7.34	5.95	20	1960

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 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 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 Stantec field investigation, it is appropriate to classify the existing ground conditions at the subject site as a Site Class of C. We note that a building founded on the bedrock at this site can likely be designed with a better site class (i.e. a Site Class of A or B); however, the OBC requires measurement of shear wave velocities in the bedrock be carried out before these site classes can be used in design.

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

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. However, footings bearing on competent, undisturbed limestone bedrock are not required to be founded below the frost penetration depth.

Exterior slabs-on-grade or slabs-on-grade within unheated area will be subject to the risk of heave 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.

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.

5.3 SITE PREPARATION

5.3.1 Grade Raise Restrictions

The final site grades in the area of the proposed building are understood to vary between approximately 77.7 m and 78.4 m which are close to existing site grades. Based on this information, significant grade raises are not planned at the site.

The native subsurface materials present at the site consist of a thin layer of fill overlying limestone bedrock which is not highly compressible. Therefore, grade raises, if required, are not anticipated to result in settlements of the underlying soil/bedrock that would adversely affect the performance of the proposed facilities.

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 building) 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.

A portion of the existing residential building contains a basement that would extend below the level of both the floor slab and typical founding elevations for the proposed building. To provide consistent subgrade conditions, all below-grade portions of the existing building as well as the basement wall backfill materials should be removed to expose the surrounding bedrock.

Following removal of the above noted materials, the prepared subgrade will require inspection by geotechnical personnel to verify all unsuitable material has been removed. Where removal of unsuitable materials extends below the floor slab subgrade level, the grade beneath the building floor slab should be raised/reinstated to 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 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 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 form which the water is pumped. The requirements for a underslab vapour barrier should be in accordance with the requirements of the Ontario Building Code.

As noted later in this report, the proposed building is recommended to be supported on shallow foundations bearing on the limestone bedrock. In the area of the existing basement, the new building foundations will need to be lowered to found on the undisturbed bedrock. If, in other areas, fill materials are present beneath the proposed founding elevation (e.g. areas where the bedrock was previously excavated for service construction) or if the bedrock surface is found to be irregular, all fill materials and/or loose rock should be removed to expose the competent bedrock surface and the grade brought up to the founding level by placing 5 MPa concrete; the limits of the concrete placement should be determined on site by a geotechnical engineer.

Inspection and testing services will be critical to ensure that all fill is removed, 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 from a geotechnical perspective for this site is the use of shallow strip and/or spread footings founded on the limestone bedrock.

5.4.1 Foundation Design Parameters - Shallow Footings

Shallow foundations bearing directly on competent limestone bedrock can be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1 MPa. The factored geotechnical bearing resistance at ULS incorporates a resistance factor of 0.5. The settlement of footings sized using the above factored bearing resistance are expected to be less than 15 mm and, therefore, Serviceability Limit States (SLS) are not anticipated to control design for footings bearing on the bedrock at this site.

As discussed in the site preparation section of the report, the foundations will need to be lowered in the vicinity of the portion of the existing building that contains a basement in order to be founded on the bedrock. Stepping of the foundations should be carried out in accordance with the requirements of the Ontario Building Code.

All footings should be founded on above a relatively level rock surface. All soil, and broken, fractured and/or loose bedrock should be removed to expose the competent bedrock surface. The bedrock surfaces beneath all footings **must be inspected by qualified geotechnical personnel** prior to placing concrete to verify assumed foundation bearing conditions and integrity.

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

0.65 between fractured limestone bedrock 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 base of the pavement subgrade 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 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 and the small retaining wall around the stairwell at the north end of the building. 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 will be utilized behind the wall system. 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 5.1: Non-Seismic Lateral Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I	Existing Fill Materials
Bulk Unit Weight, γ (kN/m^3)	22	20
Effective Friction Angle	32°	30°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.5
Coefficient of Active Earth Pressure (K_a)	0.31	0.33
Coefficient of Passive Earth Pressure (K_p)	3.25	3.0

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 5.3 below. The angle of friction between the soil and the wall has been assumed to be 0° to provide a conservative estimate.

Table 5.2: Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B - Type I	Existing Fill Materials
Bulk Unit Weight, γ (kN/m^3)	22	21
Effective Friction Angle	32°	30°
K_{AE} (Non-Yielding Wall)	0.51	0.54
Height of Application of P_{AE} from base as a ratio of wall height, (H) – Non Yielding Wall	0.44	0.44
Active Earth Pressure (K_{AE}) – Yielding Wall	0.39	0.42
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.39	0.39
Passive Earth Pressure, (K_{PE})	3	2.75
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.31	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 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 cave-in are encountered in the excavation, the slopes should be further flattened to achieve a stable configuration.

5.6.1.1 Excavations in Soil

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). The building footprint is offset a minimum of 4 m from adjacent structures but is located within 1.5 m of the driveway for the structure on the lot to the south of the site.

Based on the boreholes advanced within the site, shallow excavations within the overburden at the site are anticipated to extend through fill materials varying in composition from silty sand with gravel to clayey sand that are located above the groundwater table. Conventional hydraulic excavating equipment is considered suitable for developing excavations in these materials.

The existing fill materials are above the water table and 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 (e.g. guy wire foundations for the hydro pole located near the north property line) or encroaching into adjacent properties, the installation of a shoring system meeting the requirements of the OHSA would be required. Given the shallow depth to bedrock, a cantilevered soldier pile and lagging system could likely be used. 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.

5.6.1.2 Excavations in Bedrock

The bedrock surface was encountered at an approximate depth of 1.3 m, in both BH18-1 and BH18-2. For shallow depths of bedrock excavation, rock removal could be accomplished using mechanical methods (such as hoe ramming in conjunction with closely-spaced line drilling), although this method may be slow and hindered by the presence of thick beds/slabs of very strong limestone bedrock. Excavations extending significantly into the rock (if required) will more-efficiently be carried out using drill and blast procedures, if permitted by the City of Ottawa. All blasting should be conducted by a licensed blasting contractor with sufficient experience on projects of similar scale and detail.

Bedrock excavations with near vertical sidewalls should stand unsupported for the construction period, at least for moderate depths (i.e., total excavation depths of less than about 3 metres). However, blast damage to the bedrock walls must be avoided or else rock reinforcement would be required. Special construction procedures such as line drilling at close spacing prior to blasting or pre-shearing of the excavation limits using controlled blasting may be required to minimized blast-related bedrock damage. Furthermore, all loose rock must be removed from the sidewalls of such excavations and the sidewalls should be inspected by experienced to geotechnical personnel prior to worker-entry into the excavation to assess the sidewall conditions and confirm that additional support measures are not required.

Where new/replacement service lines will be located below the bedrock surface, consideration could be given to replacing the pipes within the existing trenches in order reduce the amount of bedrock excavation required. If the existing trench widths within the bedrock are not wide enough, excavation into the sidewalls of the existing trenches could be carried out to increase the trench width. This method of excavation will result in less bedrock removal and decreased vibrations in comparison to excavating new trenches in the bedrock.

5.6.2 Vibration Considerations and Precondition Surveys

The required construction activities for the proposed building will generate vibrations that will be perceptible to nearby residents. The vibrations are expected to be greatest if bedrock excavation by blasting or mechanical methods is carried out. A pre-construction survey should also be carried out on all of the surrounding structures and utilities prior to excavation works.

Significant precautions should be exercised if blasting is allowed/planned to be carried out due to the close proximity of existing buildings. The blasting and rock excavation activities should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs to be prepared by a specialist in this field. If practical, blasting should commence at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics and to confirm the anticipated ground vibration levels based on the contractor's blasting methods.

It is recommended that Contractor be required to submit a detailed blasting or rock excavation design plan and monitoring proposal prepared by a vibrations specialist prior to commencement of blasting or rock excavation. The construction vibrations should generally be limited to the maximum, frequency dependent peak particle velocities outlined below.

Frequency Range (Hz)	Vibration Limits (mm/sec)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

Should there be structures in the area sensitive to vibrations, more stringent specifications should be developed by a vibration specialist. For instance, the peak particle velocities may need to be limited further if there are any historic buildings in the area. Vibration monitoring should be carried out prior to and throughout the construction period.

5.6.3 Temporary Dewatering Considerations

The groundwater level measured in the piezometer installed in BH18-2 was at a depth of about 3.1 m below ground surface at the time of the field investigation which is below the bottom of the overburden materials. However, perched groundwater may be encountered at the time of excavation and may require dewatering. Groundwater inflows into excavations developed within the overburden should be possible to be handled by pumping from filtered sumps within the excavation areas.

Increased water inflows would occur if excavations extend below the groundwater table in fractured bedrock. Control of groundwater into shallow excavations into the bedrock is expected to be able to be handled by pumping from sumps in the excavations; however, further investigation would be required to assess potential inflow volumes for these conditions.

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 or more if required 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 subbedding 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 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/duct 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 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 on 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 1960 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 higher than 1000 µg/g generally indicate that a sulphate attack may be expected on concrete in contact with soil and groundwater. High Sulphate-Resistant Portland cement is therefore recommended for use in concrete at this 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 7.3 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 5.95 (ohm-m) suggests a severe corrosive environment.

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 TransCanada Pipelines Limited. 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.

Ramy Saadeldin, PhD, P.Eng.
Geotechnical Engineering

Kevin Nelson, P.Eng.
Principal, Geotechnical Engineering

Appendix E DRAWINGS