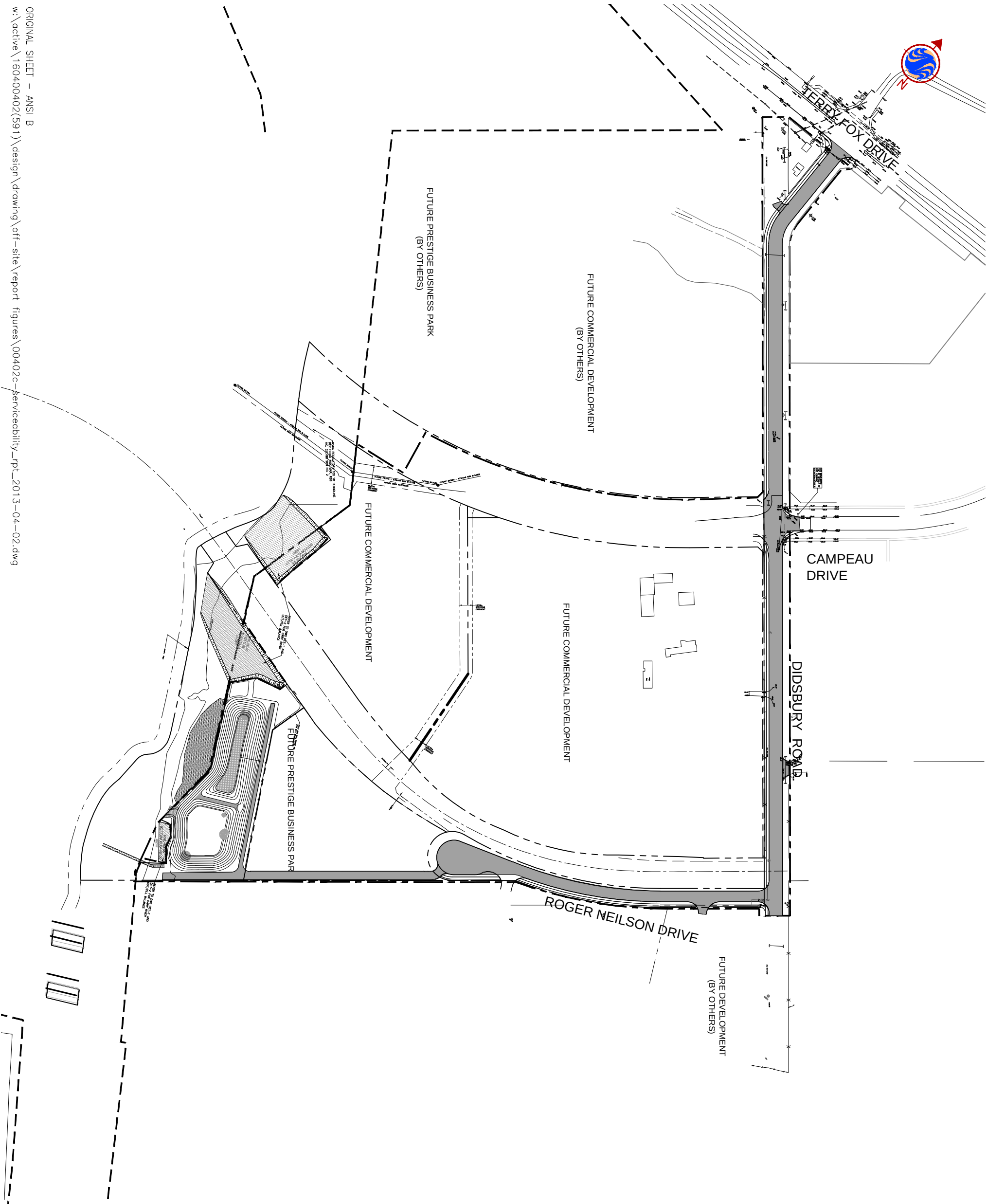


ORIGINAL SHEET – ANSI B
w:\active\160400402(591)\design\drawing\off-site\report figures\00402c-serviceability_rpt_2013-04-02.dwg



Stantec Consultin
1505 Laperriere Av
Ottawa ON Cana
K1Z 7T1
Tel. 613.722.44
Fax. 613.722.27
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Stantec

Legend

--- LIMITS OF SUBDIVISION



Figure No.

FIG-1

Title

LIMITS OF SUBDIVISION

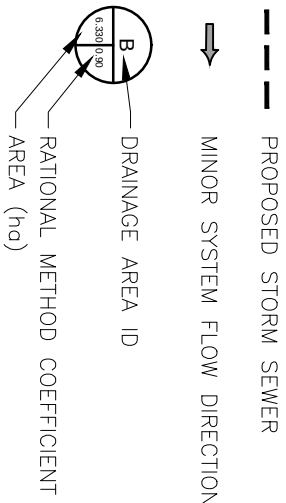
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TAGGART-LOBLAWS SUBDIVISION

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Notes

REFER TO DRAWING SD-1 FOR DETAILS



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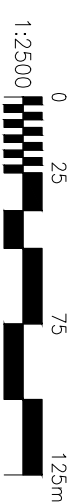
Figure No.
FIG-2

Title
STORM DRAINAGE AREAS



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- Legend
- PROPOSED WATERMAIN
 - EXISTING WATERMAIN



2011.11.11
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Figure No.
FIG-3

Title
**PROPOSED WATER
DISTRIBUTION SYSTEM**

Appendix A

Sewer Design Sheets / Calculations



SUBDIVISION:

Taggart / LPL
KW - North of 417

DATE: 11-Nov-11
REV: 6-Jun-13
DESIGNED BY: aml
CHECKED BY: tw

SANITARY SEWER DESIGN SHEET

(City of Ottawa)

FILE NUMBER: 1604-00402/20

DESIGN PARAMETERS

AVG. DAILY FLOW / PERSON =	350.00 l/p/day	COMMERCIAL	50,000 L/d/Ha
MINIMUM VELOCITY =	0.60 m/s	MIXED USE	50,000 L/d/Ha
MANNINGS n =	0.013	INSTITUTIONAL	35,000 L/d/Ha
MAX PEAK FACTOR =	4.0	INFILTRATION	0.28 l/s/Ha
MIN PEAK FACTOR =	2.4	RESIDENTIAL HARMON PEAKING FACTOR PERSONS/UNIT =	4.0
Peaking Factor Industrial:	1.5		
Peaking Factor Comm. / Inst.:	1.5		

LOCATION			RESIDENTIAL AREA AND POPULATION						COMM		MIXED		INDUST.		C+M+I	INFILTRATION				PIPE					
STREET / LOCATION	FROM M.H.	TO M.H.	AREA (ha)	UNITS	POP.	CUMULATIVE AREA (ha)	POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA (mm)	SLOPE (%)	CAP. (FULL) (l/s)	VEL. (FULL) (m/s)	(ACT.) (m/s)	
Exist. Didsbury Sewer ¹	Exist	SAN15	102.31											54.44			58.650	113.09	103.0	450	0.25	149.24	0.91	0.88	
Transitway/S																									
Roger Neilson T-2/D	SAN19	SAN18	2.23			2.23	0			1.44	1.44			1.25	3.67	3.67	1.028	2.28	35.0	250	0.40	39.78	0.78	0.35	
Roger Neilson E2	SAN18	SAN17				2.23	0			2.89	4.33			3.76	2.89	6.56	1.837	5.60	79.7	250	0.40	39.78	0.78	0.45	
Roger Neilson	SAN17	SAN16				2.23	0				4.33			3.76	0.00	6.56	1.837	5.60	47.1	300	0.40	63.51	0.87	0.44	
Roger Neilson E1	SAN16	SAN15				2.23	0			1.39	5.72			4.97	1.39	7.95	2.226	7.20	116.1	300	0.40	63.51	0.87	0.48	
Didsbury	SAN15	EX.SAN7	1.66			3.89	0				5.72			59.41	1.66	9.61	61.341	120.75	100.0	450	0.25	149.24	0.91	0.90	
	EX.SAN7	EX.SAN6				3.89					5.72			59.41		9.61	61.341	120.75	100.0	450	0.25	149.24	0.91	0.90	
	EX.SAN6	EX.SAN5				3.89					5.72			59.41		9.61	61.341	120.75	95.0	450	0.25	149.24	0.91	0.90	
	EX.SAN5	EX.SAN4				3.89					5.72			59.41		9.61	61.341	120.75	38.0	450	0.25	149.24	0.91	0.90	
Exist. Campeau Sewer ²	Exist	EX.SAN4								25.54	25.54			22.17	25.54	25.54	7.151	29.32		250	0.38	38.76	0.76	0.74	
	EX.SAN4	SAN4				3.89					31.26			81.58		35.15	68.492	150.07	6.5	450	0.25	149.24	0.91	0.96	
Campeau West of Carp ³	FUT	SAN8	57.05		3252	57.05	3252	3.412	45.54	55.56	55.56		65.40	87.99	178.01	178.01	49.843	183.37	35.8	675	0.20	391.14	1.06	0.89	
On-site C	SAN10	SAN9				0.00	0			2.43	2.43			2.11	2.43	2.43	0.680	2.79	42.9	250	0.65	50.49	0.99	0.44	
On-site	SAN9	SAN8				0.00	0				2.43			2.11	0.00	2.43	0.680	2.79	31.0	250	0.40	39.78	0.78	0.38	
Campeau A2/Campeau Drive	SAN8	SAN7	1.55			58.60	3252	3.412	45.54		57.99	2.87	2.87	92.60	4.42	184.86	51.761	189.90	47.8	675	0.20	391.14	1.06	0.90	
Campeau	SAN7	SAN6				58.60	3252	3.412	45.54		57.99		2.87	92.60	0.00	184.86	51.761	189.90	70.8	675	0.20	391.14	1.06	0.90	
Campeau	SAN6	SAN5				58.60	3252	3.412	45.54		57.99		2.87	92.60	0.00	184.86	51.761	189.90	55.3	675	0.20	391.14	1.06	0.90	
Campeau B	SAN5	SAN4				58.60	3252	3.412	45.54	6.27	64.26		2.87	98.04	6.27	191.13	53.516	197.10	68.8	675	0.20	391.14	1.06	0.91	
Didsbury	SAN4	SAN3				62.49	3252	3.412	45.54		64.26		34.13	179.62	0.00	226.28	122.008	347.17	94.2	750	0.16	465.12	1.02	0.99	
Didsbury	SAN3	SAN2				62.49	3252	3.412	45.54	4.69	68.95		34.13	183.69	4.69	230.97	123.322	352.55	96.0	750	0.16	465.12	1.02	0.99	
Didsbury	SAN2	SAN1				62.49	3252	3.412	45.54		68.95		34.13	183.69	0.00	230.97	123.322	352.55	92.1	750	0.16	465.12	1.02	0.99	
Didsbury	SAN1	SANMH				62.49	3252	3.412	45.54		68.95		34.13	183.69	0.00	230.97	123.322	352.55	35.2	750	0.55	861.84	1.89	1.52	

1 Existing Didsbury sanitary sewer currently conveys wastewater flow from existing developed areas south of Hwy 417. See the Kanata West Master Servicing Study's Fig. 4.2-2 and drawing S-1.

2 Existing Didsbury sanitary sewer north of Campeau Drive currently conveys wastewater flow from existing developed areas east of Didsbury Rd. See the Kanata West Master Servicing Study's Fig. 4.2-2 and drawing S-1.

3 Future wastewater flows from development west of Carp River per Kanata West Master Servicing Study's Fig. 4.2-1 and drawing S-1.

4 Missing drainage area information for SANMH north and west inlet pipes (375mm dia. SAN @ 0.30% and 375mm dia. SAN @ 0.15% respectively). Pipes assumed to be flowing at full capacity. Flows (100.32L/s and 70.68L/s) added to total flow column.

 Stantec	KANATA WEST - TAGGART		STORM SEWER DESIGN SHEET (City of Ottawa)										DESIGN PARAMETERS																	
	Didsbury North of Campeau												I = a / (t+b) ^c (As per City of Ottawa Guidelines, 2004)																	
	DATE:	23-Jul-12											1:5 yr		1:10 yr		MANNING'S n = 0.013													
	DWG REV:	REV 4											998.07		1174.184		MINIMUM COVER: 2.00 m													
DESIGNED BY:	npc	FILE NUMBER: 1604-00402		6.014		0.816		MIN T _C 10 min																						
CHECKED BY:																														
LOCATION			DRAINAGE AREA										PIPE SELECTION																	
AREA ID / DESCRIPTION	FROM M.H.	TO M.H.	5-year					10-year					T of C	I ₅ -YEAR	I ₁₀ -YEAR	Q ₅ -YEAR (CIA/360)	Q ₁₀ -YEAR (CIA/360)	Q _{TOTAL} (CIA/360)	LENGTH	PIPE SIZE	SLOPE	Q _{CAP} (FULL)	Q _{ACT} (FULL)	VEL. (m/s)	VEL. (m/s)	TIME OF FLOW (min)				
			AREA	C ¹	ACCUM. AREA	A x C	ACCUM. AxC	AREA	C	ACCUM. AREA	A x C	ACCUM. AxC																		
			(ha)	(-)	(ha)	(ha)	(ha)	(ha)	(-)	(ha)	(ha)	(ha)	(min)	(mm/h)	(mm/h)	(L/s)	(L/s)	(L/s)	(m)	(mm)	%	(L/s)	(-)	(m/s)	(m/s)	(min)				
CBMH123 CBMH122 CBMH121 CBMH120A CBMH120 CBMH120 DICB119 and CBMH119 CBMH118	CBMH123	CBMH122	0.118	0.67	0.118	0.079	0.079	0.424	0.67	0.424	0.284	0.284	10.00	104.19	122.14	22.9	0.0	22.9	60.4	300	1.24	112.3	0.20	1.54	1.00	1.01				
	CBMH122	CBMH121	0.122	0.67	0.240	0.082	0.161						11.01	99.15	116.21	44.3	0.0	44.3	58.4	300	1.42	120.2	0.37	1.65	1.28	0.76				
	CBMH121	CBMH120A			0.240		0.161						11.77	95.70	112.14	42.7	88.5	131.2	46.0	450	0.61	232.3	0.56	1.41	1.25	0.61				
	CBMH120A	CBMH120	0.069	0.67	0.309	0.046	0.207						12.38	93.11	109.09	53.5	86.1	139.6	52.7	450	1.44	356.9	0.39	2.17	1.73	0.51				
	CBMH120	CBMH119	0.105	0.67	0.414	0.070	0.277						12.89	91.07	106.69	70.2	84.2	154.4	54.3	525	0.45	301.0	0.51	1.35	1.16	0.78				
	CBMH119	CBMH118	0.293	0.46	0.707	0.134	0.411						13.67	88.13	103.23	100.7	81.5	182.1	77.7	600	0.33	368.0	0.49	1.26	1.08	1.20				
	CBMH118	STM117	0.295	0.67	1.002	0.198	0.609						14.87	83.99	98.36	142.0	77.6	219.7	98.9	600	0.35	379.0	0.58	1.30	1.16	1.42				
	STM117	ExSTM_MH			1.002		0.609						16.29	79.62	93.22	134.6	73.6	208.2	9.8	600	0.30	350.9	0.59	1.20	1.08	0.15				

Notes:
 1) Land coverage was known therefore the Runoff Coefficient was calculated based on percent 'soft' and 'hard' areas (C=0.20 'soft' & C=0.90 'hard').

	KANATA WEST - TAGGART		STORM SEWER DESIGN SHEET (City of Ottawa) Pond 3 Network - Scenario A / B: Richcraft lands not included										DESIGN PARAMETERS I = a / (t+b) ^c (As per City of Ottawa Guidelines, 2004)													
	SWM Pond Drainage Area												PIPE SELECTION													
	DATE: 7-Jun-13	1:5 yr 1:10 yr											MANNING'S n = 0.013		MINIMUM COVER: 2.00 m		MIN T _C 10 min									
DWG REV:	pw		FILE NUMBER: 163600402		a = 998.07	1174.184																				
DESIGNED BY:	aml				b = 6.053	6.014																				
CHECKED BY:					c = 0.814	0.816																				
LOCATION			DRAINAGE AREA																							
AREA ID / DESCRIPTION	FROM M.H.	TO M.H.	AREA	C	5-year ACCUM. AREA	A x C	ACCUM. AxC	AREA	C	10-year ACCUM. AREA	A x C	ACCUM. AxC	T of C	I ₅ YEAR	I ₁₀ YEAR	Q ₅ YEAR (CIA/360)	Q ₁₀ YEAR (CIA/360)	Q _{TOTAL} (CIA/360)	LENGTH	PIPE SIZE	SLOPE	Q _{cap} (FULL)	Q _{acc} (FULL)	VEL. (FULL)	VEL. (ACT)	TIME OF FLOW
			(ha)	(-)	(ha)	(ha)	(ha)	(ha)	(ha)	(-)	(ha)	(ha)	(ha)	(min)	(mm/h)	(mm/h)	(L/s)	(L/s)	(L/s)	(m)	(mm)	%	(L/s)	(L/s)	(m/s)	(m/s)
CAMPEAU 4 ^{1C,4B} CAMPEAU 3 ^{4B} CAMPEAU 2 ^{4B}	STM116	STM115						0.421	0.70	0.421	0.295	0.295	10.00	104.19	122.14	0	100	100	56.8	525	0.20	192.3	0.52	0.89	0.77	1.23
	STM115	STM114						0.270	0.70	0.691	0.189	0.484	11.23	98.09	114.96	0	154	154	72.9	525	0.25	215.0	0.72	0.99	0.95	1.28
	STM114	STM113						0.199	0.70	0.890	0.139	0.623	12.51	92.55	108.44	0	188	188	53.3	600	0.25	307.0	0.61	1.09	0.99	0.90
													13.42													
CAMPEAU 1 ^{1C, 4B}	STUB	STM113						0.636	0.70	0.636	0.445	0.445	10.00	104.19	122.14	0	151	151	10.0	450	1.75	372.4	0.41	2.36	1.90	0.09
											10.09															
A1/A2 ^{1D, 2, 3, 4C}	STUB	STM113	7.558	0.79	7.558	5.942	5.942						17.00	77.61	90.86	1281	0	1281	24.8	1200	0.20	1743.6	0.73	1.54	1.48	0.28
											17.28															
B 4C- TAGGART C 4B- TAGGART	STM113	STM112			7.558		5.942			1.526		1.068	17.28	76.85	89.97	1268	267	1535	17.4	1200	0.30	2135.4	0.72	1.89	1.80	0.16
	STM112	STM111	6.298	0.86	13.856	5.416	11.358			1.526		1.068	17.44	76.42	89.46	2411	265	2677	42.9	1500	0.35	4182.0	0.64	2.37	2.18	0.33
	STM111	STM110	2.438	0.84	16.294	2.048	13.406			1.526		1.068	17.77	75.57	88.46	2814	262	3077	91.7	1500	0.35	4182.0	0.74	2.37	2.28	0.67
	STM110	STM109			16.294		13.406			1.526		1.068	18.44	73.88	86.47	2751	257	3008	66.9	1500	0.35	4182.0	0.72	2.37	2.26	0.49
	STM109	STM103			16.294		13.406	0.951	0.700	2.477	0.666	1.734	18.93	72.69	85.07	2707	410	3117	52.8	1500	0.35	4182.0	0.75	2.37	2.28	0.39
													19.32													
CBMH 108 ^{1A,4A}	CBMH108	STM107A	0.101	0.67	0.101	0.068	0.068						10.00	104.19	122.14	20	0	20	33.8	300	0.50	68.0	0.29	0.97	0.70	0.81
											10.81															
	STM107A	STM107			0.101		0.068	2.792	0.690	2.792	1.926	1.926	19.40	71.60	83.79	13	448	462	22.9	900	0.10	572.5	0.81	0.90	0.89	0.43
	STM107	STM106	1.766	0.61	1.867	1.073	1.141			2.792		1.926	19.83	70.63	82.65	224	442	666	112.1	1050	0.10	863.5	0.77	1.00	0.97	1.92
	STM106	STM105	0.081	0.60	1.948	0.049	1.190			2.792		1.926	21.75	66.62	77.95	220	417	637	50.6	1050	0.10	863.5	0.74	1.00	0.96	0.88
STREET 2-2a / STREET 2-1 ^{4B} / E2 ^{4C}	STM105	STM104	3.187	0.60	5.135	1.912	3.102			2.792		1.926	22.63	64.95	75.99	560	407	966	82.3	1050	0.20	1221.2	0.79	1.41	1.38	0.99
													23.63													
EXT-2	B2	STM104			0.000		0.000	0.908	0.800	0.908	0.726	0.726	10.00	104.19	122.14	0	246	246	24.5	600	0.25	307.0	0.80	1.09	1.07	0.38
	STM104	STM103			5.135		3.102			3.700		2.653	23.63	63.18	73.91	544	545	1089	105.0	1200	0.12	1350.6	0.81	1.19	1.18	1.49
													25.11													
D - FUTURE DEV. ^{4B}	STM103	STM102	1.520	0.80	22.949	1.216	17.724			6.177		4.387	25.11	60.71	71.01	2989	865	3855	128.0	1800	0.16	4597.9	0.84	1.81	1.81	1.18
	STM102	STM101								0.000		0.000	26.29	58.91	68.89	0	0	1100	17.5	1050	0.25	1365.4	0.81	1.58	1.55	0.19
	STM101	POND								0.000		0.000	26.48	58.63	68.56	0	0	1100	19.5	1050	0.25	1365.4	0.81	1.58	1.55	0.21
	STM102	STM102-2											26.29	58.91	68.89	0	0	2754.5	117.2	1650	0.25	4557.2	0.60	2.13	1.94	1.01
STM102-2	POND2												26.48	58.63	68.56	0	0	2754.5	10.5	1650	0.25	4557.2	0.60	2.13	1.94	0.09
													26.69													
Total Area			22.949	ha		17.724		6.177		4.387																

Notes:

- Time of Concentration for upstream areas were determined as follows:
 - Area CBMH 108 - Assumed initial Tc of 10 minutes per City of Ottawa Design Guidelines.
 - Transitway - Assumed initial Tc of 10 minutes per City of Ottawa Design Guidelines.
 - Campeau Drive - Assumed initial Tc of 10 minutes per City of Ottawa Design Guidelines.
 - A1/A2 - Assumed initial time of concentration of 10 minutes plus travel within future storm sewers (longest path approx. 425m) at approx 1.0m/s = +/- 17minutes
- Outlet pipe from Area's A1/A2 was sized to convey peak pump rate of 150L/s. Downstream sewers are sized to accommodate unrestricted release from A1 / A2.
- Runoff coefficient evaluated based on a weighted average of A1 4.691ha at C=0.90 and A2 2.867ha at C=0.20 per KWMSS
- Runoff coefficients were assigned to the areas as follows:
 - Land coverage was known therefore the Runoff Coefficient was calculated based on percent 'soft' and 'hard' areas (C=0.20 'soft' & C=0.90 'hard').
 - A Runoff Coefficient per City of Ottawa Design Guidelines was assigned, since proposed land use results in higher runoff coefficient value than provided by KWMSS.
 - A Runoff Coefficient per KWMSS was assigned, since suggested C value is greater than specified by City of Ottawa Sewer Design Guidelines for proposed land use.
 - Runoff Coefficient for Transitway Lands was calculated based on percent 'soft' and 'hard' areas (C=0.30 'soft' & C=0.90 'hard').
- Subcatchment areas from non-transitway road were interpreted in the following way
 - W4 was designed to capture at the 10 year rate as Terry Fox Dr is assumed to be an arterial road. Earl Grey sag contributing areas (W2,W3) also assumed to be serviced to 10-year rate
 - J2 was designed to capture at the 10 year rate as Highway 417 off ramp is assumed to be an arterial roadway
 - I2, I3 and G10 were designed to capture at the 10 year rate as Terry Fox Dr and Highway 417 off ramp is assumed to be an arterial roadway
- Includes transitway area west of local drainage area (A2 - C2), previously directed to STM107.

Calculation of Velocity Factors for Partially Filled Pipes

manning's n varies with depth as per Camp 1944

formulae taken from "Explicit solutions of the Manning equation for partially filled circular pipes" -Omer Akgıray
W:\resource\Water Resources Group\SWM Articles\Mannings n Roughness\Closed Circular\Explicit solutions

A= flow area (m²)
d= depth of flow (m)
D= pipe diameter (m)
n= manning's n at flow depth=d
n_f= manning's n at full flow
Q_f= full flow (m³/s)
R_h= hydraulic radius
S= Slope of energy grade line
V_f= Full velocity (m/s)
X= 1-(d/D)
θ = surface angle

values and constants used for calculations

n_f = 0.013
S= 0.25 %
D= 0.9 m
A_f= 0.636 m²
Q_f= 0.905 m³/s
V_f= 1.423 m/s

$$\theta = 2\cos^{-1}\left(1 - 2\frac{d}{D}\right)$$

$$A = \frac{D^2}{8}(\theta - \sin\theta)$$

$$n = n_f(1 - 0.8627x^5 + 0.4281x^4 + 0.7626x^3 - 1.02x^2 + 0.8057x)$$

$$Q = \frac{A}{n}R_h^{2/3}S^{1/2}$$

$$R_h = \frac{D}{4}\left(\frac{\theta - \sin\theta}{\theta}\right)$$

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

	A (m ²)	x	n	R _s	Q (m ³ /s)	V (m/s)	Q/Q _f	V/V _f
0	0	0	1	0.014478	0	0	0	0
0.01	0.40067	0.001077	0.99	0.014673	0.005972	0.000120774	0.112165	0.008783
0.02	0.567588	0.003036	0.98	0.014856	0.011888	0.000532281	0.175306	0.005588
0.03	0.696332	0.005561	0.97	0.015028	0.017747	0.001258888	0.226375	0.001391
0.04	0.805432	0.008536	0.96	0.015188	0.02355	0.002308644	0.270473	0.001907
0.05	0.902054	0.011892	0.95	0.015338	0.029296	0.003684075	0.309794	0.002173
0.06	0.989868	0.015584	0.94	0.015478	0.034985	0.005385129	0.345558	0.004287
0.07	1.071053	0.019576	0.93	0.015607	0.040617	0.007410441	0.378539	0.008187
0.08	1.147026	0.023842	0.92	0.015728	0.046192	0.009757964	0.409268	0.010787
0.09	1.218771	0.02836	0.91	0.015839	0.051709	0.01242532	0.438135	0.013727
0.1	1.287002	0.033109	0.9	0.015942	0.057168	0.01540999	0.465432	0.017025
0.11	1.352261	0.038075	0.89	0.016037	0.062569	0.01870943	0.491389	0.02067
0.12	1.414966	0.043242	0.88	0.016124	0.067912	0.022321126	0.516189	0.02466
0.13	1.475452	0.048599	0.87	0.016203	0.073197	0.02624263	0.539979	0.028992
0.14	1.533988	0.054135	0.86	0.016275	0.078423	0.030471565	0.562882	0.035661
0.15	1.590798	0.059839	0.85	0.01634	0.08359	0.035005622	0.585002	0.04115
0.16	1.646067	0.065701	0.84	0.016398	0.088698	0.039842558	0.606422	0.04621
0.17	1.699955	0.071714	0.83	0.016451	0.093746	0.044980172	0.627218	0.049829
0.18	1.752596	0.077869	0.82	0.016497	0.098735	0.050416295	0.64745	0.055049
0.19	1.804107	0.084159	0.81	0.016538	0.103664	0.056148767	0.667174	0.060321
0.2	1.85459	0.090577	0.8	0.016573	0.108532	0.062175418	0.686435	0.066849
0.21	1.904135	0.097117	0.79	0.016604	0.11334	0.068494049	0.705274	0.075689
0.22	1.952821	0.103772	0.78	0.016629	0.118088	0.07510241	0.723725	0.080657
0.23	2.000718	0.110537	0.77	0.01665	0.122774	0.081998183	0.741819	0.090292
0.24	2.047891	0.117405	0.76	0.016666	0.1274	0.089178966	0.759583	0.098523
0.25	2.094395	0.124372	0.75	0.016679	0.131963	0.096642248	0.777039	0.106769
0.26	2.140283	0.131433	0.74	0.016687	0.136465	0.104385402	0.794209	0.115323
0.27	2.185602	0.138583	0.73	0.016692	0.140905	0.11240566	0.81111	0.124184
0.28	2.230395	0.145816	0.72	0.016693	0.145282	0.120700106	0.827757	0.133347
0.29	2.274702	0.153129	0.71	0.016691	0.149596	0.12926566	0.844164	0.14281
0.3	2.318559	0.160516	0.7	0.016686	0.153847	0.138099064	0.860343	0.152569
0.31	2.362	0.167975	0.69	0.016678	0.158034	0.147196873	0.876303	0.162621
0.32	2.405057	0.1755	0.68	0.016668	0.162158	0.156555446	0.892055	0.17296
0.33	2.447759	0.183087	0.67	0.016654	0.166218	0.166170932	0.907605	0.183583
0.34	2.490134	0.190733	0.66	0.016639	0.170213	0.176039269	0.92296	0.194486
0.35	2.532207	0.198434	0.65	0.016621	0.174142	0.186156173	0.938126	0.205662
0.36	2.574004	0.206186	0.64	0.0166	0.178007	0.196517133	0.953106	0.217109
0.37	2.615548	0.213985	0.63	0.016578	0.181806	0.207117406	0.967906	0.22882
0.38	2.656861	0.221828	0.62	0.016554	0.185539	0.217952016	0.982529	0.240795
0.39	2.697964	0.22971	0.61	0.016529	0.189205	0.229015749	0.996976	0.253013
0.4	2.738877	0.23763	0.6	0.016501	0.192804	0.240303151	1.01251	0.265483
0.41	2.77962	0.245582	0.59	0.016473	0.196335	0.251808529	1.025355	0.278194
0.42	2.820211	0.253564	0.58	0.016442	0.199799	0.263525951	1.039289	0.291139
0.43	2.86067	0.261572	0.57	0.016411	0.203194	0.275449245	1.053053	0.304311
0.44	2.901013	0.269603	0.56	0.016378	0.20652	0.287572005	1.066649	0.317705
0.45	2.941258	0.277654	0.55	0.016344	0.209777	0.299887587	1.080077	0.331311
0.46	2.981421	0.285721	0.54	0.016309	0.212964	0.312389121	1.093337	0.345122
0.47	3.021521	0.293801	0.53	0.016273	0.21608	0.325069505	1.106428	0.359131
0.48	3.061571	0.301891	0.52	0.016235	0.219125	0.337921415	1.119351	0.37333
0.49	3.10159	0.309987	0.51	0.016197	0.222099	0.350937311	1.132104	0.387709
0.5	3.141593	0.318086	0.5	0.016159	0.225	0.364109436	1.144688	0.402262
0.51	3.181595	0.326186	0.49	0.016119	0.227828	0.377429824	1.157101	0.416978
0.52	3.221614	0.334282	0.48	0.016079	0.230583	0.390890308	1.169343	0.431815
0.53	3.261665	0.342372	0.47	0.016037	0.233263	0.404482518	1.181414	0.446865
0.54	3.301764	0.350452	0.46	0.015995	0.235868	0.418197894	1.193311	0.462018
0.55	3.341927	0.358519	0.45	0.015953	0.238398	0.432027681	1.205035	0.477297
0.56	3.382172	0.366569	0.44	0.01591	0.240851	0.445962942	1.216585	0.492655
0.57	3.422515	0.3746	0.43	0.015866	0.243226	0.459994555	1.22796	0.508194
0.58	3.462974	0.382609	0.42	0.015821	0.245524	0.474113216	1.23916	0.523792
0.59	3.503566	0.390591	0.41	0.015776	0.247742	0.488309442	1.250182	0.539476
0.6	3.544308	0.398543	0.4	0.01573	0.24988	0.502573569	1.261027	0.555234
0.61	3.585222	0.406462	0.39	0.015684	0.251937	0.516895751	1.271694	0.571057
0.62	3.626324	0.414345	0.38	0.015637	0.253912	0.531265956	1.282183	0.586933
0.63	3.667637	0.422188	0.37	0.015589	0.255804	0.54567396	1.292492	0.602851
0.64	3.709181	0.429987	0.36	0.01554	0.257611	0.560109338	1.302621	0.618799
0.65	3.750978	0.437738	0.35	0.015491	0.259333	0.574561454	1.312568	0.634765
0.66	3.793052	0.445439	0.34	0.015441	0.260968	0.589019446	1.322334	0.650738
0.67	3.835426	0.453085	0.33	0.015391	0.262515	0.603472205	1.331918	0.666706
0.68	3.878128	0.460673	0.32	0.015339	0.263972	0.617908359	1.341317	0.682654
0.69	3.921185	0.468198	0.31	0.015287	0.265338	0.632316238	1.350533	0.698572
0.7	3.964626	0.475656	0.3	0.015234	0.266611	0.646683845	1.359562	0.714445
0.71	4.008483	0.483044	0.29	0.01518	0.26779	0.660998814	1.368403	0.73026
0.72	4.05279	0.490357	0.28	0.015126	0.268872	0.675248364	1.377056	0.746003
0.73	4.097583	0.49759	0.27	0.01507	0.269856	0.689419239	1.385517	0.761658
0.74	4.142902	0.504739	0.26	0.015013	0.270738	0.703497644	1.393784	0.777212
0.75	4.18879	0.5118	0.25	0.014955	0.271518	0.717469166	1.401854	0.792647
0.76	4.235295	0.518767	0.24	0.014897	0.272193	0.731318674	1.409724	0.807948
0.77	4.282467	0.525636	0.23	0.014837	0.272759	0.745030211	1.417389	0.823096
0.78	4.330364	0.5324	0.22	0.014775	0.273213	0.758586861	1.424843	0.838073
0.79	4.37905	0.539056	0.21	0.014713	0.273553	0.771970586	1.43208	0.85286
0.8	4.428595	0.545595	0.2	0.014649	0.273774	0.785162034	1.439093	0.867433
0.81	4.479078	0.552013	0.19	0.014584	0.273873	0.798140308	1.445871	0.881771
0.82	4.530589	0.558304	0.18	0.014517	0.273844	0.810882685	1.452405	0.895849
0.83	4.583223	0.564459	0.17	0.014449	0.273683	0.823364262	1.458679	0.909638
0.84	4.637118	0.570471	0.16	0.014379	0.273384	0.835557533	1.464679	0.923109
0.85	4.692388	0.576334	0.15	0.014308	0.27294	0.847431839	1.470383	0.936228
0.86	4.749197	0.582038	0.14	0.014235	0.272344	0.858952686	1.475768	0.948956
0.87	4.807733	0.587573	0.13	0.01416	0.271587	0.870080847	1.480804	0.96125
0.88	4.868219	0.59293	0.12	0.014084	0.270658	0.880771191	1.485455	0.973061
0.89	4.930924	0.598098	0.11	0.014006	0.269545	0.890971104	1.489674	0.984329
0.9	4.996183	0.603064	0.1	0.013925	0.268233	0.900618301	1.493405	0.994987
0.91	5.064415	0.607813	0.09	0.013843	0.266703	0.909637736	1.496575	1.004951
0.92	5.136159	0.61233	0.08	0.013758	0.264932	0.917937027	1.499089	1.014121
0.93	5.212132	0.616596	0.07	0.013672	0.262889	0.925399435	1.50082	1.022365
0.94	5.293317	0.620589	0.06	0.013583	0.260533	0.931872457	1.501594	1.029516
0.95	5.381132	0.624281	0.05	0.013492	0.257806	0.937147967	1.501165	1.035345
0.96	5.477754	0.627637	0.04	0.013398	0.254621	0.940924337	1.499154	1.039517
0.97	5.586853	0.630611	0.03	0.013303	0.250831	0.942724181	1.494937	1.05069
0.98	5.715597	0.633613	0.02	0.013204	0.246163	0.941676716	1.493721	1.040348
0.99	5.825216	0.63096	0.01	0.013103	0.239918	0.935693718	1.4731	1.03373
1	6.283185	0.36173	0	0.013	0.225	0.905155576	1.42825	1

Appendix B

Potable Water Servicing Analysis



**Taggart - Loblaws Subdivision -
Potable Water Servicing Analysis**

Prepared by:

Stantec Consulting Ltd.
1505 Laperriere Avenue
Ottawa ON
K1Z 7T1
CANADA

Project #160400402/83

March 31, 2012

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Taggart - Loblaws Subdivision - Potable Water Servicing Analysis

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1.0 Potable Water Analysis

1.1 BACKGROUND

Stantec Consulting Ltd. (Stantec) has undertaken a hydraulic assessment of the potable water servicing for a proposed commercial development located adjacent to the intersection of Didsbury Rd. and Campeau Dr. in the former City of Kanata regional boundary. **Figure 1.1** shows the location of the proposed subdivision. The proposed development is located within the City of Ottawa (the City) water distribution Pressure Zone 3W. This zone is supplied by water from Pressure Zone 2W through the Glen Cairn Pumping Station. An elevated water storage tank on Hazeldean Road (Stittsville Tank) provides floating storage to Zone 3W.

Figure 1.1: Proposed Taggart-Loblaws Subdivision location



1.2 ALLOWABLE PRESSURES

The City has until recently followed the 1991 Regional Municipality of Ottawa Carleton (RMOC) Water Distribution Design Guidelines. In late May 2010, the City of Ottawa's Infrastructure Services Department issued its own *Design Guidelines for Water Distribution*, dated as July 2010. This report uses the new July 2010 City of Ottawa Water Distribution Design Guidelines for the analysis of the water distribution system.

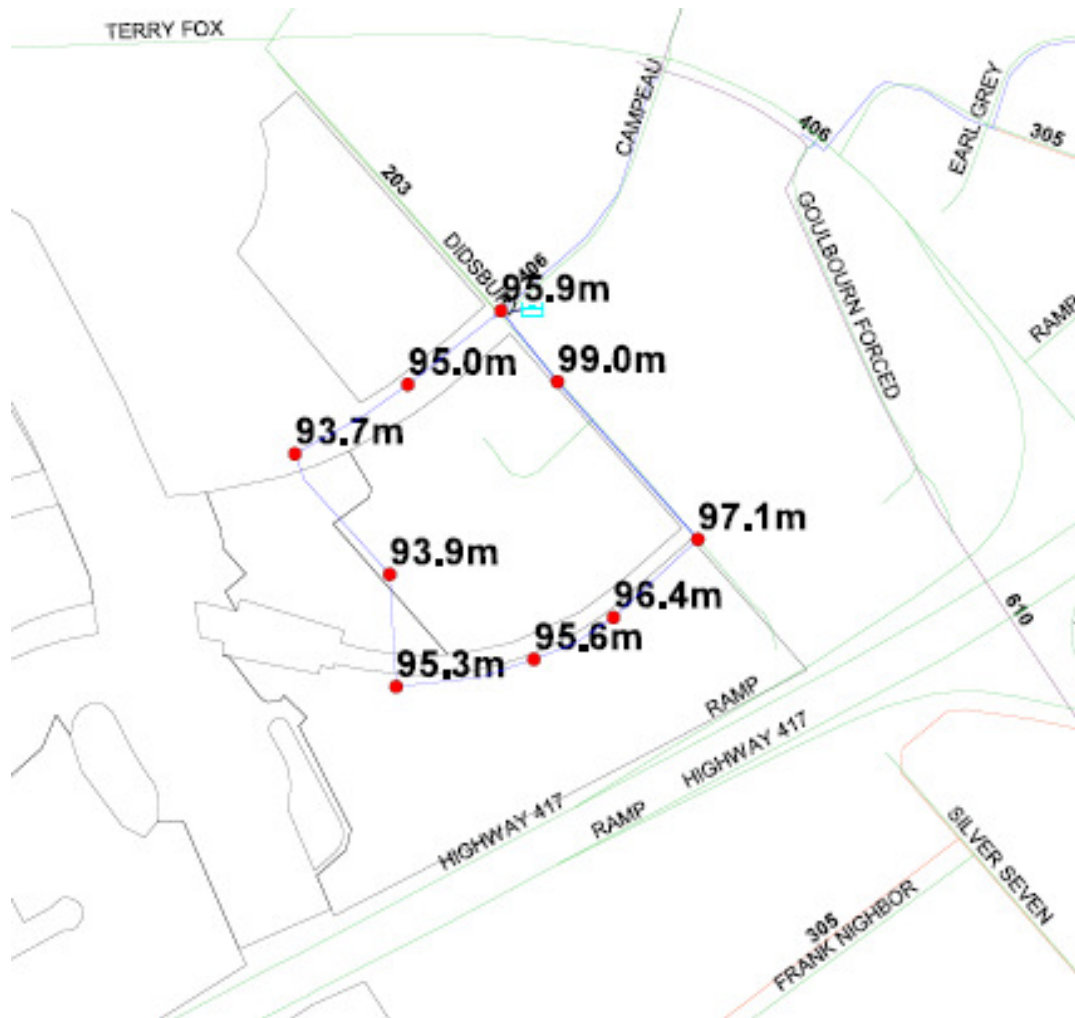
The Guidelines state the design objective for system pressures in section 4.2.2 of the document (p.4.2):

- *Normal operating pressures to be between 345 kPa (50 psi) and 552 kPa (80 psi) under maximum daily flow.*
- *In accordance with MOE Guidelines, pressures to be not less than 276 kPa (40 psi) under maximum hourly demand.*
- *Under fire flow analysis, the combination of maximum day and fire flow demand shall not reduce the residual pressure at any point in the distribution system below 140 kPa (20 psi.)*
- *In accordance with the Ontario Code & Guide for Plumbing, the maximum pressure at any point in the distribution system in occupied areas outside of the public right-of-way shall not exceed 552 kPa (80 psi.)*
- *The maximum pressure at any point in the distribution system in unoccupied areas shall not exceed 689 kPa (100 psi.)*
- *Feeder mains, which have been provided primarily for the purpose of redundancy, shall meet, at a minimum, the basic day plus fire flow demand.*

1.3 GROUND ELEVATIONS

The ground elevations of the proposed development range from approximately 93.7 to 99.0m. The elevations shown in **Figure 1.2** were assigned to the nodes in the hydraulic model.

Figure 1.2: Ground elevations (m) in area of proposed development



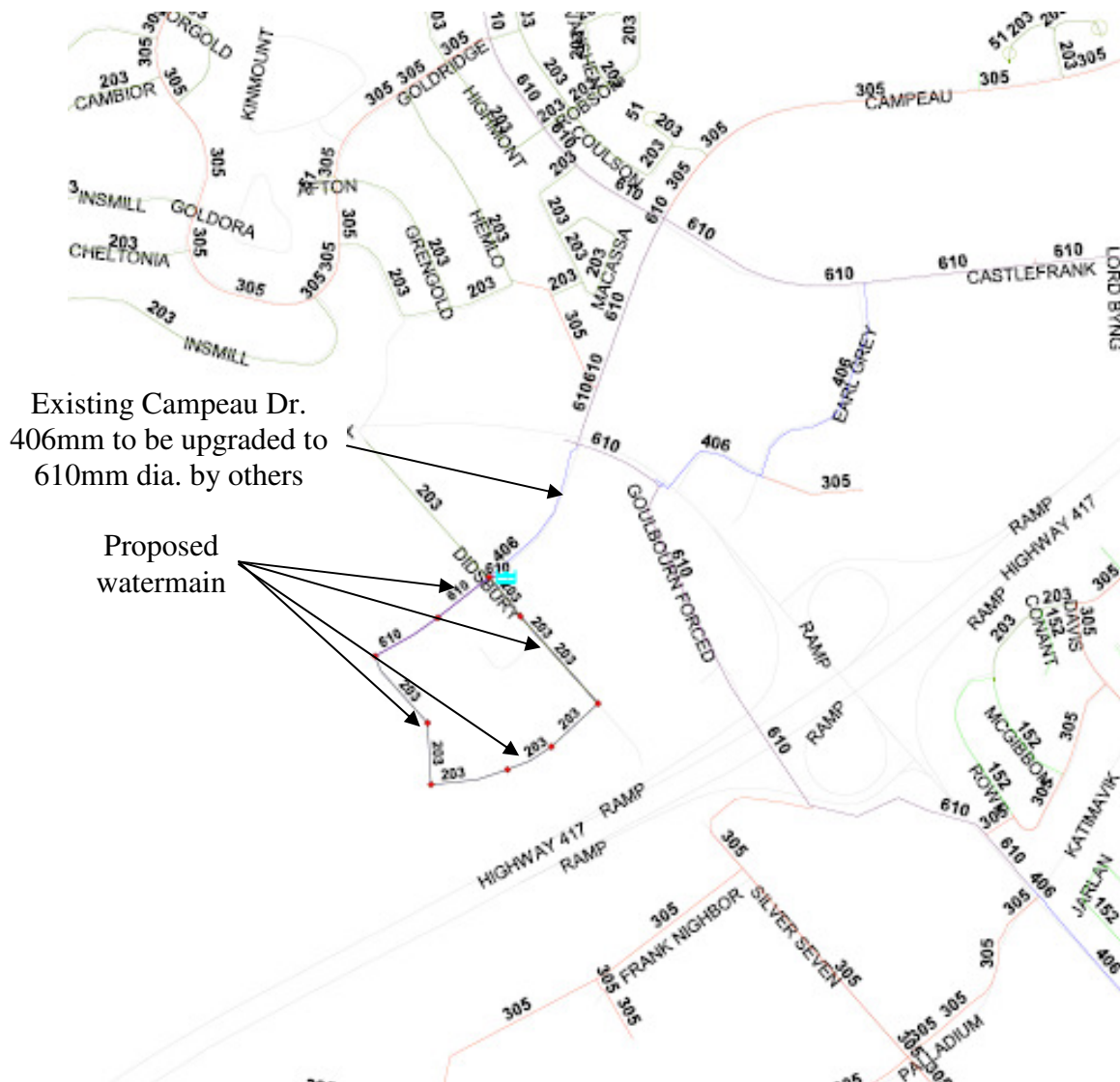
1.4 EXISTING & PROPOSED WATERMAIN NETWORK

As shown in **Figure 1.3**, the proposed subdivision will be serviced from an existing single feed 406mm diameter watermain on Campeau Drive that increases to a 610mm diameter watermain 300m upstream. It is noted that the portion of watermain identified as 406mm diameter is anticipated to be upsized to a 610mm diameter watermain as part of the Campeau Drive redevelopment project being carried out by others. Furthermore, it is noted that the proposed development is located at the eastern boundary of the future Kanata West Community (KWC). As part of and in order to service the KWC, the large diameter watermain on Campeau Drive will be extended to the west towards Huntmar Road where it will loop into a larger north-south

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diameter watermain. As such, the interim single feed is considered to be temporary until the remainder of the KWC is developed in the near future.

Figure 1.3: Existing and proposed watermain network (diameters in mm)



(Note: the watermains shown were created in the previous version of this report and show a diameter of 203mm. The models have been re-run using 204mm as the diameter, as per the 2010 City of Ottawa Water Distribution guidelines).

A hydraulic model was created using H2OMAP software to represent the proposed watermain. Hazen-Williams carrying capacity coefficients were applied to the new watermain in accordance with the July 2010 Watermain Design Guidelines, which state in section 4.2.12 of that document: *the following "C" values shall be used for the design of water distribution systems regardless of pipe materials:*

Pipe Diameter (mm)	C-Factor
150	100
200 to 250	110
300 to 600	120
over 600	130

The Guidelines also specify the actual inside diameters that are to be used when nominal pipe sizes are used:

Pipe Diameters to be used in modeling (in mm):

Nominal Size Allowed (C900 & C905 PVC Pipe)	PVC, DI & C303 ID to be used for Modeling	HDPE DR 11 ID to be used for Modeling
50	50	NA
100	108	92
150	155	136
200	204	177
250	250	220
300	297	261
400	393	328

Pipe diameter for two-foot diameter watermain is not shown, therefore a diameter of 610 mm was used.

As shown in **Figure 1.3**, the proposed water distribution network of the Taggart Loblaws development consists of a 610mm diameter watermain along Campeau Drive that will loop into the KWC in the future along with an internal 204mm diameter looping watermain that will connect to the Campeau Drive watermain in two locations.

1.5 BOUNDARY CONDITIONS

The City of Ottawa provided Stantec the following boundary conditions on June 18th, 2007 which were used in previous submissions of this report. Revised boundary conditions were requested and received by Stantec on February 3, 2012.

The following are revised boundary conditions, HGLs, for hydraulic analysis of the Taggart Development. They can be applied at the end of the 406mm at the corner of Campeau and Didsbury. The fire flow value applied was 15,000Lpm.

- Max_Day + FF = 150.5 m
- Peak_Hour = 155.8 m
- Average Day / High Pressure Check = 163.4 m

1.6 WATER DEMAND

The proposed servicing area covers an area of approximately 13.17ha of commercial lands along with an existing church with an approximate seating capacity of 300 persons. The various consumption rates are estimated using the City of Ottawa design guidelines as discussed below. Note that commercial demands were applied to gross area and not building area as:

For Commercial Shopping Centres, an **average daily demand of 25,000 L/ha/d** is applied to the 13.17ha of proposed commercial/retail lands.

For **maximum day demands a factor of 1.5 times average day** was applied.

For peak hour demands, a **peak demand of 1.8 times maximum day** was applied.

Demands for the existing church were estimated using the Ontario Building Code (OBC) design flow for a church with kitchen facilities (36 litres per seat), see OBC Table 8.2.1.3.B.

Table 1.1 provides a summary of the demands that were applied for the various demand scenarios.

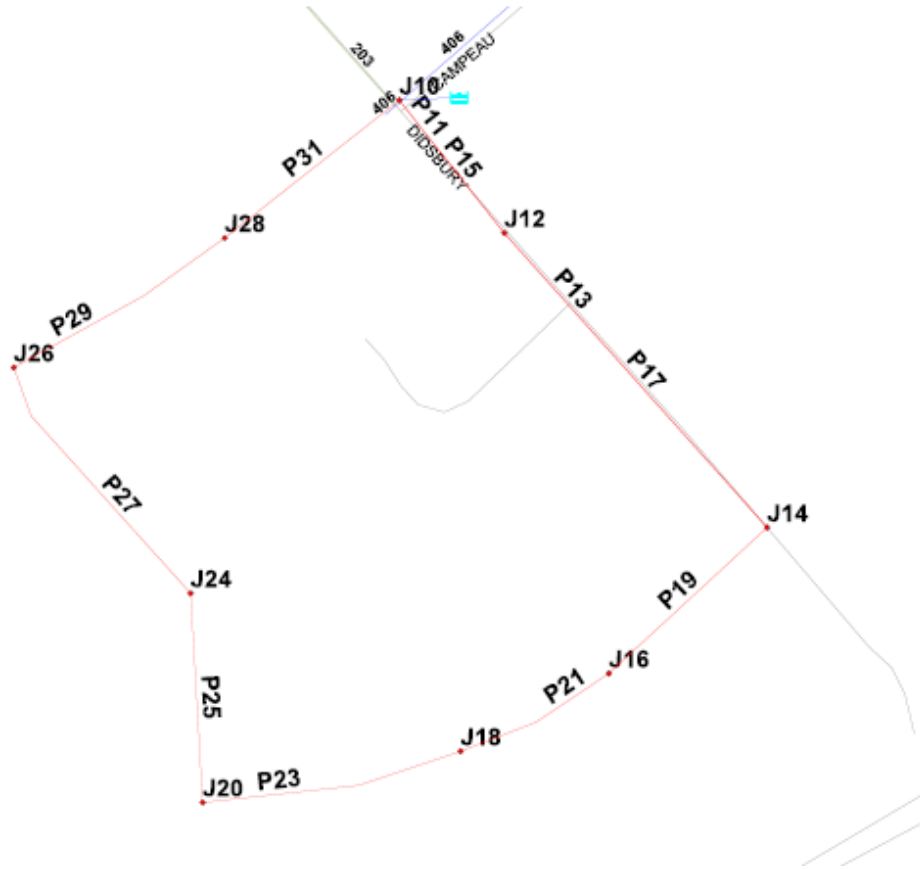
Table 1.1: Demand projections for proposed Taggart - Loblaws development

Service Area	Land Area or # of Seats	Average Day L/s (Lpm)	Maximum Day L/s (Lpm)	Peak Hour L/s (Lpm)
Commercial	13.17ha	3.81 (228.6)	5.72 (343.0)	10.29 (617.3)
Church	300 seats	0.13 (7.5)	0.19 (11.3)	0.34 (20.3)
Total		3.94 (236.1)	5.91 (354.3)	10.63 (637.6)

1.7 JUNCTION AND PIPE IDS

Figure 1.4 provides the IDs of each of the proposed junctions and pipes inputted into the hydraulic model. These IDs are used to present the results in tabular format in the following sections.

Figure 1.4: Junction & Node IDs



1.8 HYDRAULIC MODEL RESULTS

The software package used to carry out the analysis was H₂OMAP Water by MWHSoft. The model was tested under three different demand conditions: average day (AVDY), peak hour (PKHR) and maximum day steady state plus fire.

1.8.1 Average Day Demand – Typical Conditions

The results of the AVDY analysis are presented in **Table 1.2**. The boundary condition used for typical conditions is the same HGL as observed during peak hour (155.8m), which results in local pressures ranging from **557 to 609kPa (81 to 88 psi)** based on the local ground elevations and pipe head loss. While these pressures are within the allowable pressure range of 276kPa to 689kPa (40 to 100 psi) as recommended by the City of Ottawa's (formerly RMOC) design guidelines, it is noted that for services that are expected to exceed 552kPa (80psi), Section 7.6.3.3 of the Ontario Plumbing Code requires a pressure reducing valve (PRV) be installed on each dwelling service to limit maximum pressures. ***Based on the modeling results all services within the Taggart development will require PRVs to meet the Ontario Plumbing Code requirements.***

Table 1.2: Average Day Demand – Typical Conditions Model Output

Average Day Demand - Typical (HGL 155.8m) Node Results

ID	Demand (Lpm)	Elevation (m)	Head (m)	Pressure (kPa)	Pressure (psi)
J10	28.56	95.92	155.80	587	85
J12	28.56	99.00	155.80	557	81
J14	28.56	97.07	155.80	575	83
J16	28.56	96.38	155.80	582	84
J18	7.50	95.63	155.80	590	86
J20	28.56	95.33	155.80	593	86
J24	28.56	93.90	155.80	607	88
J26	28.56	93.66	155.80	609	88
J28	28.56	95.01	155.80	596	86

Average Day Demand - Typical (HGL 155.8m) Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)	Headloss (m)	HL/1000 (m/km)
P11	J10	J12	90	204	110	0.78	0.02	0.00	0.01
P13	J12	J14	210	204	110	0.54	0.02	0.00	0.00
P15	J10	J12	90	204	110	0.78	0.02	0.00	0.01
P17	J12	J14	210	204	110	0.54	0.02	0.00	0.00
P19	J14	J16	114	204	110	0.6	0.02	0.00	0.00
P21	J16	J18	90	204	110	0.13	0	0.00	0.00
P23	J18	J20	141	204	110	0	0.00	0.00	0.00
P25	J20	J24	111	204	110	-0.47	0.01	0.00	0.00
P27	J24	J26	154	204	110	-0.95	0.03	0.00	0.01

TAGGART - LOBLAWS SUBDIVISION - POTABLE WATER SERVICING ANALYSIS

Potable Water Analysis
March 31, 201

Average Day Demand - Typical (HGL 155.8m) Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)	Headloss (m)	HL/1000 (m/km)
P29	J26	J28	132	610	130	-1.43	0	0.00	0.00
P31	J28	J10	118	610	130	-1.9	0.01	0.00	0.00
P33	7000	J10	33	610	130	3.93	0.01	0.00	0.00

1.8.2 Average Day Demand – High Pressure Conditions

The City provided a “high pressure check” boundary condition to be used to verify that maximum pressures did not exceed allowable limits. Using the low demand average day scenario, the high-pressure check was carried out for the proposed development. As shown in the results presented in **Table 1.3**, if the HGL in the development increases to 163.4m, there are a couple of nodes that will experience pressures just under 689kPa (100 psi). Regardless, based on the typical average day conditions, noted in **Section 1.8.1**, pressure reducing valves will be required for each service in order to meet Building Code requirements.

Table 1.3: Average Day Demand – High Pressure Conditions Model Output
Average Day Demand - High Pressure Check (HGL 163.4) Node Results

ID	Demand (Lpm)	Elevation (m)	Head (m)	Pressure (kPa)	Pressure (psi)
J10	28.56	95.92	163.40	661	96
J12	28.56	99.00	163.40	631	92
J14	28.56	97.07	163.40	650	94
J16	28.56	96.38	163.40	657	95
J18	7.50	95.63	163.40	664	96
J20	28.56	95.33	163.40	667	97
J24	28.56	93.90	163.40	681	99
J26	28.56	93.66	163.40	683	99
J28	28.56	95.01	163.40	670	97

Average Day Demand - High Pressure Check (HGL 163.4) Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)	Headloss (m)	HL/1000 (m/km)
P11	J10	J12	90	204	110	0.78	0.02	0.00	0.01
P13	J12	J14	210	204	110	0.54	0.02	0.00	0.00
P15	J10	J12	90	204	110	0.78	0.02	0.00	0.01
P17	J12	J14	210	204	110	0.54	0.02	0.00	0.00
P19	J14	J16	114	204	110	0.6	0.02	0.00	0.00
P21	J16	J18	90	204	110	0.13	0	0.00	0.00
P23	J18	J20	141	204	110	0	0.00	0.00	0.00
P25	J20	J24	111	204	110	-0.47	0.01	0.00	0.00
P27	J24	J26	154	204	110	-0.95	0.03	0.00	0.01
P29	J26	J28	132	610	130	-1.43	0	0.00	0.00
P31	J28	J10	118	610	130	-1.9	0.01	0.00	0.00

TAGGART - LOBLAWS SUBDIVISION - POTABLE WATER SERVICING ANALYSIS

Potable Water Analysis
March 31, 201

Average Day Demand - High Pressure Check (HGL 163.4) Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)	Headloss (m)	HL/1000 (m/km)
P33	7000	J10	33	610	130	3.93	0.01	0.00	0.00

1.8.3 Peak Hour Demand

The results of the PKHR analysis are presented in **Table 1.4**. The boundary condition used for the peak hour condition is (155.8m), which results in local pressures ranging from **557 to 609kPa (81 to 88 psi)** based on the local ground elevations and pipe head loss. These pressures are within the allowable pressure range of 276kPa to 689kPa (40 to 100 psi) as recommended by the City of Ottawa's (formerly RMOC) design guidelines.

Table 1.4: Peak Hour Demand Model Output
Peak Hour Demand - (HGL 155.8m) Node Results

ID	Demand (Lpm)	Elevation (m)	Head (m)	Pressure (kPa)	Pressure (psi)
J10	77.16	95.92	155.80	587	85
J12	77.16	99.00	155.80	557	81
J14	77.16	97.07	155.79	575	83
J16	77.16	96.38	155.79	582	84
J18	20.28	95.63	155.79	590	85
J20	77.16	95.33	155.79	592	86
J24	77.16	93.90	155.79	606	88
J26	77.16	93.66	155.80	609	88
J28	77.16	95.01	155.80	596	86

Peak Hour Demand - (HGL 155.8m) Pipe Results

ID	From Node	To Node	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)	Headloss (m)	HL/1000 (m/km)
P11	J10	J12	90	204	110	2.1	0.06	0.00	0.04
P13	J12	J14	210	204	110	1.46	0.04	0.00	0.02
P15	J10	J12	90	204	110	2.1	0.06	0.00	0.04
P17	J12	J14	210	204	110	1.46	0.04	0.00	0.02
P19	J14	J16	114	204	110	1.63	0.05	0.00	0.03
P21	J16	J18	90	204	110	0.34	0.01	0.00	0.00
P23	J18	J20	141	204	110	0.01	0.00	0.00	0.00
P25	J20	J24	111	204	110	-1.28	0.04	0.00	0.02
P27	J24	J26	154	204	110	-2.57	0.08	0.01	0.06
P29	J26	J28	132	610	130	-3.85	0.01	0.00	0.00
P31	J28	J10	118	610	130	-5.14	0.02	0.00	0.00
P33	7000	J10	33	610	130	10.63	0.04	0.00	0.00

1.8.4 Maximum Day + Fire flow Results

The City's design guidelines for water distribution systems require a minimum pressure of 140 kPa (20 psi) to be maintained at all points in the distribution system under fire flow conditions. For this commercial/retail development, the City is recommending a fire flow of **15,000 Lpm** (250L/s) as a guideline for sizing watermain. A fire flow analysis was carried out using the hydraulic model to determine the distribution network's ability to convey this fire flow under maximum day demand conditions.

The fire flow results in **Table 1.5** show that the minimum required fire flow of 15,000 Lpm can be supplied to all nodes within the proposed local network while maintaining a residual pressure greater than 140kPa (20 psi).

Table 1.5: Maximum Day and Fire Flow Analysis Model Output

Maximum Day Demand and Fire Flow Analysis - (HGL 150.5m) Node Results

ID	Static Demand (Lpm)	Static Pressure (kPa)	Static Head (m)	Fire-Flow Demand (Lpm)	Residual Pressure (kPa)	Residual Pressure (PSI)
J10	42.90	535	150.5	15,000	534	78
J12	42.90	505	150.5	15,000	446	65
J14	42.90	524	150.5	15,000	373	54
J16	42.90	530	150.5	15,000	275	40
J18	11.28	538	150.5	15,000	246	36
J20	42.90	541	150.5	15,000	254	37
J24	42.90	555	150.5	15,000	330	48
J26	42.90	557	150.5	15,000	554	80
J28	42.90	544	150.5	15,000	542	79

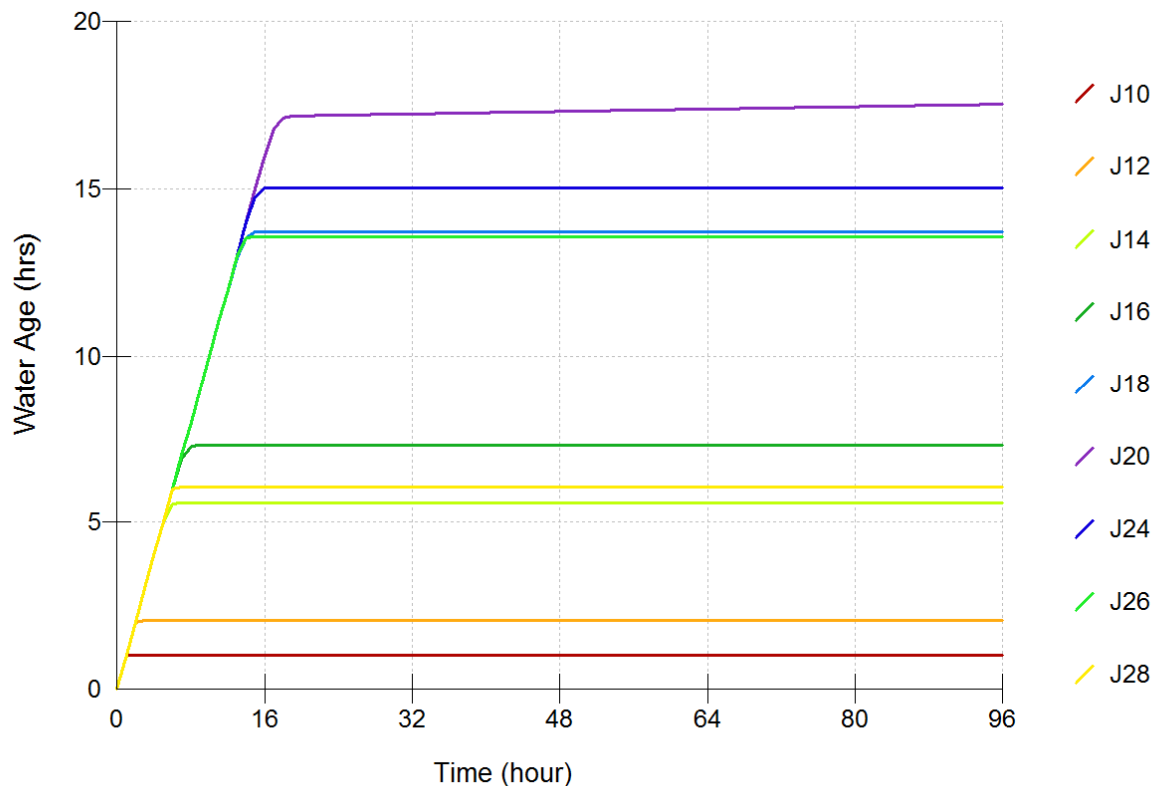
1.9 WATER QUALITY

A water age analysis for the local watermain in the proposed subdivision was conducted.

Figure 1.5 shows that the age of water within the proposed network can reach up to 17 hours based on the projected average day demands. It is noted that the proposed development is anticipated to be fed from a single feed source in the interim until the looping network of the Kanata West Community are in place. Although the local watermain in the Taggart development will be looped to help ensure the flow of water, old water is a possibility. Furthermore, the design guidelines sometimes overestimate actual flows and as a result if lower flows were used in the age of water analysis, an increased age of water would be observed.

It is therefore recommended that in the interim an automatic flushing valve be considered to ensure the turnover of water and help to prevent water quality problems.

Figure 1.5: Water Age Analysis - Taggart Subdivision - Zone 3W, AVDY



1.10 SUMMARY OF FINDINGS

The proposed commercial/retail development is located in an area of the City's water distribution system that has sufficient capacity to provide both the required domestic and emergency fire flows. Based on the modeling results and the boundary conditions provided, **fire flows greater than 15,000 Lpm will be available** within the watermain servicing the proposed development, it is noted that in order to achieve these high flows, a combination of two or more hydrants would likely be required. Following construction and installation of the hydrants, it is recommended that the hydrants be tested in order to ensure the expected fire flows are achievable.

The minimum observed pressure during the peak hour scenario of **557kPa (81 psi)** is considered to be well within the acceptable pressure range.

The maximum observed pressure under typical demand conditions of **609kPa (88psi)** is within the allowable pressure range of 276kPa to 689kPa (40 to 100 psi) as recommended by the City of Ottawa's design guidelines however it does exceed the 550kPa (79.8psi) threshold for maximum pressures, above which the Ontario Plumbing Code requires a pressure reducing valve (PRV) be installed on each dwelling service to limit maximum pressures. ***Based on the modeling results all services within the Taggart development will require PRVs to meet the Ontario Plumbing Code requirements.***

The new development is proposed to be fed from a single feed large diameter watermain in the interim and looped internally. The large diameter watermain along Campeau Drive is expected to become part of a larger looping network to service the Kanata West Community in the near future (assuming develop continues to proceed as anticipated). Once the large diameter loop is completed, the proposed development area will have full redundancy in the event of a pipe failure.

Due to the interim condition of a single feed large diameter watermain terminating in the proposed development, there is a concern that the age of water could result in the inability to maintain appropriate chlorine residuals. Therefore it is recommended that the developer consider installing an automatic flushing valve at an appropriate location in the local network in accordance with the City of Ottawa design guidelines and specifications.

Potable Water Analysis
March 31, 201

All of which is respectfully submitted;

STANTEC CONSULTING LTD.

Neal P. Cody, P.Eng.
Water Resources Engineer
Ph: (613) 724-4380
Fx: (613) 722-2799
neal.cody@stantec.com

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Appendix C

Sanitary Sewer and Signature Ridge Pump Station Background Reports

- **Appendix C.1: Excerpts from *Preliminary Design for the Signature Ridge Pumping Station – Final Report*, R.V.Anderson, January 25, 2011**
- **Appendix C.2: Excerpts from *Signature Ridge Pump Station Hydraulic Grade Line Analysis*, IBI Group, July 2011**
- **Appendix C.3: Excerpts from Kanata West Master Servicing Study, Stantec Consulting Ltd and IBI. Group, June 2006**
- **Appendix C.4: Excerpts from Kanata West Pumping Station Agreement, Soloway Wright, May 18, 2007.**

Appendix C.1

**Excerpts from *Preliminary Design for the Signature Ridge Pumping Station – Final Report*, R.V.Anderson,
January 25, 2011**

Due to the depth required for Option 1, the savings of going through the grassed area may be offset. Option 2 may prove to be a more viable option because it takes a shallower path and it avoids the majority of underground infrastructure as compared to Option 1. Although, the design for Terry Fox and Campeau Drive are currently in progress at the City and actual reconstruction of these roads is anticipated in 2014. With this in mind the options for the second forcemain will need to be investigated further during the detail design to determine the best option. Coordination with the City will be needed during detail design to ensure conflicts are avoided with the other works scheduled on these roads.

4.3 Overflow

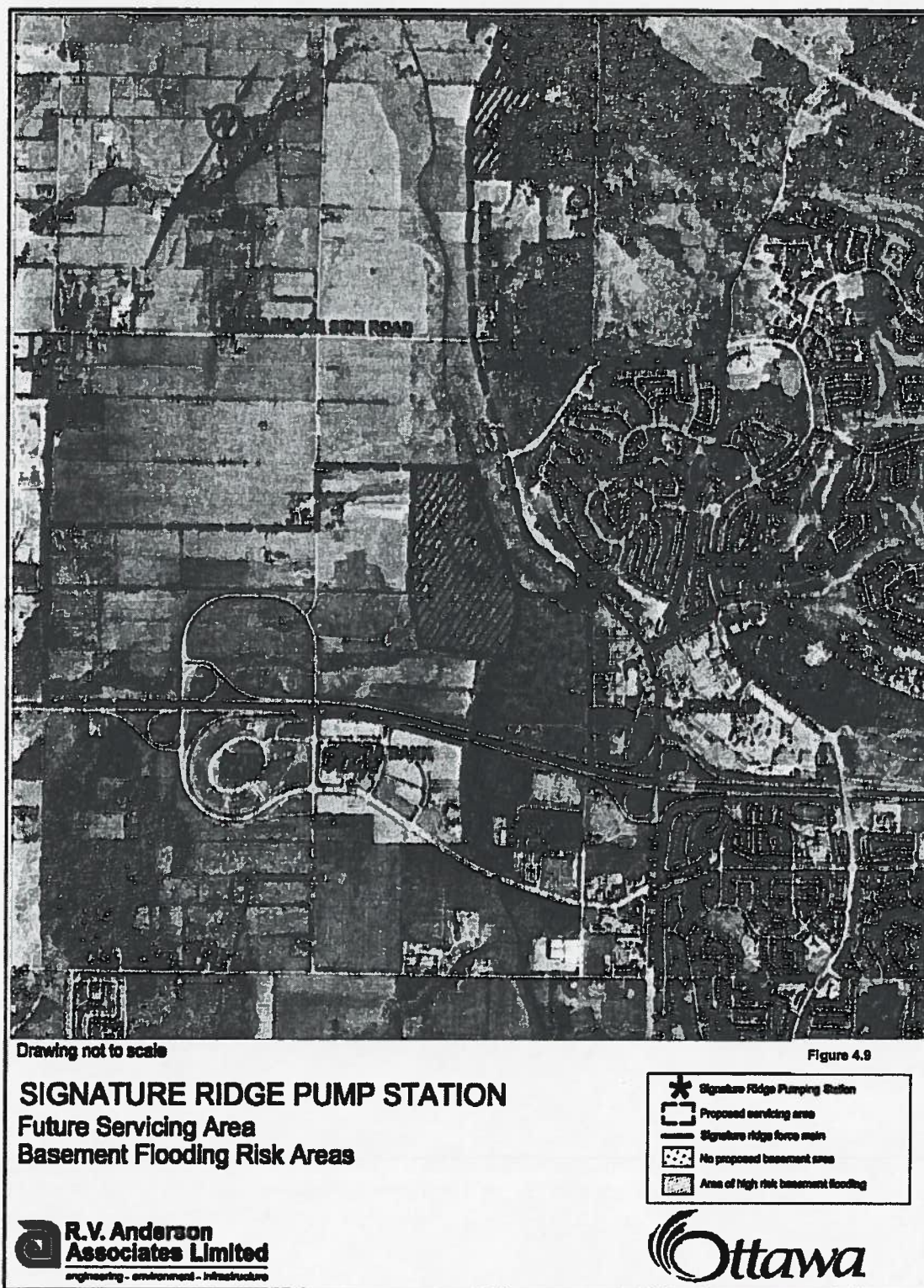
A wastewater overflow at the station would be considered an extremely rare event, but it may be necessary to alleviate conditions within the system that, if not permitted, would result in a threat to human health or property. These conditions could result in failure of mechanical or electrical components, flows in excess of the system design flow criteria caused by extreme weather conditions, or other unusual circumstances. While the existing pump station does not currently have an overflow, a number of components provide system redundancy at the station.

The following provides a description on the station's system redundancy components and the level of protection currently being provided. On review of this information, the station currently has an acceptable level of protection based on the existing flows. The redundancy components consist of:

- Three pumps, one as an emergency standby pump;
- Standby diesel generator;
- Bypass with quick connection for a 150 mm diameter pump;
- Storage available in the wet well and area pipes; and
- No basement below the ground elevation of 95.35 m, therefore overflow to ground first.

The current SRPS facility does not have an overflow. In the event of partial or complete facility failure, surcharging at the pump station can result in the flooding of the wet well building and sanitary discharge to the surrounding area. An overflow to the area stormwater management (SWM) pond could provide an enhanced level of protection. At present, the station does not have an emergency overflow but has a bypass chamber located on the north side of the wet well with a 150 mm quick connect that can be used to divert flow when necessary. The ability to protect the station with the quick connection during an extended full station failure will become very limited at peak flows above 100 L/s.

Nearby SWM ponds exist in the area with the nearest one located approximately 72 m to the west of SRPS. The advantage of this option is that the SWM pond can provide some form of treatment and contain the overflow. The pond's exact location and elevation were identified in order to assess the possibility to convey the sanitary sewerage overflow from SRPS to it. Currently the SWM pond has a permanent water level of 92.3 m. As this pond may be located within the Carp River flood plain, its potential use as an emergency spill location will have to consider the final 1:100 year water level of the Carp River, which is 93.67 m. See Carp River Level data in **Appendix E**.



An overflow into the pond would result in a flow into the downstream creek, but the quality of flow should be significantly better than direct discharge to a surface stream. A review of topographical drawings of the study area suggests the lowest basement floor elevation tributary to the local sewers connected to SRPS is currently in the range of 96.8 m (for existing development). Review of projected future developments with the land developers identified two areas of concern as the basement floor elevation would be below the proposed overflow elevation of 93.7 m. **Figure 4.9**, on the previous page, shows these two areas. These areas are part of the Interstitial and Minto developments. If the overflow is not built, additional lands would fall within the area of basement flooding risk. In discussion with the developers, the Interstitial Lands may have basement elevations in the 93 m zone, while the Minto Lands are proposing the lowest basement elevation around 93.4 m.

It must be noted that even with an overflow in place at the SRPS, some parts of the Minto and Interstitial Lands will not be protected in the event of a full system failure. Figure 4.9, on the previous page, shows the two areas in general. The developers should review these lands to determine if the lot grading can be adjusted to eliminate or minimize the zone of influence for basement flooding. It must also be noted that overflows implemented by developers may not work to protect against basement flooding.

The permanent water level in the pond will allow an overflow by gravity from the pump station into it without the risk of flooding the basements of adjacent residences. Drawings provided by IBI Group in May 2009 of the Stormwater Management Pond (Drawing 3793-LD-SW1, June 1994) show the top of the berm for this pond at 93.60. Any event above this elevation would outlet into the surrounding low lands.

Proposed Overflow

There are several options available for the exact layout of the new overflow. The proposed overflow options are shown in the attached **Appendix G**. The different layouts are described below.

The first option, Option A in **Appendix G**, would have a 600 mm pipe run approximately 29 m to a new 1500 mm manhole structure where the backflow preventer and valve would be located. An additional manhole structure would be located at the bend. The disadvantage of this option would be that the manhole structure at the bend would be above grade due to the shallow pipe. A 750 mm concrete pipe will discharge from the new manhole structure to the nearby SWM pond.

The second option, Option B in **Appendix G**, would have a 600 mm pipe run approximately 19 m to a new 1500 mm manhole structure where the backflow preventer and valve would be located. The disadvantage of this option would be that the new manhole structure would be above grade due to the shallow pipe. A 750 mm concrete pipe will discharge from the new manhole structure to the existing 2400 mm storm pipe.

The third option, Option C in **Appendix G**, would have a 600 mm pipe run approximately 9 m to a new 1500 mm manhole structure where the backflow preventer and valve would be located. This option proves advantageous as the new manhole structure would be off the road allowance. A 750 mm concrete pipe will discharge from the new manhole structure to the nearby SWM pond.

The fourth option, Option D in **Appendix G**, would have a 600 mm pipe run approximately 9 m to a new 1500 mm manhole structure where the backflow preventer and valve would be located. This option proves advantageous as the new manhole structure would be off the road allowance. A 750 mm concrete pipe will discharge from the new manhole structure to the second manhole and then finally to the existing 2400 mm storm pipe.

For all the options described above, Options A through D, an overflow will be connected to the existing storm manhole structure outside the wet well building; a 600 mm pipe will be installed discharging to a new 1500 mm circular manhole structure. The new overflow piping will be a concrete pipe to allow for the road loads. The discharging 600 mm pipe into the new manhole structure will have a WaStop backflow valve (see **Appendix F** for more details on the WaStop) added to the line as well as a shut off valve to manually close in the event that the WaStop cannot prevent backflow in an extremely rare storm event. The shut off valve will need further investigation as to the specific type, such as a knife gate valve or a plug valve. The knife gate valve has the advantage of being shorter in length providing more room in the manhole structure although it is a tall valve which may result in the manhole structure extending above grade, see **Appendix F** for a drawing of the DeZurik knife gate valve. The plug valve has the advantage of being shorter in height although having a longer body, see **Appendix F** for a drawing of the DeZurik plug valve. For all the options described above, Options A through D, the area, including the road near the pump station, will require re-grading to accommodate the shallow pipes and in most areas up to 0.5 m of additional fill will be needed. The proposed grade modifications are shown in the attached **Appendix G**.

During a December 2010 meeting the City brought forward concerns with the overflow options. As a result an additional option was developed to address the City's concerns. This final option, Option E in **Appendix G**, would have a 750 mm pipe run approximately 3 m from the existing storm manhole structure outside the wet well building to a new 1500 mm manhole structure within the SRPS property. This option proves advantageous as the new manhole structure would be off the road allowance. The new overflow piping will be a concrete pipe to allow for the road loads. The discharging 750 mm pipe into the new manhole structure will have a WaStop backflow valve (see **Appendix F** for more details on the WaStop) added to the line as well as a shut off valve to manually close in the event that the WaStop cannot prevent backflow in an extremely rare storm event. The shut off valve will need further investigation as to the specific type, such as a knife gate valve or a plug valve. The knife gate valve has the advantage of being shorter in length providing more room in the manhole structure although it is a tall valve which may result in the manhole structure extending above grade, see **Appendix F** for a drawing of the DeZurik knife gate valve. The plug valve has the advantage of being shorter in height although having a longer body, see **Appendix F** for a drawing of the DeZurik plug valve. A 750 mm concrete pipe will run approximately 23.5 m from the new manhole structure and discharge to the nearby SWM pond. Option E will only require minor re-grading in the swale to accommodate the shallow pipes. The proposed grade modifications are shown in the attached **Appendix G**. As discussed in December 2010 meeting with the City Option E is the preferred and recommended overflow solution.

In all options, which include Options A through E, the outlet of the overflow will be at elevation 93.70 m. This elevation is above the 1:100 year storm of the SWM pond which is 93.60 m and above the 1:100 year storm of the Carp River which is 93.67 m.

The local developers should evaluate the impact of the proposed overflow elevation and their proposed basement elevations. With a number of storm water ponds being

proposed, the developers should evaluate overflows to the storm water ponds to protect the sanitary systems in the event of an extended full station failure. It must also be noted that overflows implemented by developers may not work to protect against basement flooding.

5.0 AREAS REQUIRING SPECIAL CONSIDERATION

This section is not intended to fully address all issues, but merely document those that may have an impact on the final design of the proposed infrastructure.

Kanata West Servicing Plan

As a result of the present assessment of the SRPS, it is recommended that the Kanata West Servicing Plan be updated to reflect these findings. It is also recommended that the current program of flow monitoring be continued and that scheduled upgrades and improvements of the SRPS be implemented on the basis of the flow monitoring. It must be noted that the developers for the Minto and interstitial lands should review their proposed lowest basement elevations, as elevations below about 94.5 m cannot be protected from basement flooding in an extended complete system failure.

The Kanata West Servicing Plan identifies the need to upsize the incoming pipe at the SRPS from the existing 600 mm diameter to a 750 mm diameter.

Hydraulic Consideration

It is recommended that the City carry out monitoring of both the flows in the collection system and the associated level of development over an extended period of time to ensure appropriate timing of the recommended infrastructure. The hydraulic assessment conducted on each of the options and the resulting infrastructure phasing and design must be considered in light of the City's "just in time" strategy. Furthermore, the level of uncertainty with respect to future population and land use projection in the Kanata area makes monitoring of the existing flows and development rates critical to ensuring that capacity issues are not unforeseen or unexpected, and that the City maintains the desired level of service.

The flows considered in this report reflect design values based on per capita demands and land use conditions. Other master planning studies are underway that will consider a combination of design values and monitored flow values. These results will be incorporated in the Infrastructure Master Plan update expected in 2011. This study considered that the maximum estimated peak flows were higher than actual monitoring results for two pumping stations. Hazeldean Pumping Station (maximum observed on record +/- 800 L/s) and March Pumping Station (with the exception of Sept. 9, 2004) where the maximum observed is approximately 400 L/s and with regards to the SRPS, maximum observed is about 80 L/s. Even though the estimated design numbers show the stations are over capacity, flow-monitoring results indicate otherwise.

Geotechnical Investigation

Soil issues are known at the SRPS site. A detailed geotechnical investigation will be required at the site to identify water and soils issues, and to propose any design or construction



Figure 2.1

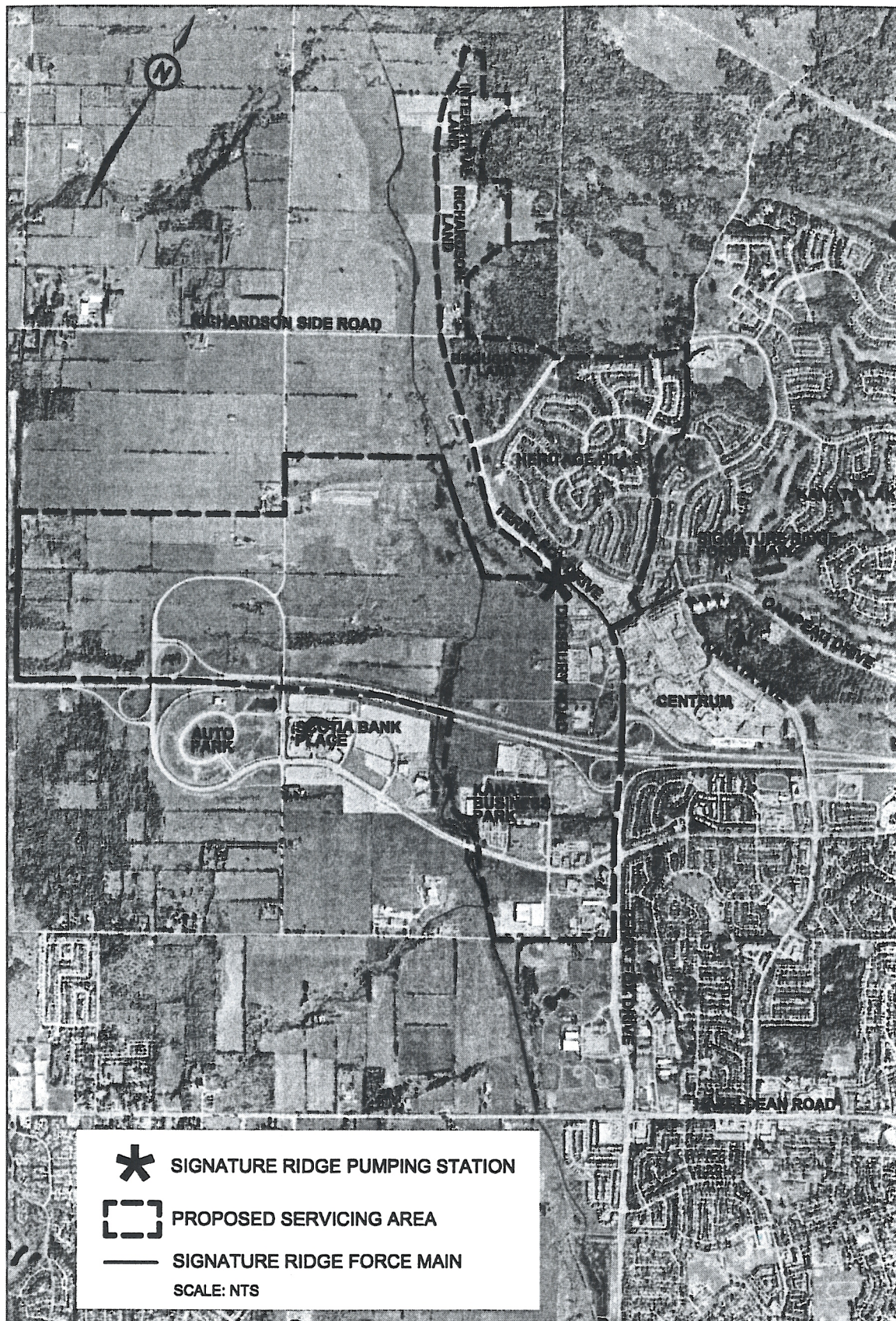


Figure 2.2

Appendix C.2

**Excerpts from *Signature Ridge Pump Station
Hydraulic Grade Line Analysis*, IBI Group, July 2011**

2. EMERGENCY OVERFLOW ANALYSIS

As part of the 2008 report, Stantec revised the original XPSWMM models used for the Kanata West MSS and the Master Servicing Sanitary Plan for the Interstitial Lands (Stantec, December 2007). The revised model included existing sanitary sewer information within the Heritage Hills community and preliminary design of sanitary sewers within the Arcadia development. This model was used to assess potential emergency overflow alignments and impacts within the sanitary sewer network tributary to the SRPS.

Currently, the XPSWMM model has been obtained from Stantec and updated to reflect the revised servicing strategy presented within the Functional Design report by RV Anderson. The model has also been updated to reflect current sanitary design information available from the interim and ultimate developments within Arcadia, and Terry Fox Dr sanitary sewer servicing the Baylis, Richardson Ridge and Broughton lands.

2.1 Interim vs. Ultimate Development Scenario

Not correct.

As the development areas tributary to the SRPS will be constructed in phases, the sanitary hydraulic grade line was evaluated using an interim and ultimate development scenario. This approach will ensure that the function of the interim and ultimate overflows will provide sufficient flooding protection as the development areas proceed to full build out. Details of the development scenarios are further outlined within the following sections.

2.1.1 INTERIM DEVELOPMENT CONDITION

The Interim development scenario is illustrated within the attached Figure 1. A summary of the tributary drainage areas and emergency sanitary overflow locations is provided below:

- Phases I and II of the Arcadia Development
- Commercial lands along Didsbury Rd. North of HWY417
- Terry Fox and Kanata Business Parks, and Scotia Bank Place south of HWY417
- Baylis, Richardson and Broughton Lands
- Existing Heritage Hills Community

The below table summarizes the sanitary drainage areas and populations for the interim development condition:

Table 1: Interim Development Drainage Areas and Populations

Tributary Lands	Residential Area (ha)	Cmrl/Instnl/Indstl Area (ha)	Population
Baylis	2.26	0	199
Richardson	32.98	0	2249
Broughton	17.72	1.2	1106
Kanata West (North of HWY417)	31.20	45.85	2570
Heritage Hills	81.52	4.88	3437
Kanata West (South of HWY417)	0	102.31	0
TOTALS	165.68	154.24	9561

The Interim development scenario includes the following emergency sanitary overflow locations:

- Proposed sanitary overflow at Arcadia Interim SWM Facility (100 year w/l 94.10m)
- Proposed sanitary overflow at Richardson Ridge Interim SWM Facility (100 year w/l 93.83m)
- Proposed sanitary overflow at Broughton SWM Facility (100 year w/l 93.60m)

2.1.2 ULTIMATE DEVELOPMENT CONDITION

The Ultimate development scenario is illustrated within the attached Figure 2. A summary of the tributary drainage areas and emergency sanitary overflow locations is provided below:

- Kanata West Business Park west of Huntmar Rd.
- Full build out of the Arcadia Development
- Commercial lands south of future Campeau Dr
- Commercial lands along Didsbury Rd., north of HWY417
- Terry Fox and Kanata Business Parks south of HWY417
- Baylis, Richardson and Broughton Lands
- Existing Heritage Hills Community

The below table summarizes the sanitary drainage areas and populations for the ultimate development condition:

Table 2: Ultimate Development Scenario – Drainage Area and Population

Lands	Residential Area (ha)	Coml/Instnl/Indstl Area (ha)	Population
Baylis	2.26	0	199
Richardson	32.98	0	2249
Broughton	17.72	1.2	1106
Kanata West (North of HWY417)	55.98	173.75	4215
Heritage Hills	81.52	4.88	3437
Kanata West (South of HWY417)	0	76.11	0
TOTAL	190.46	255.94	11206

The Ultimate development scenario includes the following emergency sanitary overflow locations:

- Proposed sanitary overflow at Arcadia Ultimate SWM Facility (100 year w/l 93.75m)
- Proposed sanitary overflow at Richardson Ridge SWM Facility (100 year w/l of 93.70m)
- Proposed sanitary overflow at Broughton SWM Facility (100 year w/l of 93.60m)

3. SANITARY FLOW CALCULATION AND BACKGROUND INFORMATION

Sanitary design flows for the tributary drainage areas have been calculated in accordance with the report "Design Flow Verification – Signature Ridge Pumping Station" (CH2MHILL, 2010) within Appendix D of the RV Anderson "Functional Design Report." Sewer design spreadsheets used for the HGL analysis are provided within Appendix A and B for the interim and ultimate development condition, respectively. The parameters used for the calculation are summarized as follows:

- Residential average flow – 350 Lpcd
- Commercial/Institutional – 50,000 L/ha/d
- Light Industrial average flow – 35,000 L/ha/d
- Residential peaking factor – Harmon formula with K=1.0
- Commercial / institutional /Industrial peaking factor – 1.5
- Infiltration/Inflow – 0.5 L/s/ha (as per City of Ottawa request)
- Population densities as per City of Ottawa Sewer Design Guidelines (2004):

Minto Communities Inc.
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- o Single Family home = 3.4 people/unit
- o Multi-Family Home = 2.7 people/unit

In addition to the above sanitary design flow criteria, the following background and supporting information was used to calculate sanitary flows within the system:

- Existing sewer information and unit counts in the Heritage Hills community was obtained from Stantec.
- Sanitary sewer information and proposed ground elevations along Didsbury Road were obtained from Stantec.
- Servicing details for the Baylis, Richardson and Broughton Lands were available in house from other studies

*are these
consistent
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RU
Report*

3.1 Carp River Water Levels

The following table outlines the Carp River 100 year water levels used for the preliminary design of the proposed sanitary overflows, and compares the calibrated model existing and future conditions water levels.

Carp River 100 Year Water Levels

Location	Carp River Water Levels (m)		
	Used For Design	Calibrated Existing Conditions	Calibrated Future Conditions
Signature Ridge	93.67 (As per R.V. Anderson Report)	93.58*	93.45*
Arcadia	93.63	93.46	93.41
Richardson Ridge	93.53	93.12	93.16

*Interpolated

Storm hydraulic grade line elevations are indicated within Figures 3 through 5.

3.2 Extraneous Inflow Through Sanitary Manhole Covers

The Sewer Design Guidelines clause 4.4.1.4 indicates extraneous inflows through cover vent holes for manholes located within low points should be quantified and the system analyzed to assess the impacts of the excess inflow through the cover vent holes. The sanitary sewer system has been evaluated with additional flows introduced to the system (stormwater inflow through the sanitary manhole covers). Details of the extraneous inflow are provided below.

The sanitary sewershed consists of existing residential development and proposed commercial and residential development. For the proposed Arcadia Interim development area, sanitary manholes located within low points (where on-site surface storage is located) were identified and inflow rates quantified. For new development, the current City of Ottawa Standard sanitary sewer manhole cover includes one 12mm diameter hole. The total extraneous inflow within the Arcadia Interim development was used to determine an average inflow rate for all new developments within the sewershed. Inflows have been added to the sewer design spreadsheet and corresponding XPSWMM model, and calculations of ponding depths and inflow rates are provided within Appendix C.

For the existing Heritage Hills residential area, a field survey of the existing Heritage Hills residential area was performed to estimate relative ponding depths at manholes within low points in order to calculate inflows. The manhole covers within the Heritage Hills area are a mixture of covers with two square holes and the previous City Standard eight round holes. The resulting extraneous inflows have been added to the

sewer design spreadsheet and corresponding XPSWMM model, and calculations are provided within Appendix C.

4. PRELIMINARY OVERFLOW DESIGN

The overflow alternatives examined within the 2008 Stantec study have been further analyzed and refined within this report. The following sections outline the proposed sanitary overflow locations and preliminary design details.

4.1 Arcadia Overflow at Interim SWM Facility

The sanitary overflow into the interim Arcadia SWM facility will be comprised of a 450mm pipe connection from the sanitary system to the storm sewer system. The pipe will be approximately 3m in length with reverse slope at -0.1%. The upstream invert elevation will be set at 94.24m and the downstream elevation will be 94.29m, which is approximately equal to the storm obvert at the manhole. Details of the overflow connection are illustrated within Figure 3.

4.2 Arcadia Overflow at Ultimate SWM Facility

Sanitary overflow outlet into the ultimate Arcadia SWM facility will be comprised of a 900x900mm box pipe connected to the storm sewer system via a control manhole. The control manhole will be constructed with a control weir with crest elevation 93.80m, approximately equal to the storm HGL. The pipe will be approximately 3m in length at 3.33% slope. The upstream invert elevation will be set at 92.80m and the downstream elevation will be 92.70. Details of the overflow connection are illustrated within Figure 4.

4.3 Richardson Ridge Overflow at Ultimate SWM Facility

The sanitary overflow outlet into the Richardson Ridge SWM facility will be comprised of a 450mm pipe connection from the sanitary system to the storm system. The pipe will be approximately 4m in length at 1.0% slope. The upstream invert elevation will be set at 94.14m and the downstream elevation will be 94.10m. Details of the overflow connection are illustrated within Figure 5.

4.4 SPRS Overflow at Broughton SWM Facility

The sanitary overflow from the Signature Ridge Pump Station to the Broughton SWM facility was proposed within the "Preliminary Design for the Signature Ridge Pumping Station, Final Report" (January 2011). The preferred alternative (identified as Option E within that report) consists of a 750mm pipe run approximately 3m from the existing storm manhole structure outside the wet well building to a new 1500mm manhole structure within the SRPS property. The 750mm pipe will run approximately 23.5m from the new manhole structure and discharge to the nearby SWM pond. The outlet of the overflow will be at elevation 93.70m. This elevation is above the 1:100 year storm of the SWM pond which is 93.60m, and the Carp River 93.67m (As per the RVA Report). For reference, the preliminary plan and profile for Option E (DWG023) is provided along with the figures within this report.

To prevent stormwater flows from backing up and spilling into the sanitary collection system, it is proposed to install backflow preventer valves at each of the overflow locations.

5. HYDRAULIC GRADE LINE RESULTS

Underside of footing and underside of slab elevations have been estimated for each node within the XPSWMM model. For estimation purposes, typical underside of footing elevations for residential construction is approximated at 2.4m below adjacent proposed road grades. Also for estimation purposes, typical underside of slab elevations for commercial construction is approximated at 0.5m above proposed road grades (or 0.55m below finished floor elevations, if available).

5.1 Interim Development Condition

Results of the Interim development condition XPSWMM model are summarized within the below table, and the XPSWMM output file is provided within Appendix A. An electronic copy of the model is provided within Appendix D.

Table 3 – Interim Development Condition HGL Results

COMMUNITY	NODE	Proposed Ground (m)	Estimated U.S.F. (m)	Sanitary HGL (m)	Freeboard (m)
Campeau Dr	2 (Huntmar Dr)	100.21	96.49	94.60	1.89
	2B	97.82	95.14	94.09	1.05
	3	94.88	94.70	94.07	0.63
	4	94.86	N/A	94.06	n/a
	5	96.00	93.65 (Stantec)	94.05	-0.40
	12465	95.70	95.70	94.05	1.65
South of HWY 417	15	97.60	97.60	95.68	1.92
Didsbury	5E	94.20	94.70	94.00	0.70
	5D	94.00	94.50	93.96	0.54
	5C	94.50	95.00	93.93	1.07
	5A	95.08	95.58	93.91	1.67
Arcadia	S2	99.00	96.25	95.27	0.98
	S1	100.00	96.78	95.41	1.37
	S5	97.00	95.18	94.20	0.98
	S5B	96.82	95.16	94.18	0.98
	S6	97.24	95.14	94.16	0.98
Heritage Hills	20116	107.90	105.50	103.87	1.63
	20164	107.30	104.90	101.67	3.23
	12735	102.10	99.70	96.39	3.31
	12732	99.40	97.00	95.47	1.53
	20098	106.80	104.40	104.33	0.07
	20179	106.60	104.20	103.69	0.51
	20123	106.10	103.70	103.43	0.27
	20127	105.40	103.00	101.72	1.28
	20130	102.90	100.50	99.31	1.19
	13058	98.00	96.80 (RVA report)	95.32	1.48
	20161	98.00	96.80 (RVA report)	95.03	1.77
Terry Fox Drive	Baylis	97.15	95.03	94.88	0.15
	Rchrdsn N.	96.00	94.97	94.82	0.15
Richardson Ridge	L. Rchrdsn	95.57	94.90	94.29	0.61
	MH329A	97.00	94.68	94.31	0.37
	MH328A	98.77	94.74	94.32	0.42

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COMMUNITY	NODE	Proposed Ground (m)	Estimated U.S.F. (m)	Sanitary HGL (m)	Freeboard (m)
	L. Rchrdsn Ea.	95.61	94.59	94.29	0.30
Broughton	N62	95.57	95.56	94.29	1.27
	Broughton	96.19	96.10 (RVA report)	94.19	1.91
	MH205	97.01	94.61	94.19	0.42
	MH207	98.54	97.26	94.79	2.47
Terry Fox Drive (Broughton to SRPS)	TBD	95.42	n/a	94.13	n/a
	SRPS	95.35	95.50 (wet well floor elevation)	93.91	1.58

5.1.1 DISCUSSION OF RESULTS AND PROPOSED MITIGATION MEASURES

The above results indicate that the hydraulic grade line is at least 0.3m below the proposed underside of footing elevation within the Arcadia development area. It should be noted that for the interim condition there will be no development constructed adjacent to Nodes 3 and 4 along future Campeau Dr, and therefore no impacts on the MUSF elevations resulting from the HGL elevation in this area.

The results indicate that the HGL will be at least 0.3m below proposed USF elevations within the Richardson Ridge development area (MH328A and MH329A). Also, the HGL will be at least 0.3m below the proposed USF elevations within the Broughton development area (MH205 and MH207). The minimum USF elevations in the Baylis and Richardson North lands are unknown at the present time, and will need to be confirmed as part of individual subdivision design.

Node 20098, the uppermost node in the Heritage Hills subdivision, has a sanitary HGL which is closer than 0.3m to the USF elevation. This is due to the elevation of the existing sewer, and not because of backwater effect from the downstream pump station or the proposed overflow locations.

Node 20123 has a sanitary HGL which is slightly closer than 0.3 below the estimated underside of footing elevation. This manhole is upstream of a short section of 200mm diameter pipe, which causes a localized spike in HGL. This is due to the limited capacity of the smaller diameter pipe, and not because of backwater effect from the downstream pump station or the proposed overflow locations.

Development adjacent to Node 5 is proposed to be commercial with standard slab on grade construction with no basements. Preliminary grading plans for the area indicate a lowest finished floor elevation of 94.20m. The HGL in this area is approximately 94.05m. It is recommended that the grading for this area be revised to provide 0.3m between USF and HGL and measures such as backflow valves be installed for the commercial buildings.

The results indicate that the HGL is just below proposed ground elevation along Didsbury Rd. between the SRPS and Campeau Dr. Proposed ground elevations in this area are 94.00m to 95.08m. As part of the initial XPSWMM model set-up by Stantec, these nodes were sealed to prevent volume loss from the system. Development along Didsbury Road is expected to be commercial with standard slab on grade construction. When constructed, it is recommended that the manholes along Didsbury Road, between the SRPS and Campeau Drive, be outfitted with sealed covers to prevent sanitary flooding during overflow events and that the road profile be raised to ensure that 0.3m freeboard below the underside of slab elevations is provided. Backwater valves should also be installed on the service laterals of the commercial buildings.

5.2 Ultimate Development Condition

Results of the Ultimate development condition XPSWMM model are summarized within the below table, and the XPSWMM output file is provided within Appendix B. An electronic copy of the model is provided within Appendix D.

Table 4 – Ultimate Development Condition HGL Results

COMMUNITY	NODE	Proposed Ground (m)	Estimated U.S.F. (m)	Sanitary HGL (m)	Freeboard (m)
Campeau Dr Extension	1 (Palladium Dr.)	103.50	104.00	99.56	4.44
	2 (Huntmar Dr.)	100.21	96.49	95.73	0.76
	2B	97.82	95.14	94.70	0.44
	14	99.50	100.00	94.54	5.46
	14A	96.00	96.50	94.38	2.12
	3	94.88	94.70	94.27	0.43
	4	94.86	n/a	94.21	n/a
	5	96.00	93.65 (Stantec)	94.12	-0.47
	12465	95.70	95.70	94.12	1.58
South of 417	15	97.60	97.60	94.66	2.94
Didsbury	5E	94.20	94.70	94.05	
	5D	94.00	94.50	94.00	
	5C	94.50	95.00	93.94	
	5A	95.08	95.58	93.92	
Arcadia	S2	99.80	96.25	95.29	0.96
	S1	100.00	96.78	95.42	1.36
	S5	97.00	95.18	94.81	0.37
	S5B	96.82	95.16	94.79	0.37
	S6	97.24	95.14	94.77	0.37
	S4	96.40	94.40	93.92	0.48
	S3	96.88	94.88	94.14	0.74
Heritage Hills	20116	107.90	105.50	103.87	1.63
	20164	107.30	104.90	101.67	3.23
	12735	102.10	99.70	96.39	3.31
	12732	99.40	97.00	95.48	1.52
	20098	106.80	104.40	104.33	0.07
	20179	106.60	104.20	103.69	0.51
	20123	106.10	103.70	103.43	0.27
	20127	105.40	103.00	101.72	1.28
	20130	102.90	100.50	99.31	1.19
	13058	98.00	96.80 (RVA report)	95.33	1.47
	20161	98.00	96.80 (RVA report)	95.04	1.76
Terry Fox Drive	Baylis	97.15	95.24	94.88	0.36
	Rchrdsn N.	96.00	94.95	95.25	0.30
Richardson Ridge	L. Rchrdsn	95.57	94.90	94.60	0.30
	MH329A	97.00	94.68	94.31	0.37
	MH328A	98.77	94.74	94.32	0.42
	L. Rchrdsn Ea.	95.61	94.59	94.29	0.30
Broughton	N62	95.57	95.56	94.29	1.27
	Broughton	96.19	96.10	94.19	1.91

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HYDRAULIC GRADE LINE ANALYSIS**

COMMUNITY	NODE	Proposed Ground (m)	Estimated U.S.F. (m)	Sanitary HGL (m)	Freeboard (m)
			(RVA report)		
	MH205	97.01	94.61	94.19	0.42
	MH207	98.54	97.26	94.79	2.47
Terry Fox Drive (Broughton to SRPS)	TBD	95.42	n/a	94.14	n/a
	SPRS	95.35	95.50 (wet well floor elevation)	93.92	1.58

5.2.1 DISCUSSION OF RESULTS AND PROPOSED MITIGATION MEASURES

The above results indicate that the hydraulic grade line is 0.3m below the proposed underside of footing elevations within the Arcadia development area.

The results indicate that the HGL will be at least 0.3m below proposed USF elevations within the Richardson Ridge development area (MH328A and MH329A). Also, the HGL will be at least 0.3m below proposed USF elevations within the Broughton development area (MH205 and MH207). The minimum USF elevations in the Baylis lands are unknown at the present time, and will need to be confirmed as part of individual subdivision design.

Node 20098, the uppermost node in the Heritage Hills subdivision, has a sanitary HGL which is closer than 0.3m to the USF elevation. This is due to the elevation of the existing sewer, and not because of backwater effect from the downstream pump station or the proposed overflow locations.

Node 20123 has a sanitary HGL which is slightly closer than 0.3 below the estimated underside of footing elevation. This manhole is upstream of a short section of 200mm diameter pipe, which causes a localized spike in HGL. This is due to the limited capacity of the smaller diameter pipe, and not because of backwater effect from the downstream pump station or the proposed overflow locations.

Development adjacent to Node 5 is proposed to be commercial with standard slab on grade construction with no basements. Preliminary grading plans for the area indicate a lowest finished floor elevation of 94.20m. The HGL in this area is approximately 94.12m. It is recommended that the grading for this area be revised to provide 0.3m between USF and HGL and measures such as backflow valves be installed for the commercial buildings.

The results indicate that the HGL is at or just below proposed ground elevation along Didsbury Road between the SRPS and Campeau Drive. Proposed ground elevations in this area are 94.00m to 95.08m. As part of the initial XPSWMM model set-up by Stantec, these nodes were 'sealed' to prevent volume loss from the system. Development along Didsbury Road is expected to be commercial with standard slab on grade construction. When constructed, it is recommended that the manholes along Didsbury Road, between the SRPS and Campeau Drive be outfitted with sealed covers to prevent sanitary flooding during overflow events and that the road profile be raised as required to ensure that 0.3m freeboard below the underside of slab elevations is provided. Backwater valves should also be installed on the service laterals of the commercial buildings.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Arcadia

6.1.1 INTERIM DEVELOPMENT

An emergency sanitary overflow will be installed to provide flooding protection for the interim Arcadia development. The overflow will be comprised of a 450mm diameter pipe connection from the sanitary system to the storm system adjacent to the proposed interim stormwater management facility. The location of the proposed overflow is illustrated within Figure 1 and details of the overflow connection are illustrated within Figure 3.

6.1.2 ULTIMATE DEVELOPMENT

An emergency sanitary overflow will be installed to provide flooding protection for the ultimate development. The overflow will be comprised of a 900x900 box pipe connection from the sanitary system to the storm system adjacent to the proposed ultimate stormwater management facility. The location of the overflow is illustrated within Figure 2 and details of the overflow connection are illustrated within Figure 4.

6.2 Richardson Ridge

An emergency sanitary overflow will be installed to provide flooding protection for the development. The overflow will be comprised of a 450mm pipe connection from the sanitary system to the storm system adjacent to the proposed stormwater management facility. The location of the overflow is illustrated within Figure 1, and details of the overflow connection are illustrated within Figure 5.

6.3 Signature Ridge Pump Station

An emergency sanitary overflow will be installed to provide flooding protection for the Signature Ridge Pump Station. The overflow will be comprised of a 750mm pipe connection from the sanitary system to the adjacent stormwater management facility. The location of the overflow is illustrated within Figure 1, and preliminary details of the overflow connection are illustrated within the attached DWG 023.

6.4 Didsbury Road

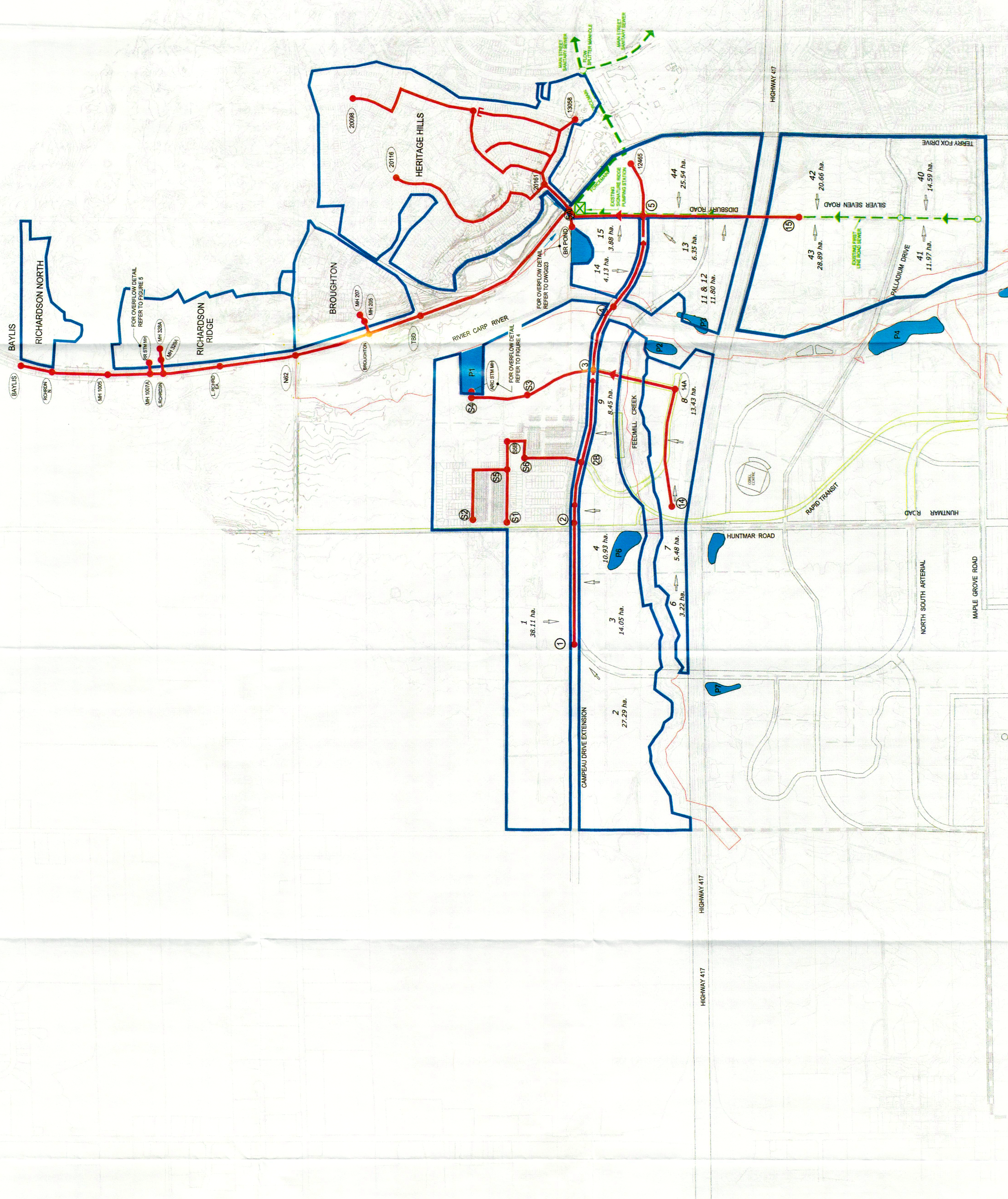
When constructed, it is recommended that the manholes along Didsbury Road, between the SRPS and Campeau Drive, be outfitted with sealed covers to prevent sanitary flooding during overflow events. Development along Didsbury Road is expected to be commercial and standard slab on grade construction. Backflow valves should be installed on the service laterals of the commercial buildings and the road profile raised to ensure 0.30m of freeboard between the underside of the slab and the sanitary HGL.

Report by:


Peter Deir, E.I.T.
Water Resources



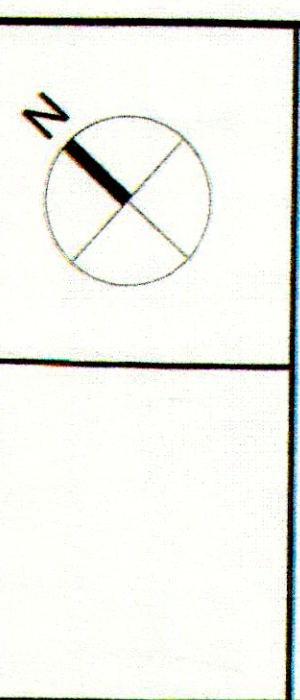

Peter Spal, P.Eng
Associate
Manager, Water Resources



No.	By	Date
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IBI GROUP
3250 Broadview Street
Toronto, Ontario M8S 4G9
Canada K1S 5N4
Tel: (416) 223-1311
Fax: (416) 223-9888

Project Title
**SIGNATURE RIDGE PUMP
STATION HYDRAULIC
GRADE LINE ANALYSIS**



Drawing Title
**ULTIMATE DEVELOPMENT
CONDITION TRIBUTARY
DRAINAGE AREA PLAN**

Scale
1:8000

Design	P.D.	Date	OCT. 2010
Drawn	M.B.	Checked	P.S.
Project No.	3775	Drawing No.	FIGURE 2

Appendix C.3

**Excerpts from *Kanata West Master Servicing Study*, Stantec Consulting Ltd and IBI. Group,
June 2006**

West. The alignment of this minor collector sewer in a north-south direct along the east limit of the development area also allows the internal sanitary sewer system to parallel the storm sewer system throughout the development area. This is a critical element in providing a coordinated and cost effective servicing scheme, which can accommodate the phased development of the KWCP over several years. The location of this sewer also facilitates the decommissioning of the Palladium Siphon and avoids placing this sewer in an area of significant fill. It also places the sewer in a designated corridor that can be graded to provide a minimum of 1.5 meters of freeboard between the 1:100 year flood level of the Carp River and the top of the covers in the access chambers. All pond outlets to the Carp River are above the 1:100 year flood plain, as well as the water elevations associated with the historical storm events analyzed in the Flood Characterization and Flood Level Analysis, prepared on CH2MHill. A designated corridor width of 20 meters is recommended for this sewer between Maple Grove Road and Palladium Drive.

Drawing S-1 also illustrates the overall major sanitary sewer system for the entire KWCP including tributary areas, pipe sizes, external pumping stations, which are to be decommissioned, and the proposed outlet location for these existing systems.

A profile of the trunk sanitary sewer system is included in the appendices of this report. These profiles illustrate the proposed elevation of the sanitary sewers relative to the storm sewers demonstrating the coordinated design of these two systems.

4.4 Design Criteria

The following City of Ottawa design guidelines were used in the preparation of the sanitary sewer analysis provided in this report:

Residential Average Daily Flow Rate	350 L/P Day
Residential Peak Factor	Modified Harmon Formula
Business Park Flow Rates	35,000 L/Ha/Day
Employment Area Flow Rate	50,000 L/Ha/Day
Business Park / Employment Peak Factor	1.5
Infiltration Rate	0.28 L/Sec/Day

4.5 Construction Phasing

It is anticipated that the build out of the KWCP will take many years. The high capital cost of the associated infrastructure dictates that a phased construction approach will be required. Servicing studies completed previously for the area have provided the framework for the phased construction of the wastewater system, which included the use of residual capacity in the existing sanitary sewer system in Stittsville. The following is a brief description of the proposed construction phasing plan, which builds on previous studies, incorporates recent construction proposals for the developers group, and incorporates additional, detailed analysis of the existing Stittsville sanitary system.

Figure 4.2-5 illustrates the area identified as Phase I in terms of wastewater servicing. Area "A" includes the area, which will be tributary to the Signature Ridge Pumping Station in the short term. This drainage area ultimately requires the expansion of the Signature Ridge Pumping Station to 400 l/sec, assumes that the lands south of Highway 417 currently draining to this pumping station will continue to do so in the short term, and that these lands will build out. This scheme also accommodates build out of the Broughton/Richardson and Interstitial Lands as part of Phase I (see **Figure 4.2-2** for details of this analysis). A regular flow monitoring program

Appendix C.4

Excerpts from *Kanata West Pumping Station Agreement*, Soloway Wright, May 18, 2007.

TABLE 1
CITY OF OTTAWA
1st LINE SEWER PUMPING STATION AND FORCEMAIN
COST SHARING AND RECOVERY
ADDITIONAL LANDS 1a,1b,9a&9b WITH 70L/s ALLOCATION
WITH AREA 9c [but not 9a&9b] CONTRIBUTION TO COSTS FOR FIRST LINE TRUNK SEWER

Area No.	Property	Land Use	Area [Ha]	Flow	Infiltr. Flow	Avg Flow	Tk. Sewer Factor	P. Station Factor	F. Main Factor	ORIGINAL DISTRIBUTION				Distrib. Of 1a/1b/9a/9b/9c Contribution	PROPOSED DISTRIBUTION WITH 1a, 1b & 9						Additional Contribution x 1.8	
										Gravity to P. Station	Pump Station	Force Main	Individual Share		T. Sewer Factor	P. Station Factor	F. Main Factor	Gravity to P. Station	Pump Station	Force Main		Revised Individual Share
1	SignatureRidge-DCR	Res.	92.70	22.21	12.98	35.19	0	0.4125	0.4125	\$1,250,000	\$2,000,000	\$270,000.00	\$3,520,000.00	\$886,836.94	0.5114	0.5757	0.5757	\$1,249,615	\$2,028,853	\$268,204.99	\$3,546,673.04	
		Ind.	2.60	1.35	0.36	1.72	0	0.0201	0.0201	\$0.00	\$824,987.25	\$111,373.28	\$936,360.53	\$235,908.84	0.0000	0.2375	0.2375	\$0.00	\$481,798.69	\$63,691.55	\$545,490.24	
		Com.	2.28	0.89	0.32	1.21	0	0.0142	0.0142	\$0.00	\$40,234.38	\$5,431.64	\$45,666.02	\$11,505.20	0.0000	0.0116	0.0116	\$0.00	\$23,497.18	\$3,106.22	\$26,603.40	
1a	Richardson	Res.		6.05	3.53	9.58	0	0.1123	0.1123	\$0.00	\$28,332.89	\$3,824.94	\$32,157.83	\$8,101.92	0.0000	0.0082	0.0082	\$0.00	\$16,546.62	\$2,187.39	\$18,734.01	
1b	Broughton	Res.		2.94	1.72	4.66	0	0.0546	0.0546						0.0000	0.0646	0.0646	\$0.00	\$131,141.15	\$17,336.25	\$148,477.40	\$267,259.32
2	Canderel	Ind.	11.35	5.90	1.59	7.49	0.0974	0.0878	0.0878	\$121,890.28	\$175,638.55	\$23,711.20	\$321,040.03	\$80,883.57	0.0000	0.0314	0.0314	\$0.00	\$63,806.30	\$8,434.90	\$72,241.20	\$130,034.15
3	Richcraft	Ind.	2.43	1.26	0.34	1.60	0.0214	0.0188	0.0188	\$26,715.28	\$37,803.67	\$5,078.50	\$69,395.45	\$17,483.65	0.0498	0.0506	0.0506	\$62,215.12	\$102,574.22	\$13,559.83	\$178,349.17	
4	City	Ind.	13.20	6.86	1.85	8.71	0.1336	0.1021	0.1021	\$167,040.28	\$204,266.85	\$27,576.03	\$398,883.16	\$100,495.55	0.0683	0.0588	0.0588	\$85,400.67	\$119,293.36	\$15,770.03	\$220,464.06	
5	764703 Ontario Ltd.	Ind.	25.04	13.02	3.51	16.53	0.4205	0.1937	0.1937	\$525,834.24	\$387,488.04	\$52,310.88	\$965,433.16	\$243,233.46	0.2151	0.1115	0.1115	\$268,734.68	\$226,295.90	\$29,915.26	\$524,945.84	
6a	790519 Ontario Ltd.	Ind.	10.58	5.50	1.48	6.98	0.1777	0.0819	0.0819	\$222,093.08	\$163,722.98	\$22,402.60	\$407,918.64	\$102,771.96	0.0909	0.0471	0.0471	\$113,546.84	\$95,615.44	\$12,639.91	\$221,802.19	
6b	790519 Ontario Ltd	Ind.	8.90	4.63	1.25	5.88	0.1495	0.0689	0.0689	\$186,826.88	\$137,725.38	\$18,592.93	\$343,145.18	\$86,452.79	0.0764	0.0396	0.0396	\$95,516.73	\$80,432.64	\$10,632.82	\$186,582.19	
7	Terrace Investments	Ind.	40.00	10.55	5.60	16.15	0.4109	0.1893	0.1893						0.2101	0.1090	0.1090	\$262,594.86	\$221,105.22	\$29,229.08	\$542,929.16	
8	Autopark	Com.	34.44	13.43	4.82	18.25	0.4643	0.2139	0.2139						0.2374	0.1231	0.1231	\$296,697.39	\$249,838.38	\$33,027.47	\$579,563.25	
9a	Taggart	Com/Res		1.30	0.76	2.06	0.0000	0.0242	0.0242						0.0000	0.0139	0.0139	\$0.00	\$29,267.38	\$3,736.62	\$32,004.20	\$57,607.55
9b	Minto	Com/Res		7.68	4.49	12.17	0.0000	0.1427	0.1427						0.0000	0.0822	0.0822	\$0.00	\$166,680.07	\$22,034.33	\$188,714.40	\$339,685.92
9c	Taggart-Cul-De-Sac	Com		0.00	0.00	0.00	0.0802	0.0000	0.0000						0.0410	0.0000	0.0000	\$51,250.00	\$0.00	\$0.00	\$51,250.00	\$92,250.00
TOTAL [flow excludes 9c]						148.19	1.96	1.7370	1.7370				\$3,520,000.01	\$886,836.94							\$3,546,673.04	\$886,836.94

NOTES:

- Area 9a&9b allocation is based on the assumption that these areas do not utilize First Line Road Sewer
- Additional Lands [1a,1b,9a & 9b] based on 70L/s peak residual capacity. Average flow calculated as follows:
Area No 1a&1b: Original Signature Residential flows factored to give peak flow for new area of 35L/s with 3.31 peak factor - Flows split 67.27% & 32.73% respectively
Area 9a & 9b: Total flow equal to total Area 1a + 1b flow with flows split 14.5% & 85.5% respectively
- Add Area 9c with negotiated contribution of \$92,500 [Trunk sewer factor adjusted to suit]

Spreadsheet
Created by:



Rev. 1 - May 24, 2005
Rev. 2 - June 1, 2005
Rev. 3 - Aug 4, 2005
Rev 4a - Jun 22, 2006
Rev 5 - Jan 8, 2007
Rev 6 - May 10, 2007

Appendix D

Geotechnical Reports

604-00402-31



JOHN D. PATERSON AND ASSOCIATES LIMITED

Ottawa

Consulting Engineers

North Bay

28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7 Tel: (613) 226-7381 Fax: (613) 226-6344

GEOTECHNICAL INVESTIGATION PROPOSED COMMERCIAL DEVELOPMENT - RETAIL A BUILDING FIRST LINE ROAD AT CAMPEAU DRIVE OTTAWA (KANATA), ONTARIO

December 13, 2002

Report No. G8689-1

Appendix p69



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**JOHN D. PATERSON AND
ASSOCIATES LIMITED**
Ottawa North Bay

Geotechnical Investigation
Proposed Commercial Development - Retail A Building
First Line Road at Campeau Drive
Ottawa (Kanata), Ontario

APPENDICES

Appendix 1 **Soil Profile & Test Data Sheets**
 Symbols and Terms
 Consolidation Testing Results
 Analytical Test Results

Appendix 2 **Figure 1 - Key Plan**
 Drawing No. G8689-1, Test Hole Location Plan



1.0 INTRODUCTION

John D. Paterson and Associates (JDPa) was commissioned by Taggart Realty Management (Taggart) to conduct a geotechnical investigation for a proposed commercial building in the south west quadrant of the intersection of Campeau Drive and First Line Road in the City of Ottawa (formerly in the City of Kanata), Ontario (see Figure 1 - Key Plan). A preliminary geotechnical investigation was carried out by JDPa for the subject site, and a summary of the findings were presented in our letter G8689-LET.01.

The objectives of the current investigation were to determine the subsoil and groundwater conditions at the location of the Retail A building by means of three (3) boreholes and, based on the results of the boreholes and previous test pits, to provide geotechnical recommendations for the design of the proposed building including construction considerations which may affect its design. The geotechnical investigation was performed in general accordance with the terms of reference in our letter of proposal dated July 23, 2002.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report. A building specific geotechnical investigation should be carried out for each building in this commercial development. This report specifically concerns the Retail A building.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation.



2.0 PROPOSED DEVELOPMENT

For ease of description, a project north, defined as being parallel to First Line Road and away from Highway 417, is being used in the following description.

The overall development site is bordered to the west by the Carp River and to the east by the First Line Road. The north boundary coincides approximately with the proposed extension of Campeau Drive while the south property coincides with a gravel roadway. At the time of this report, the north property was vacant.

It is our understanding that the proposed development includes the construction of a series of municipally-serviced commercial buildings and at grade parking areas located across the property.

The scope of this detailed geotechnical investigation is limited to the proposed "Retail A" building, which is to be located in the southeast quadrant of the site. It is anticipated that this building will consist of a municipally-serviced, single storey, slab on grade structure with associated parking areas and access lanes. A building specific investigation should be carried out for each building in the proposed commercial development once the locations of the proposed buildings are known.



3.0 **METHOD OF INVESTIGATION**

3.1 **Field Investigation**

The fieldwork for the current investigation was carried out on November 25th, 2002. At that time, 3 boreholes were advanced to a maximum depth of 30.7 m below existing grade. The borehole locations were located within the perimeter of the proposed "Retail A" building. Previously, a total of 16 test pits were completed across the subject property. The results of the previous investigation were reported in JDPa Letter No. G8689-LET.01 dated October 2, 2002. The locations of the test holes are presented on Drawing No. G8689-1, Test Hole Location Plan, in Appendix 2.

The boreholes were put down using a track-mounted auger drill rig, operated by a crew of two. The drilling procedure consisted of augering to the required depths at the selected locations and regularly sampling the overburden. All fieldwork was conducted under the full-time supervision of staff member from our geotechnical division working under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were recovered in the boreholes using a 50 mm diameter split-spoon sampler. Thin wall tubes were also used in the cohesive soil to recover soil samples to be submitted for unidimensional consolidation testing. All samples were initially classified on site. The samples were sealed in plastic bags and transported to our laboratory. The depths at which the split spoon and thin wall samples were collected are shown as SS and TW, respectively, on the Soil Profile & Test Data sheets in Appendix 1.

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.



At each borehole, the location of the inferred bedrock surface was evaluated by conducting a dynamic cone penetration test (DCPT). The DCPT consists of driving a steel rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm of penetration.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile & Test Data sheets.

Groundwater

Flexible PVC standpipes were installed in the boreholes to permit long term groundwater monitoring. The groundwater level was also observed in the open test holes at the time of the drilling and test pit operations.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations were selected in the field by JDPa personnel in a manner to provide general coverage of the property and of the proposed "Retail A" building. The locations of the test pits and ground surface elevation at each test pit location were surveyed by Annis O'Sullivan & Vollebakk Limited (AOVL). The boreholes were surveyed by JDPa personnel and referenced to existing test pits. It is understood that the ground surface elevations provided by AOVL are referenced to geodetic datum. The locations of the test holes as well as the ground surface elevation at each test hole, are presented on Drawing No. G8689-1, Test Hole Location Plan, in Appendix 2.

3.3 Laboratory Testing

A total of 87 soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.



Three samples were submitted for unidimensional consolidation testing. The test results are shown in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing for corrosion potential. The sample was analysed to determine its concentration of sulphate and chloride, its resistivity and its pH. The results are shown in Appendix 1.



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4.0 OBSERVATIONS

4.1 Surface Conditions

The subject property slopes gently from east to west, towards the Carp River, and is mostly grass covered. A maximum difference in ground surface elevation of 0.7 m was measured between borehole locations (Retail A building) while a maximum grade difference of 4.1 m was measured between the test holes across the site.

There are scattered trees along the south property line. Several barns and a two storey dwelling with a basement level exist on the east portion of the site (closer to First Line Road). Detritus, such as steel drums, hub caps, steel roof, were observed around the barns. There is a marsh area at the south west corner of the property, close to the Carp River.

4.2 Subsurface Profile

During the preliminary geotechnical investigation, the soil profile at this site was observed to vary across the site and to generally consist of topsoil overlying discontinuous layers of silty clay, clayey silt, silty sand, and/or sandy silt. Fill was observed at or near surface at TP's 14, 15 and 16. The findings at the three boreholes conducted as part of the current investigation generally concur with the previous test pit findings. At the Retail A building location, the soil profile consists of clayey silt and silty sand overlying silty clay. The upper layers of clayey silt and silty sand extend to depths varying between 6.4 m at BH 3 and 8.7 at BH 1. Inferred glacial till was observed at BH 1 at a depth of 29.9 m. Refusal to DCPT on inferred bedrock surface was encountered at depths varying between 20 and 30.7 m.

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

Topsoil

A topsoil layer measuring 80 mm in thickness was observed at the three borehole locations. This layer was observed to range in thickness from 25 to 380 mm at the test holes across the site.



Fill

Brown sand to silty sand, with trace of gravel and brick pieces, was encountered below the topsoil layer at TP's 13, 14, 15, and 16. The fill extends to depths varying between 0.5 and 1.8 m. The fill layer was not fully penetrated at TP's 15 and 16.

Silty Sand, Sand

A brown silty sand layer was encountered below the topsoil at TP's 1, 3, 6, 7, 9, 10, 11, and 12 and at BH 3. This layer extends to depths ranging from 0.4 to 1.7 m below existing grade. Silty sand and/or sand layers interbedded with sandy silt, silty clay and/or clayey silt are also encountered further below grade. At the location of Retail A building, these interbedded layers extend to depths varying between 6.4 and 8.7 m.

Sandy Silt

Sandy silt, extending to depths of 0.9 and 0.6 m, was encountered below the topsoil at TP's 4 and 5, respectively. Sandy silt was also encountered at greater depths at TP's 1, 3, 6, 9, 10, and 11.

Based on the SPT results and on the behaviour of the test pit walls, the state of compactness of these layers ranges from very loose to compact.

Clayey Silt, Silty Clay

Clayey silt/silty clay was encountered below the topsoil layer at TP's 2 and 8 and at BH's 1 and 2. Clayey silt/silty clay was also encountered below the interbedded layers of sand, silty sand and sandy silt at all test hole locations except TP's 13 to 16.

Generally, the upper portion of this layer has been weathered to a stiff brown crust. The thickness of this weathered crust ranges from 0.2 m to 1.9 m. No brown crust was encountered at TP's 4 and 5 and at BH's 1 to 3. In situ shear vane field testing carried out in the weathered crust yielded results ranging from about 85 to 110 kPa. These values are indicative of a stiff to very stiff consistency.



A grey clayey silt to silty clay layer was encountered at TP's 1, 2, 3, 4, 5, 8, 9 and 10 at depths varying between 2.2 and 3.5 m below grade, and at depths ranging from 6.4 to 8.7 m in BH's 1 to 3. Imbedded white sea shells and trace of organic matter were noticed in this layer at all test pit locations except for TP's 1, 9 and 10. In situ shear vane field testing carried out in this portion of the clayey silt to silty clay yielded results ranging from about 15 to 50 kPa. These values are indicative of a soft to firm consistency.

Three silty clay samples were each submitted for a unidimensional consolidation test. The results are summarized in Table 1. The results are also shown in Appendix 1. The consolidation tests were conducted in our geotechnical laboratory to determine the consolidation characteristics of representative fine-grained samples from BH's 2 and 3.

Value p'_c is the preconsolidation pressure of the sample and p'_o is the effective overburden pressure. The difference between these values is the available overconsolidation (O.C. column). The increase in stress on the soil due to the cumulative effects of fill surcharge, footing pressures, slab loadings and groundwater lowering cannot exceed the available overconsolidation, or settlements will be unacceptable.

The values C_{cr} and C_c are the recompression and compression indices, respectively, and are a measure of the compressibility of the soil due to stress increases below and above the preconsolidation pressure, respectively. The higher values for the C_c , as compared to the C_{cr} , illustrate the increased compression potential above, as compared to below, the preconsolidation pressure.

Table 1 - SUMMARY OF SITE SPECIFIC CONSOLIDATION TEST RESULTS							
Sample	Depth (m)	Elevation (m)	p'_o (kPa)	p'_c (kPa)	O.C. (kPa)	C_{cr}	C_c
BH2-TW1	8.08	89.98	132	100	32	0.019	0.653
BH2-TW3	14.15	82.91	175	148	27	0.020	0.781
BH3-TW4	9.60	88.06	177	111	66	0.015	0.650



It should be noted that the preconsolidation pressure obtained from BH 2-TW3 is inconsistent with the undrained shear strength obtained on the sample and in the borehole at that location. As well, the result is not consistent with the usual gradual increase with depth of the preconsolidation pressure and of the in-situ effective pressure. For these reasons, this result is disregarded in the following discussion.

Inferred Glacial Till

Based on the results of the DCPTs, a layer of glacial till was inferred below the silty clay layer at a depth of 29.9 m at BH 1. These layers were not inferred in BH's 2 and 3. Glacial till generally consists of fine soils mixed with gravel, cobbles and boulders.

Based on the results of the SPTs, the state of compactness of this layer is compact.

Practical Refusal to DCPTs

Practical refusal to DCPTs was encountered at BH's 1 to 3 at depths varying between 20 and 30.7 m. Due to the presence of the glacial till in this area, it is possible that the refusal was encountered on boulders rather than bedrock.

Based on available geological mapping, the subject site is located in a geological fault area. The bedrock consists of undifferentiated metamorphic and igneous bedrock, or of fine to coarse grained quartz sandstone of the Nepean formation.

4.3 Groundwater

Groundwater levels were measured in the open test pits on September 4th and 5th and in the standpipes in BH's 1 to 3 on December 4th, 2002. The measured groundwater levels are presented in Table 2.

The groundwater levels vary between 1.2 and 3.6 m below existing grade. Test Pits 13, 14, 15 and 16 were dry upon completion. The groundwater levels in the test pits are not considered to be representative of stabilized groundwater levels as they were taken in the test pits left open for only a short period of time.

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



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Appendix 3 - Summary of Geotechnical Test Results			
Test Pit	Measured Soil Depth (m)	Soil Elevation (m)	Recording
TP 1	2.5	93.34	Sept 4, 2002
TP 2	2.7	92.38	Sept 4, 2002
TP 3	3.4	90.78	Sept 4, 2002
TP 4	3.1	91.49	Sept 4, 2002
TP 5	3.3	91.65	Sept 4, 2002
TP 6	2.8	94.26	Sept 4, 2002
TP 7	1.2	93.68	Sept 4, 2002
TP 8	2.2	91.92	Sept 4, 2002
TP 9	2.4	91.35	Sept 5, 2002
TP 10	3.6	91.99	Sept 5, 2002
TP 11	3.1	93.81	Sept 5, 2002
TP 12	2.2	93.48	Sept 5, 2002
TP 13	(dry)	-	Sept 5, 2002
TP 14	(dry)	-	Sept 5, 2002
TP 15	(dry)	-	Sept 5, 2002
TP 16	(dry)	-	Sept 5, 2002
BH 1	3.7	94.08	Dec 4, 2002
BH 2	4.3	92.73	Dec 4, 2002
BH 3	3.3	94.41	Dec 4, 2002

Note: The ground surface elevation at the test pit locations were provided by Annis O'Sullivan & Vollebakk.



5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Geotechnical Assessment

It is understood that a one storey, slab on grade commercial structure is proposed at this site. It is anticipated that the building will be founded on footings placed at shallow depth within the native soils below any fill or topsoil. Alternatively, the building could be founded on piles driven to bedrock at depths of about 20 to 31 m below existing grade.

Access roadways and parking areas are also considered.

The subject site is considered to be acceptable for the proposed commercial development. However due to the presence of soft soils conditions, only small grade raises will be permissible at this site.

It should be noted that high groundwater conditions were encountered on this site. This will make the excavations and preparation for the footing and service pipe beds more difficult at the time of construction.

5.2 Site Grading and Preparation

Stripping Depth

All topsoil and fill should be removed from within the footprint of the building. Topsoil and deleterious fill, such as those containing organic material, should be stripped from under any paved areas and other settlement sensitive structures. Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.



Fill Placement

Fill used for grading beneath the building, between footings and foundation walls, and paved areas should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or II unless light weight fill is specified. The OPSS materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no thicker than 300 mm and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 95% of its standard Proctor maximum dry density. Light weight fill, if present, should consist of expanded polystyrene (EPS) blocks and should extend at least 3 m outside the perimeter of the building.

The EPS blocks should be placed as low as possible in the grade raise profile to use the permissible soil fill over the EPS blocks to resist potential uplift pressure due to hydrostatic pressure, which could be exerted on the blocks during high groundwater level conditions. The dead load of the materials above the EPS fill must be sufficient to resist the potential uplift pressure. Otherwise, another means such as an anchoring system must be provided to increase the uplift resistance. A minimum factor of safety of 1.1 is recommended when calculating the uplift resistance based on dead load only. Otherwise, a minimum factor of safety of 1.5 is recommended. The effective unit weight of the fill material should be used below the groundwater table when calculating the uplift resistance. Alternatively, means to maintain the groundwater level below the bottom of the EPS block layer should be provided. It is recommended that the top surface of the EPS block layer be covered with a polystyrene sheet to minimize water infiltration on the blocks. The EPS block layer should be backfilled with free draining materials and placed on a minimum 200 mm thick layer of 19 mm clear crushed stone material. A perimeter drainage system should be provided around the EPS block layer to minimize the risks of buildup of hydrostatic pressures.

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to a minimum of 95% of its standard Proctor maximum dry density. Site-excavated silty clay, clayey silt, sandy silt and silty sand are not suitable for use as backfill against foundation walls.



5.3 Foundation Design

General

Shallow footing foundation will be permissible at this site only if the grade raise is kept to a minimum or if the required grade raise is achieved using light weight fill such as expanded polystyrene (EPS) blocks. Otherwise deep foundation alternative such as a pile foundation would be required.

Based on the subsoil conditions and the results of the consolidation testing, the anticipated settlements for different grade raises are shown in Table 3.

Table 3 - Settlements for Grade Raise	
Grade Raise m	Anticipated Settlements mm
0.5	5
0.75	20
1	40
1.25	70
1.5	100

Shallow Foundation - Allowable Bearing Pressures

Based on the soil profile encountered at the proposed building locations, the building could be founded on shallow strip footings within the silty fine sand or clayey silt crust below any topsoil.

The allowable bearing pressures recommended for the design of the footings depend on several factors including the shear strength of the soil below the footings, the elevation of the footings and the amount of grade raise at the building.



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The bearing pressures must be checked against shear failure and against unacceptable settlements. For the shear failure case, the allowable bearing pressures should be checked using 100% of the dead and live loads. For the settlement case, the allowable bearing pressures should be checked against 100% of the dead load loads and 50% of the live loads.

Based on shear strength failure, the allowable bearing pressures for footing placed on clean, undisturbed stiff clayey silt, silty sand or engineered fill could be designed using an allowable bearing pressure of 70 kPa.

For the settlement case, the allowable pressures are presented in Tables 4 and 5 assuming that the final grade raise at the building is 0.75 m or less. Any additional grade raise will require the use of light weight fill if a shallow footing foundation is to be used. The light weight fill should extend at least 3 m outside the perimeter of the building and below the floor slabs.

The proposed grade raise (standard fill, not light weight fill) is acceptable only if the floor slabs have a standard floor loading conditions (i.e. about 5 kPa). Loading conditions such as those associated with high rack stacks would not be permissible without reducing the allowable bearing pressures, increasing the ratio of light weight fill versus standard fill, or providing structural slabs.

Allowable Bearing Pressure (kPa)			
Dead Load Plus 50% Live Load			
Strip Footings			
Bearing Width			
96.95 or above	40	25	20
Maximum grade raise of 0.75 m.			



Table 1 Allowable Bearing Pressure Based on the 1994 Square Footings					
Foundation	1.0m x 1.0m	1.5m x 1.5m	2.0m x 2.0m	3.0m x 3.0m	4.0m x 4.0m
1.0m x 1.0m	96.95	70	70	50	30
1.5m x 1.5m					20
Maximum grade raise of 0.75 m.					

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings, or mud slabs.

The footing beds will be susceptible to disturbance under workmen or construction traffic. Consideration should be given to placing a granular pad or lean concrete over the exposed and approved undisturbed subgrade to minimize the risks of disturbance.

Lateral Support

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

Grade Raise / Settlement

The grading plan for this site is not known at this time. Based on the subsoil conditions at this site, a grade raise is a concern from a settlement view point.

Footings designed using the allowable bearing pressure provided herein will be subjected to potential total and differential settlements of 25 and 15 mm, respectively, provided that the grade raise is a maximum of 0.75 m.



Pile Foundations

If the loading conditions and the amount of grade raise are beyond those possible to permit shallow footing foundations, it will be necessary to transfer the foundation loads through the sensitive silty clay to clayey silt to more competent bearing at depth. This can be achieved with piles driven to end bearing on or near the bedrock. A suitable pile type would be concrete filled steel pipe piles driven closed ended and bearing on bedrock.

It should be noted that a pile foundation could be used in conjunction with structural slabs or with floating slabs if the anticipated settlements are acceptable. Where unacceptable settlements are anticipated the ratio of light weight fill versus earth fill should be increased or a structural slab should be used.

As one possible pile design example, the allowable load on a 245 mm diameter steel pipe pile with a wall thickness of 9 mm may be taken as 940 kN if it is driven to a final set of 10 blows for the last 12 mm of penetration using a hammer developing about 28 kJ of transferred energy per blow. A second possible design example would be a 245 mm diameter steel pipe pile with a wall thickness of 13.8 mm. The allowable load on this pile may be taken as 1245 kN if it is driven to a final set of 10 blows for the last 12 mm of penetration using a hammer developing about 42 kJ of transferred energy per blow (these values assume that grade 350 steel and 30 MPa concrete are used).

Pipe piles should be equipped with a base plate having a thickness of at least 20 mm to minimize damage to the pile tip during driving.

Provision should be made for restriking all of the piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the restrike should receive additional restriking until the design set is met. All restriking should be performed after 48 hours of the previous set.

It is recommended that a pile load test or dynamic monitoring and capacity testing be carried out at an early stage in the piling operation to verify the transferred energy from the pile driving equipment and determine the load carrying capacity of the piles. The recommended number of tests is dependent on the number of piles and pile sizes; as a guidelines a minimum of 2 tests per pile size should be carried out. It is also recommended that the tested pile locations be spread out across the building footprint.



Downdrag load should be considered in the design of the piles. The downdrag load acting on a pile is proportional to the pile shaft diameter, the length of the pile portion embedded in silty clay, the pile shaft area and the undrained shear strength of the silty clay.

At this site, the downdrag force is estimated to be 270 kN for the 245 mm outside diameter concrete filled steel pipe pile.

The downdrag force should be considered only with the dead load. The downdrag load is not present when the live load is applied. Therefore, the design load should be the greater of the dead load plus the downdrag load or the dead load plus the live load. Those values should be compared to the structural load capacity of the pile.

If piles are to be installed at this time and left exposed during winter months, some form of frost protection will be required to prevent frost adhesion and jacking of the piles. This protection could consist of insulation or earth cover. Alternatively, a bond break could be installed between the piles and surrounding soil by wrapping the upper 2 m of the piles with three layers of 10 mil polyethylene sheeting.

The potential post construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

5.4 Design for Earthquakes

The foundation factor, F , for the design of structures against earthquake forces, can be taken to be 1.3 for the site considered in this investigation. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.



5.5 Concrete Floor Slabs

With the removal of all topsoil and fill within the footprint of the proposed building, the native soil surface will be considered an acceptable subgrade surface on which to commence backfilling for floor slab construction. Provision should be made for proof-rolling the soil subgrade using heavy vibratory compaction equipment prior to placing any fill. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type I or II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 150 to 200 mm of sub-slab fill consists of an OPSS Granular A crushed stone material. All backfill material within the zone of influence of a footing or within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its standard Proctor maximum dry density.

Slabs on grade will be permissible for this building only if the anticipated settlements induced by the earth fill, light weight fill, if present, and the loading on the slab are acceptable. Otherwise, a structural slab is recommended. The anticipated settlements for the slab are those shown for the grade raise only (see Table 3). It should be noted that the loads acting on the slab should also be accounted for when considering the anticipated settlements. It is suggested that at least 80% of the live loads be considered. The loads can be converted in equivalent grade raise using a unit weight of 21 kN/m^3 .

5.6 Loading Dock and Basement Wall

A short retaining wall structure is often present in the design of proposed retail store. The footing of the retaining wall should be provided with a minimum 1.8 m thick soil cover. The footing of the retaining wall should be designed using a allowable bearing pressures presented earlier.

The retaining wall should be backfilled with free draining material such as OPSS Granular B Type I. Drainage of the fill material behind the wall should be provided by means of sleeves through the base of the wall. The sleeve openings should be provided with a geotextile to prevent clogging and loss of materials. The wall should be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to $K_a \gamma H$ where:



$$\begin{aligned}K_a &= 0.33 - \text{active earth pressure coefficient} \\ \gamma &= 22 \text{ kN/m}^3, \text{ unit weight of the fill} \\ H &= \text{height of the retaining wall, m}\end{aligned}$$

The above formula should be applied only if the wall is free standing and independent of the foundation wall of the building.

The fill material behind the wall should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its standard Proctor maximum dry density. Only small vibratory compaction equipment should be used within 0.5 m of the wall to minimize compaction induced stresses.

The side wall of the footing excavation should be tapered at 3H:1V or flatter on the pavement side of the loading dock and backfilled with OPSS Granular B Type I or II to provide a taper to minimize potential frost heaving. The fill material should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's standard Proctor dry density.

The foundation wall at a depressed loading dock, if present, will act as a retaining wall. Therefore, the wall should be designed to resist the lateral earth pressure of the fill material on the inside of the foundation wall. The formula provided above should be used for the design of the foundation wall using an earth pressure coefficient of 0.35, if some movements can be tolerated, and 0.5 if no movement can be tolerated (usual foundation wall). It should be noted that the fill on the inside of the wall should consist of free draining material such as OPSS Granular Type I or II.

5.7 Pavement

Car only parking areas and heavy traffic areas are proposed at this site. The subgrade material will consist of silty clay, clayey silt, silty sand, sand and/or sandy silt. The proposed pavement structures are shown in Tables 6 and 7.



Thickness mm	Description
50	HL-3 - Wear Course Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or inorganic fill

Thickness mm	Description
40	HL-3 - Wear Course Asphaltic Concrete
50	HL-8 - Binder Course Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or inorganic fill

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's standard Proctor maximum dry density using suitable vibratory equipment.



If onerous soil conditions are encountered at the time of construction, the thickness of the subbase material could be increased and/or a woven geotextile could be used to strengthen the pavement granulars. This could be assessed at the time of construction.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Consideration should be given to installing subdrains at each catch basin during the pavement construction. These drains should be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.



6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 100 mm to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the building. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill. Imported granular materials, such as clean sand or Ontario Provincial Standard Specifications (OPSS) Granular B Type I granular material, should be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated slab-on-grade structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.



6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Above the groundwater level, the excavation side slopes extending to a maximum depth of 3.5 m should be cut back at 1H:1V. Flatter slopes will be required below the groundwater level.

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

Slopes in excess of 3.0 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the material's standard Proctor maximum dry density.

It is expected that the moist (not wet) sand/silty sand, brown silty clay and clayey silt could be used above the cover material if the excavation operations are carried out in dry weather conditions. Due to their high water content, the grey silty clay and sandy silt may be impractical to compact without a long drying period.



Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's standard Proctor maximum dry density. However, due to the variable nature of the subgrades it may be practically impossible to match the native soil strata. In this case, the existing material should still be used but a minimum frost taper of 1.5H:1V should be used.

The subgrade material is susceptible to the presence of water. It is recommended that a granular pad be placed for piling operations and, upon exposure, at the bottom of the excavations for the slabs on grade and the footings to reduce the risks of disturbance due to the construction traffic.

To avoid long term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The clay seals should be as per Standard Drawing No. S8 of the Department of Transportation, Utilities & Public Works Infrastructure Services Branch (TUPWISB) of the City of Ottawa. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers compacted to a minimum of 95% of the material's standard Proctor maximum dry density. The clay seals should be placed at the site boundaries and at a spacing no greater than 100 m along the trenches.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium and subgrades.

The rate of groundwater flow into the excavation through the clayey silt and silty clay should be relatively low due to the impervious nature of these materials. Higher inflows are expected through silty sand, sand, and sandy silt. It is expected that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the shallow excavations anticipated at this site.

↳ Permit to take water



It should be noted that most of these soils are susceptible to disturbance from workmen traffic in the presence of water. Therefore, it is recommended that the excavations be backfilled as soon as possible. Consideration should be given to using a mud slab (lean concrete) or granular pad over the approved undisturbed surfaces to protect the bottom of the excavation and provide a stable work platform.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist mostly of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the buildings and the footings are protected with sufficient soil cover to prevent freezing at the founding level.

The excavation and backfilling operations as well as pavement construction should be carried out in such a manner to avoid the introduction of frozen material in these areas. Additional information can be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing shows that the sulphate content is 55 µg/g. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH and resistivity indicate the presence of a non-aggressive to slightly aggressive environment for exposed metals at this site.



7.0 INSPECTION SERVICES

It is a requirement for the design data provided herein to be applicable that the following inspection program be carried out by the geotechnical consultant.

- ☐ Review of the proposed grading plan.
- ☐ Inspection of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic inspection of the condition of unsupported excavation side slopes in excess of 3.0 m in height, if applicable.
- ☐ Inspection of all subgrades prior to backfilling and follow-up field density tests to ensure that the specified level of compaction has been achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

The completion of an inspection program of this magnitude will result in the issuance of a report confirming that these works have been conducted in general accordance with our recommendations.



8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the grading plan, drawings and specifications are complete. A building specific geotechnical investigation should be carried out for each proposed building in this proposed commercial development.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those described by the test holes themselves. A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than the current owner(s) or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.



**JOHN D. PATERSON AND
ASSOCIATES LIMITED**
Ottawa North Bay

Geotechnical Investigation
Proposed Commercial Development - Retail A Building
First Line Road at Campeau Drive
Ottawa (Kanata), Ontario

We trust that this submission will fulfill your present requirements. Should you require additional information, please do not hesitate to contact this office.

Sincerely,

JOHN D. PATERSON AND ASSOCIATES LIMITED

Glenn Collins, P. Eng.

Andrew J. Tovell, P. Eng.

Report Distribution:

- ☐ Taggart Realty Management (3 copies)
- ☐ John D. Paterson and Associates (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TESTING RESULTS

ANALYTICAL TEST RESULTS



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28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

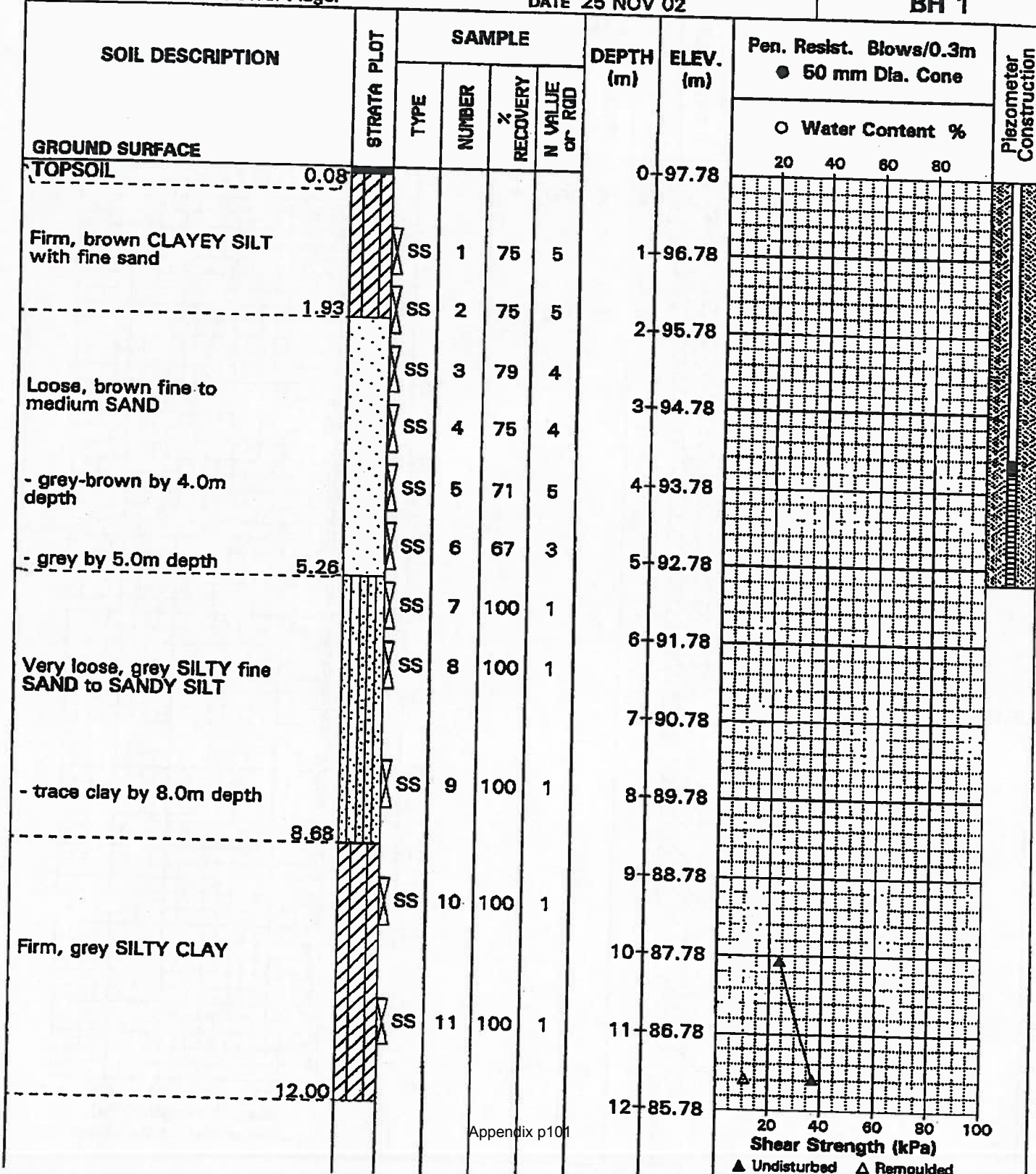
DATE 25 NOV 02

FILE NO.

G8689

HOLE NO.

BH 1





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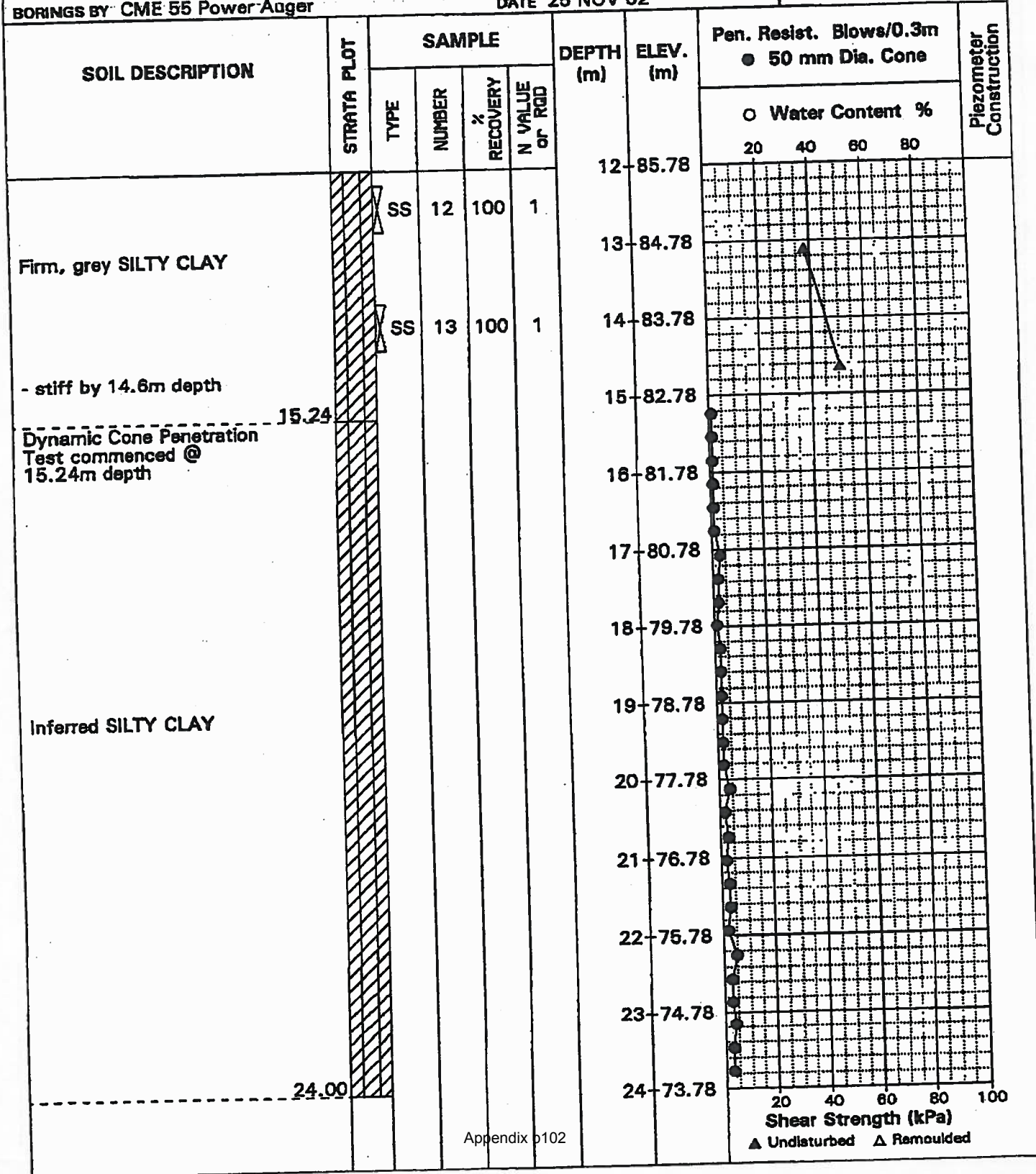
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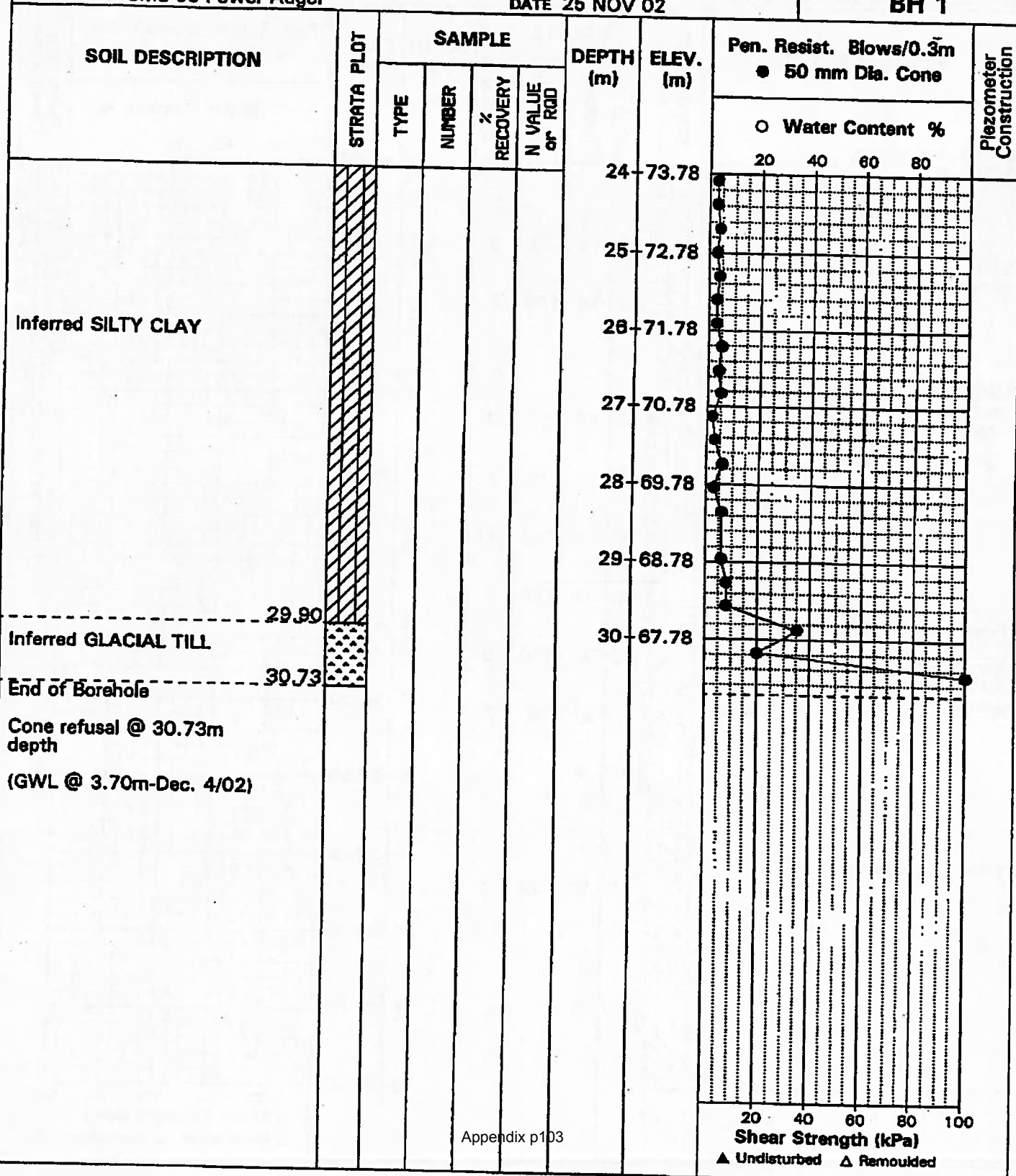
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BH 1





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Ottawa (Kanata), Ontario

DATUM: Approximate geodetic

REMARKS

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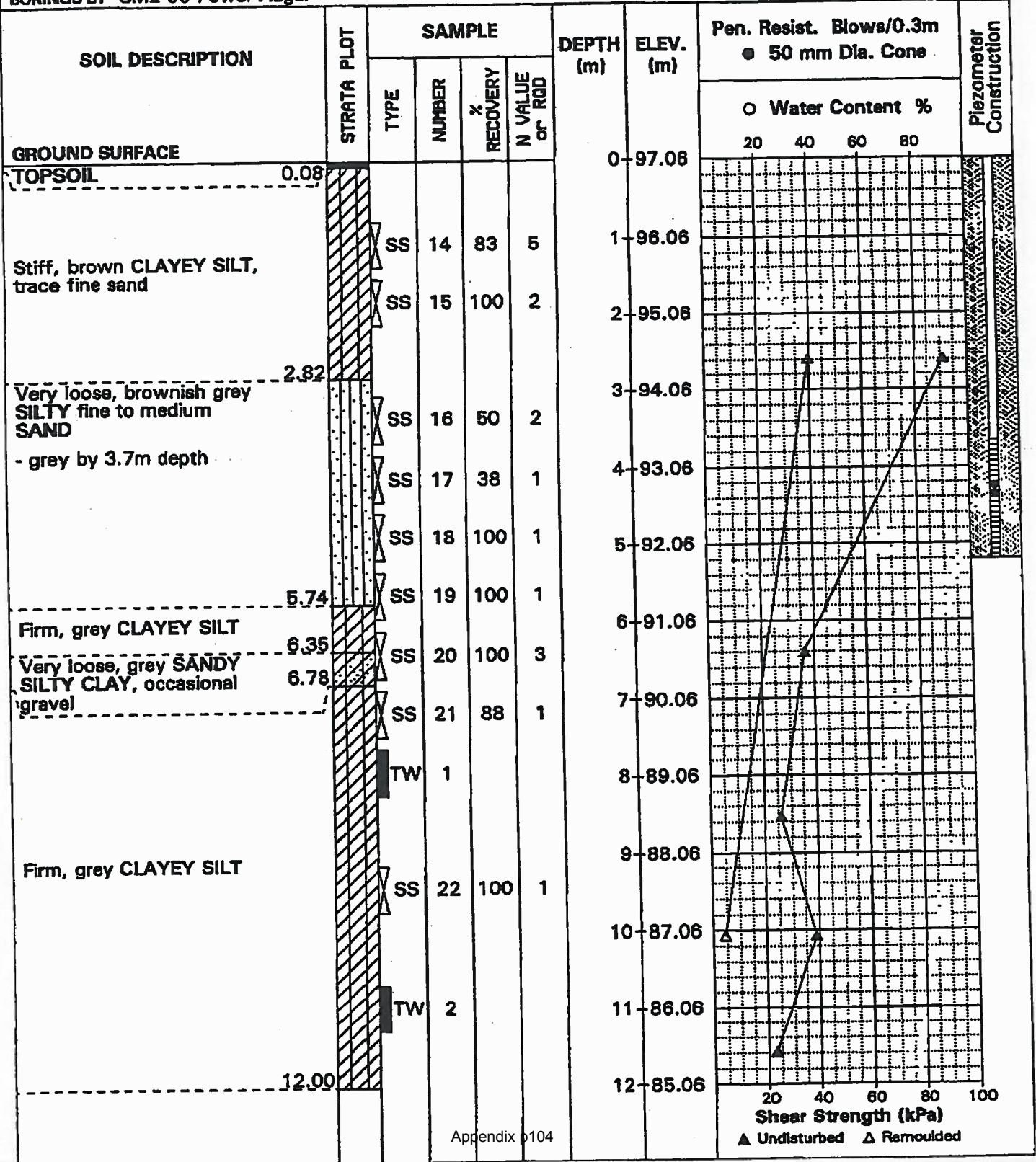
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HOLE NO.

BH 2





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SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM - Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

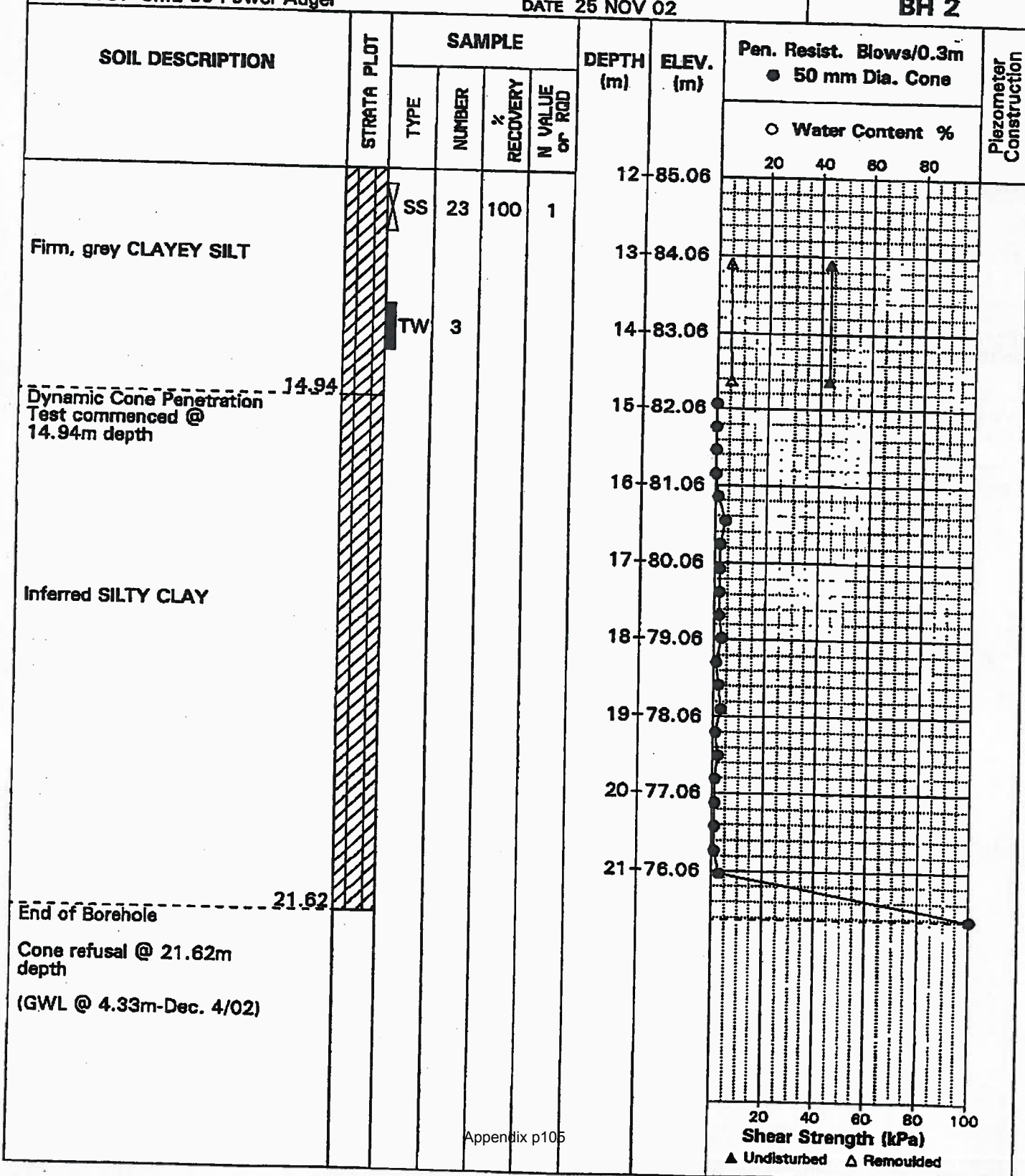
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FILE NO.

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HOLE NO.

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Ottawa (Kanata), Ontario

DATUM Approximate geodetic

FILE NO.

G8689

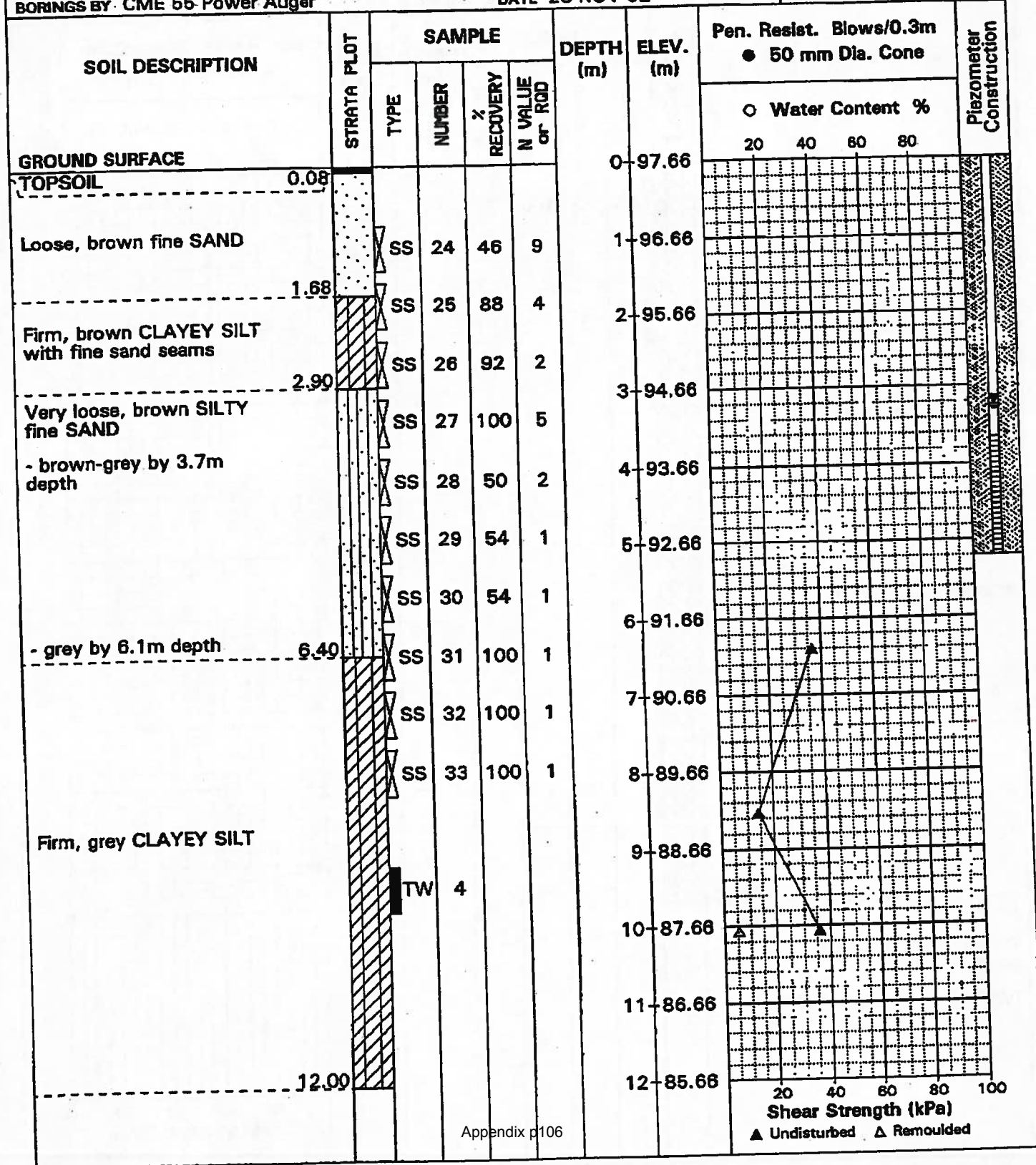
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SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Approximate geodetic

REMARKS

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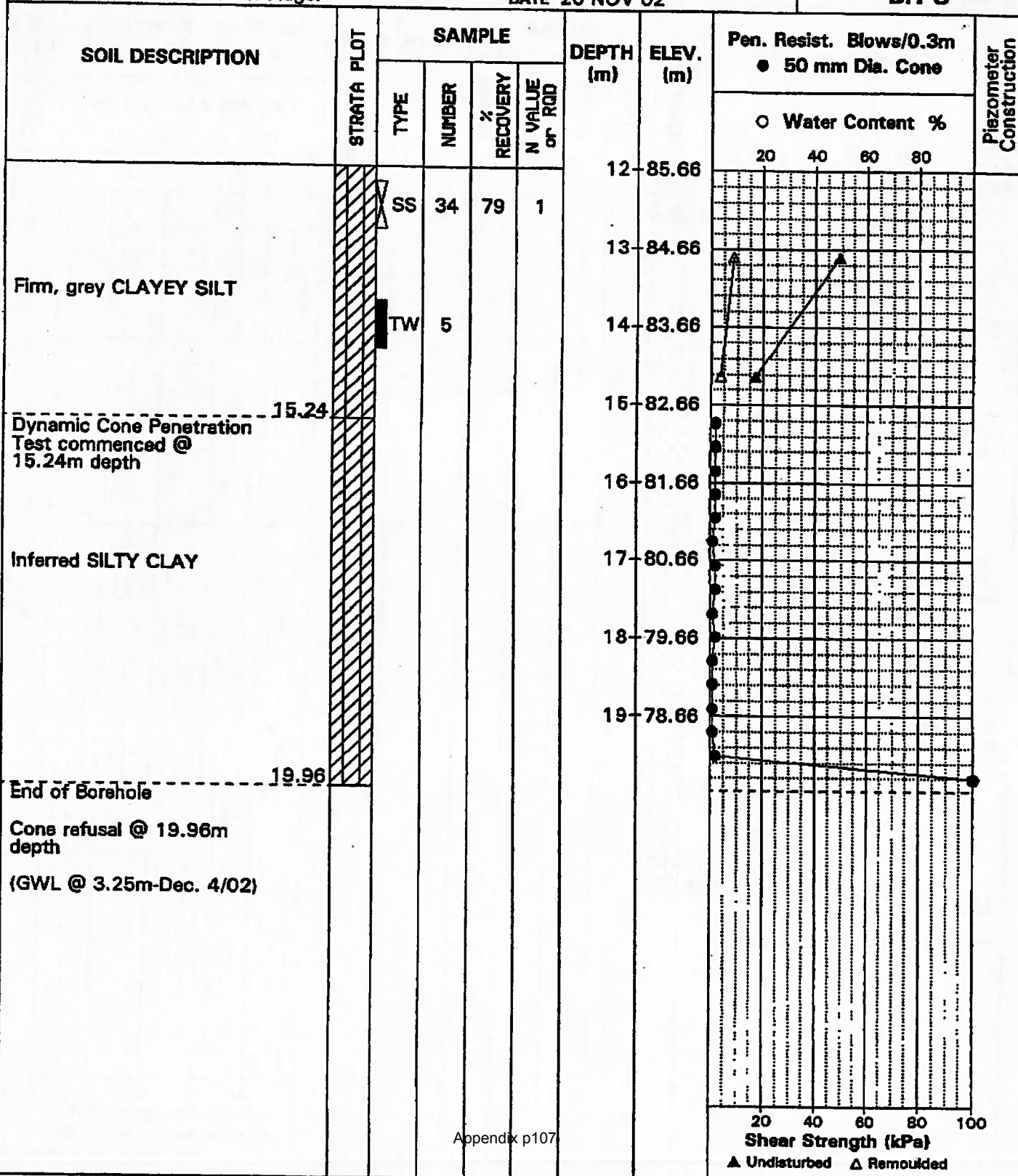
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HOLE NO.

BH 3





**Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**

TP 1

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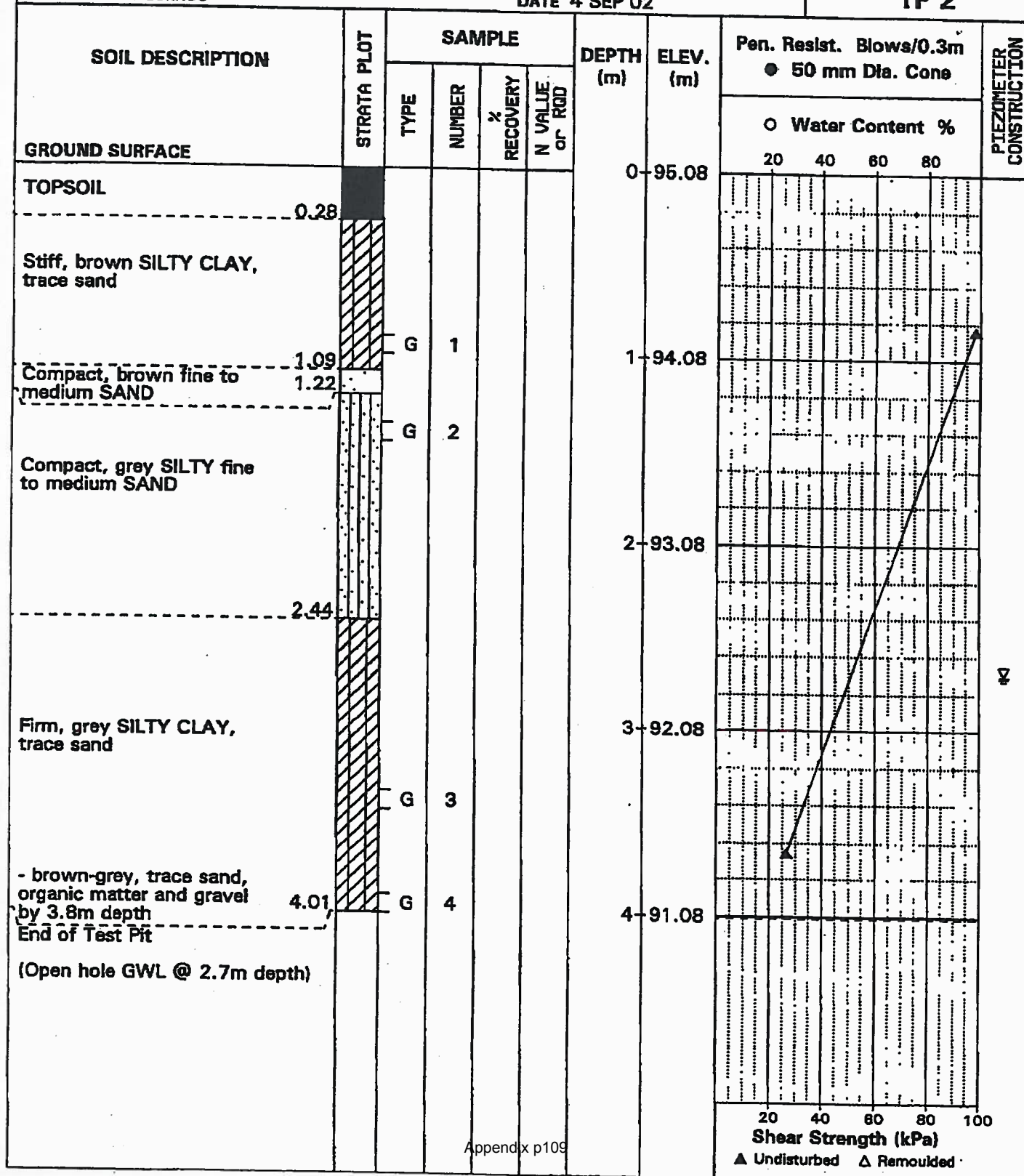
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SOIL PROFILE & TEST DATA

Geotechnical Investigation

First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**DATUM** Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.**REMARKS****FILE NO.****G8689****BORINGS BY** Backhoe**DATE** 4 SEP 02**HOLE NO.****TP 2**

SOIL PROFILE & TEST DATA

**Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**

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G8689

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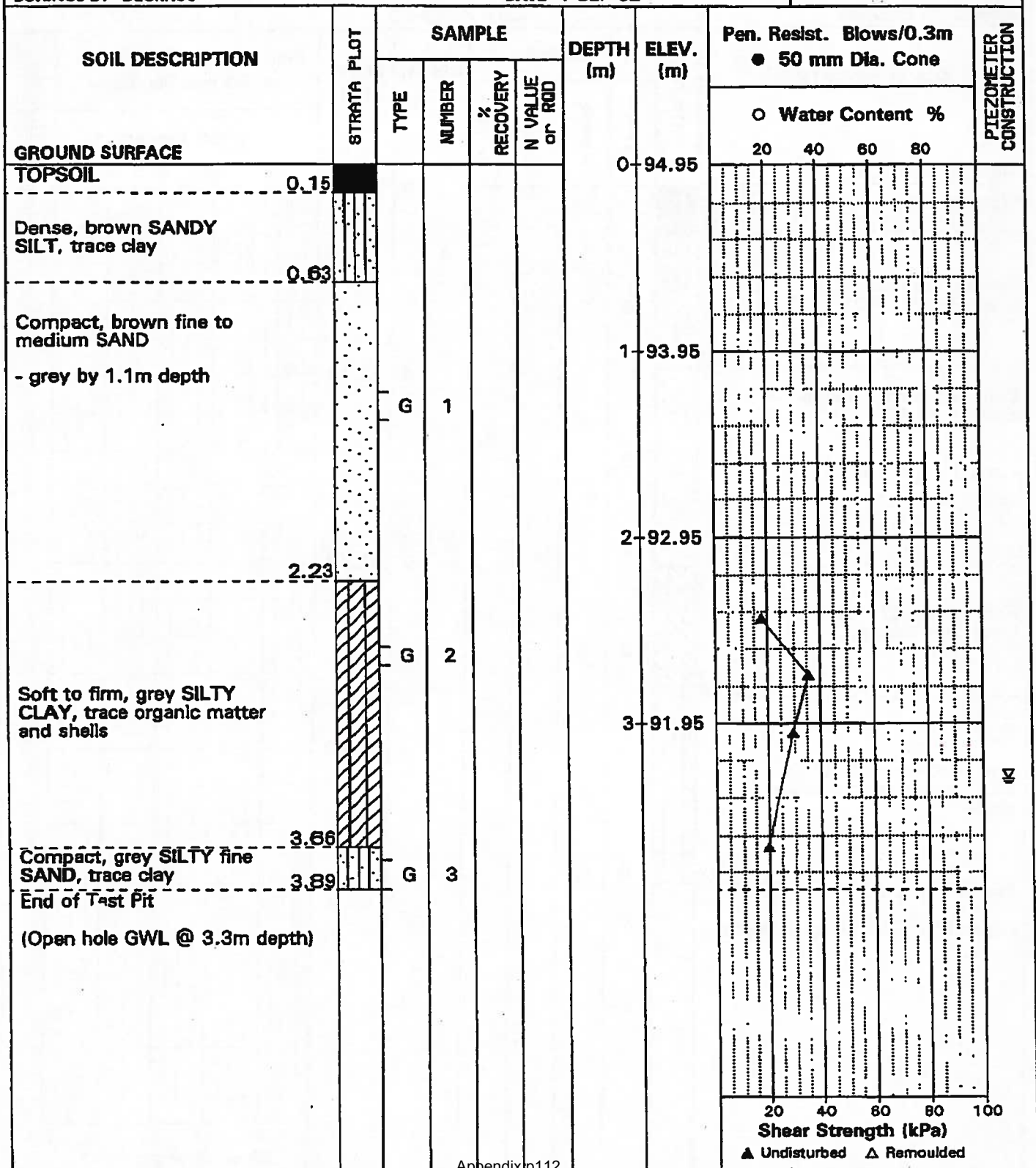
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

First Line Road @ Campeau Drive

Ottawa (Kanata), Ontario

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Ottawa (Kanata), Ontario**

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FILE NO. **G8689**

REMARKS

HOLE NO. **TP 6**

BORINGS BY Backhoe

DATE 4 SEP 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or ROD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	97.06					
TOPSOIL												
Compact to dense, brown SILTY fine SAND												
Compact, grey SANDY SILT												
Firm, brown SILTY CLAY		G	1			1	96.06					
		G	2			2	95.06					
Compact, brown fine to medium SAND						3	94.06					
End of Test Pit		G	3									
(Open hole GWL @ 2.8m depth)												

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20
40
60
80
100

Shear Strength (kPa)

▲ Undisturbed
△ Remoulded

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SOIL PROFILE & TEST DATA

**Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**

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FILE NO.

G8689

REMARKS

HOLE NO.

TP 7

BORINGS BY Backhoe

DATE 4 SEP 02

SOIL DESCRIPTION		STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone					PIEZOMETER CONSTRUCTION
			TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %					
									20	40	60	80		
GROUND SURFACE							0	94.88						
TOPSOIL														
0.25														
Compact, brown SILTY fine SAND														
0.56														
Stiff, brown SILTY CLAY, trace sand			G	1										
1.00							1	93.88						
			G	2										
Compact, brown SILTY fine to medium SAND														
2.44							2	92.88						
			G	3										
Compact, brown SAND, some shells														
3.25							3	91.88						
End of Test Pit (Open hole GWL @ 1.2m depth)														



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SOIL PROFILE & TEST DATA

Geotechnical Investigation

First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

REMARKS

FILE NO.

G8689

BORINGS BY Backhoe

DATE 4 SEP 02

HOLE NO.

TP 8

SOIL DESCRIPTION		STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
			TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
									20	40	60	80	
GROUND SURFACE													
TOPSOIL							0	94.12					
0.23													
Stiff, brown CLAYEY SILT, trace sand and organic matter		G	1										
- grey by 1.2m depth		G	2				1	93.12					
2.03													
Compact, brown fine to medium SAND, trace shells		G	3				2	92.12					
- grey by 2.3m depth		G	4										
2.74													
Firm, grey SILTY CLAY, trace shells and organic matter		G					3	91.12					
3.35													
End of Test Pit													
(Open hole GWL @ 2.2m depth)													



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SOIL PROFILE & TEST DATA

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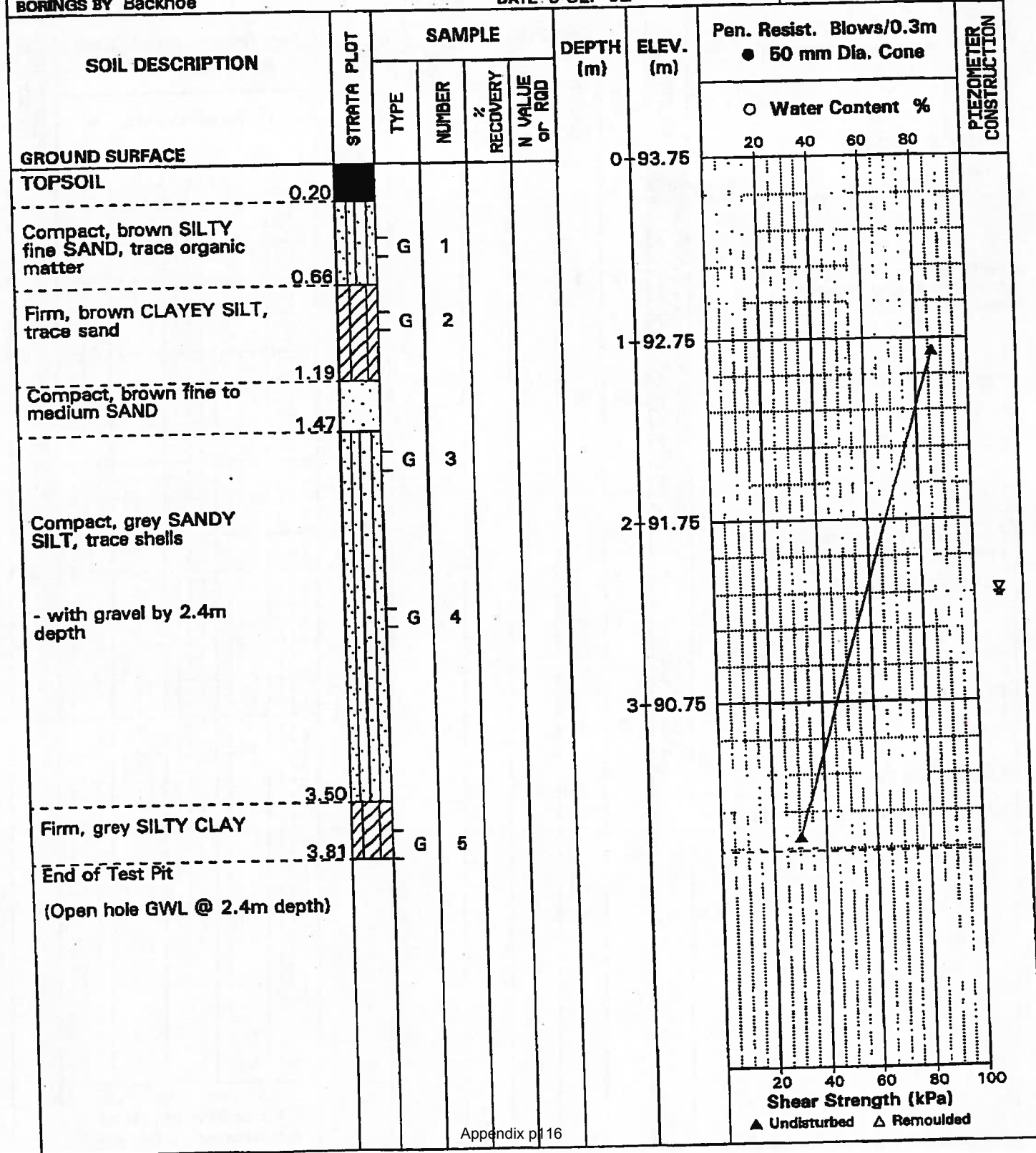
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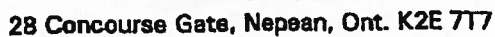
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**Geotechnical Investigation
First Line Road @ Campeau Drive
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SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

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REMARKS

BORINGS BY Backhoe

DATE 5 SEP 02

FILE NO.

G8689

HOLE NO.

TP14

SOIL DESCRIPTION

STRATA PLOT

SAMPLE

DEPTH (m)

ELEV. (m)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

PIEZOMETER
CONSTRUCTION

GROUND SURFACE

TOPSOIL

0.18

FILL: Brown silty fine sand
with interbeds of hay/grass

0.76

G

1

G

2

Compact, brown SILTY
fine SAND

2-96.75

3-95.75

End of Test Pit

3.60

(TP dry upon completion)

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed Δ Remoulded

JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

FILE NO. **G8689**

HOLE NO. **TP15**

DATUM

Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

REMARKS

DATE 5 SEP 02

BORINGS BY Backhoe

[illegible]



JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

REMARKS

BORINGS BY Backhoe

DATE 5 SEP 02

FILE NO.

G8689

HOLE NO.

TP14

SOIL DESCRIPTION

STRATA PLOT

SAMPLE

TYPE

NUMBER

%
RECOVERY

N VALUE
or R_{qd}

DEPTH
(m)

ELEV.
(m)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

PIEZOMETER
CONSTRUCTION

GROUND SURFACE

TOPSOIL

0.18

FILL: Brown silty fine sand
with interbeds of hay/grass

0.76

G

1

1-97.75

G

2

2-96.75

Compact, brown SILTY
fine SAND

G

3

3-95.75

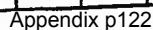
End of Test Pit

3.60

(TP dry upon completion)

20 40 60 80 100
Shear Strength (kPa)

▲ Undisturbed △ Remoulded



SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
D _{xx}	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

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

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

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

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

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

CONSOLIDATION TEST

p' _o	-	Present effective overburden pressure at sample depth
p' _c	-	Preconsolidation pressure of (maximum past pressure on) sample
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)

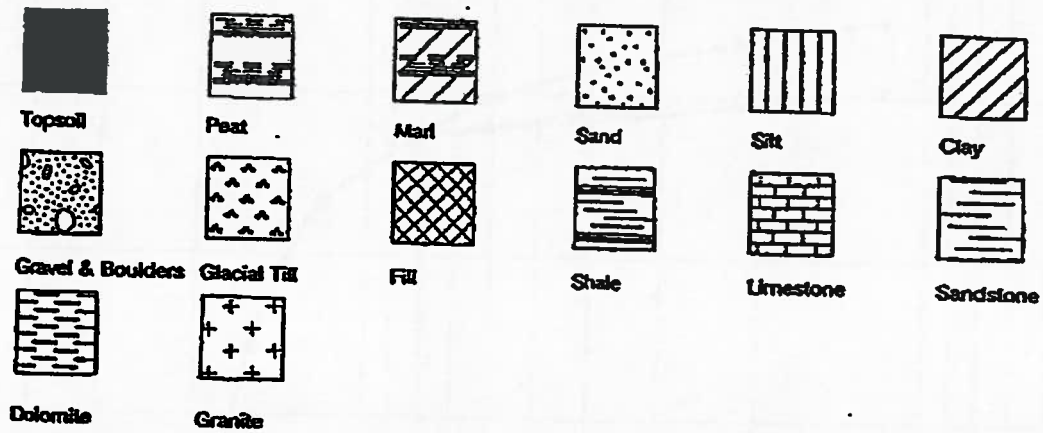
OC Ratio	Overconsolidation ratio =	p'_c / p'_o
Vold Ratio	Initial sample void ratio =	volume of voids / volume of solids
W _o	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

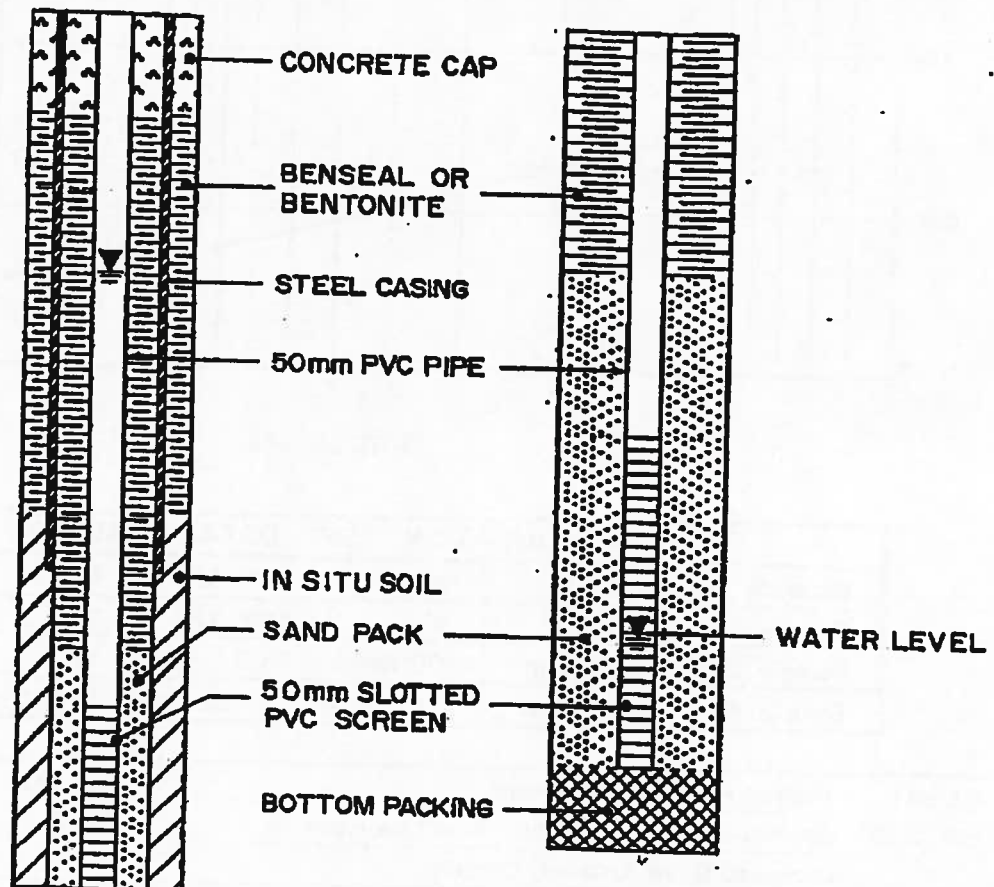
STRATA PLOT

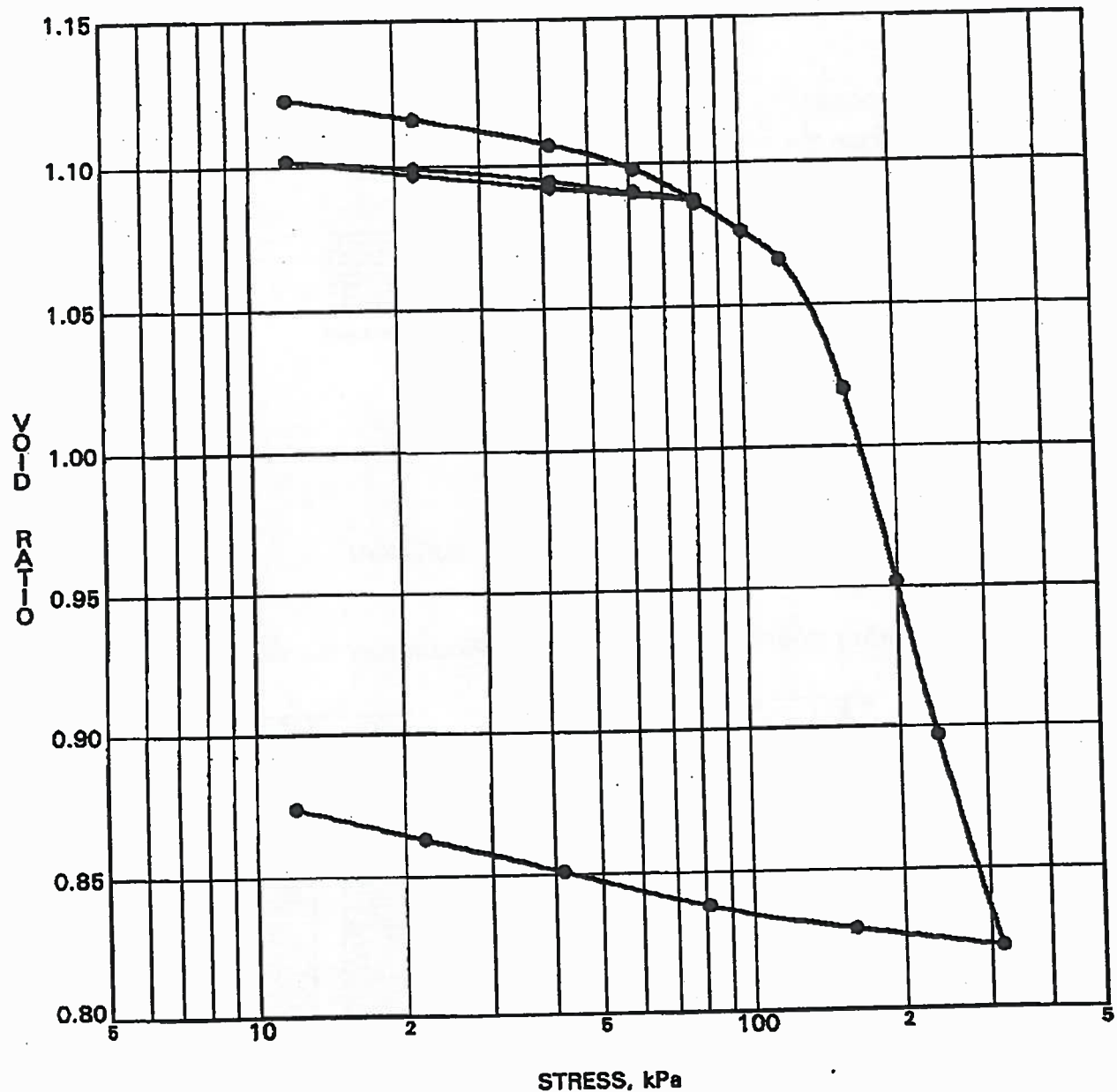


MONITORING WELL AND PIEZOMETER CONSTRUCTION

Monitoring Well Construction

Piezometer Construction





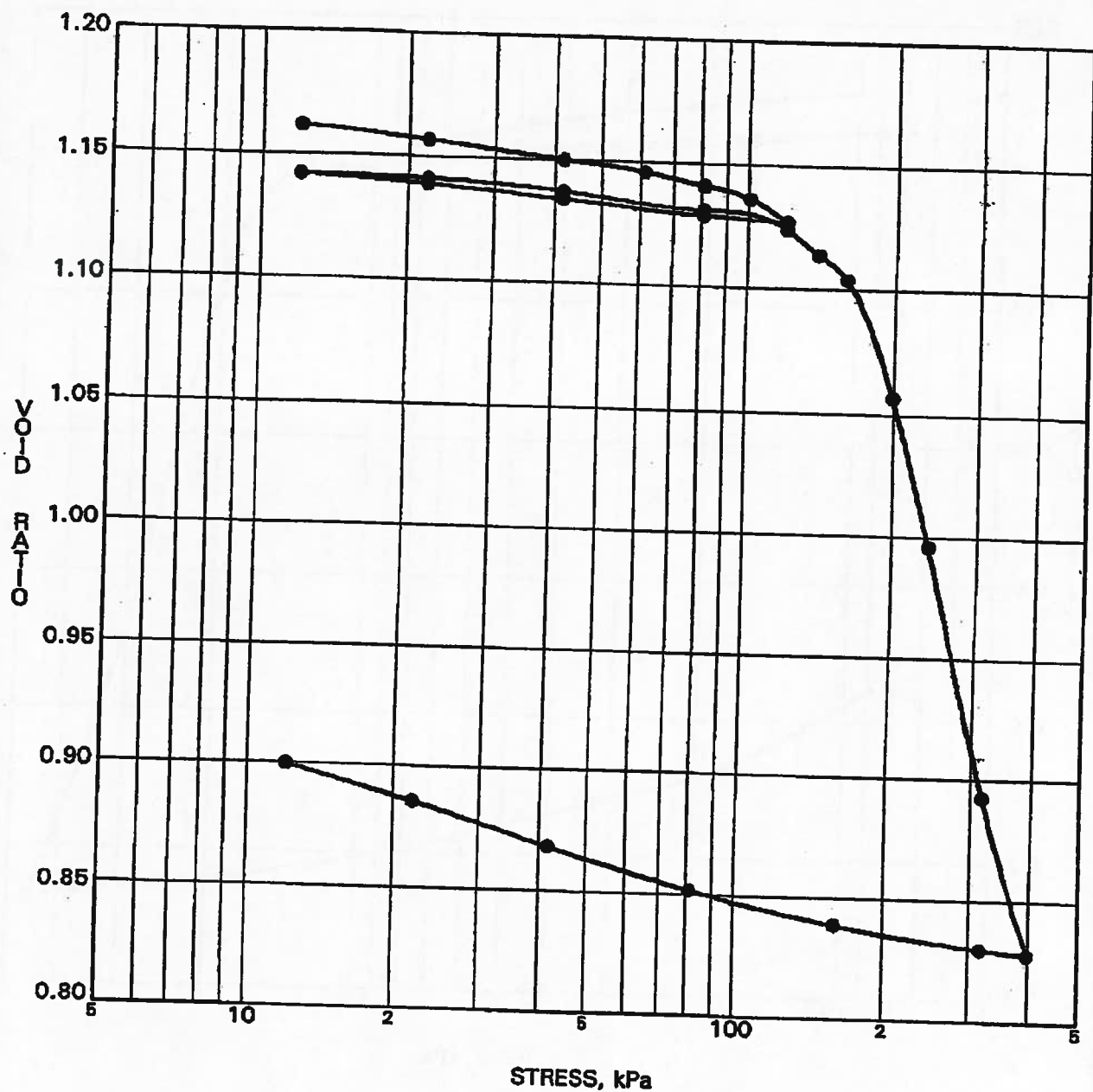
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 2	p'_o	100 kPa	C_{cr}	0.019
Sample No.	TW 1	p'_c	132 kPa	C_c	0.653
Sample Depth	8.08 m	OC Ratio	1.3	W_o	40.5 %
Sample Elev.	88.98 m	Void Ratio	1.132	Unit Wt.	17.9 kN/m ³

CLIENT Taggart Realty Management
 PROJECT Geotechnical Investigation - First Line Road @
Campeau Drive, Ottawa, Ontario

FILE NO. G8689
 DATE 27/11/02



CONSOLIDATION TEST
JOHN D. PATERSON & ASSOCIATES LTD.
 Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



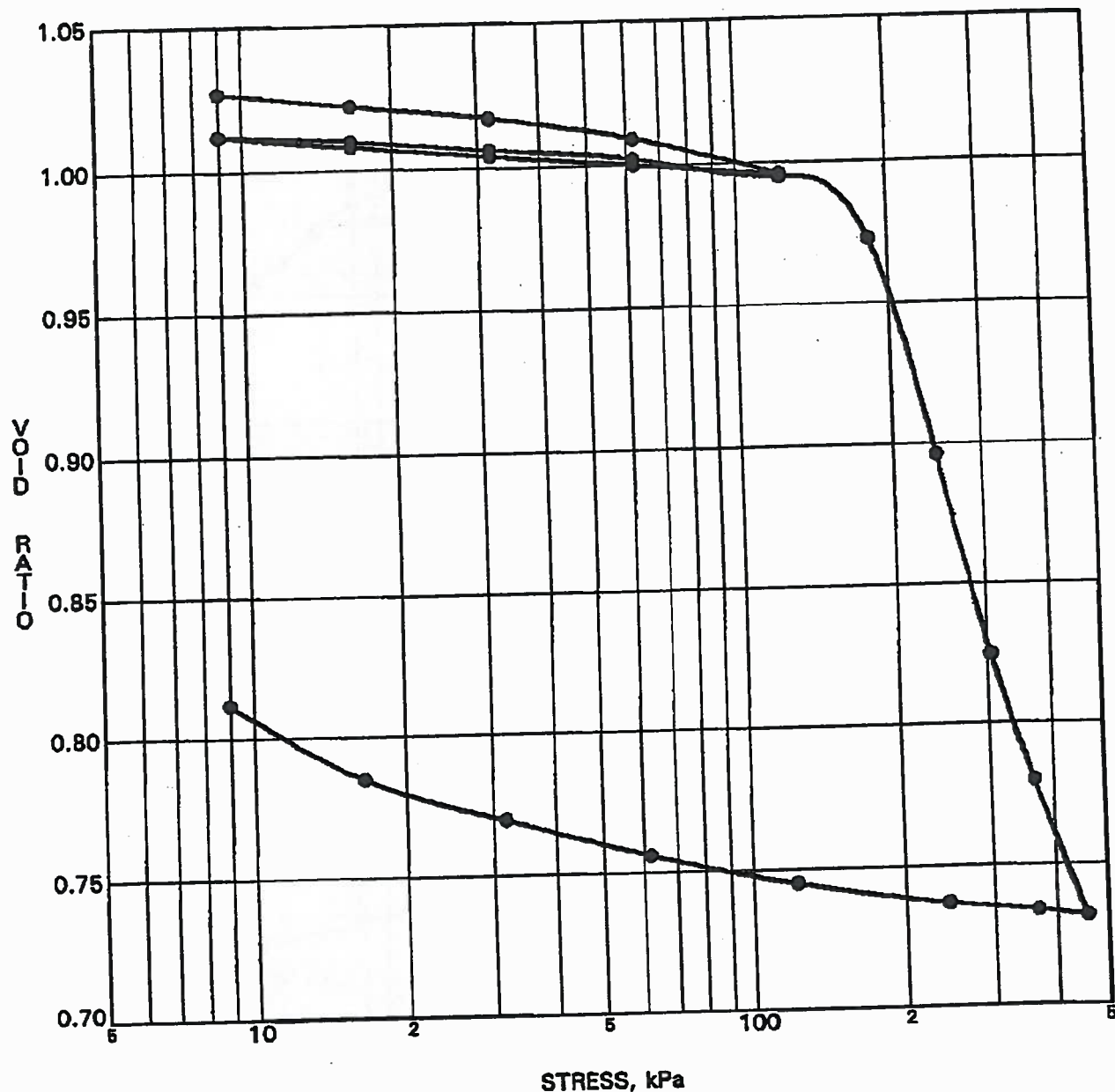
CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 2	p'_o	148 kPa	C_{cr}	0.020
Sample No.	TW 3	p'_c	175 kPa	C_c	0.781
Sample Depth	14.15 m	OC Ratio	1.2	W_o	42.2 %
Sample Elev.	82.91 m	Void Ratio	1.169	Unit Wt.	17.7 kN/m ³

CLIENT Taggart Realty Management
PROJECT Geotechnical Investigation - First Line Road @
 Campeau Drive, Ottawa, Ontario

FILE NO. G8689
DATE 27/11/02



CONSOLIDATION TEST
JOHN D. PATERSON & ASSOCIATES LTD.
 Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 3	p'_o	124 kPa	C_{cr}	0.015
Sample No.	TW 4	p'_o	178 kPa	C_c	0.666
Sample Depth	9.60 m	OC Ratio	1.4	W_o	37.0 %
Sample Elev.	88.06 m	Void Ratio	1.029	Unit Wt.	18.3 kN/m ³

CLIENT Taggart Realty Management
 PROJECT Geotechnical Investigation - First Line Road @
 Campeau Drive, Ottawa, Ontario

FILE NO. G8689
 DATE 27/11/02



CONSOLIDATION TEST
JOHN D. PATERSON & ASSOCIATES LTD.
 Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7

Paracel Laboratories Ltd.**Certificate of Analysis****Order #: H1132****Client: J.D. Paterson and Associates****Report Date: 09/23/02****Client Ref: 7223****Project: G8689****Order Date: 09/16/02****Sample Date: 09/04/02****Matrix: Soil**

Parameter	MDL	TP1 G3
		H1132.1
Chloride	5.0 ug/g	75
Sulphate	5.0 ug/g	55
pH	0.050 pH units	8.1
Resistivity	0.10 ohm.m	41

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING No. G8689-1, TEST HOLE LOCATION PLAN

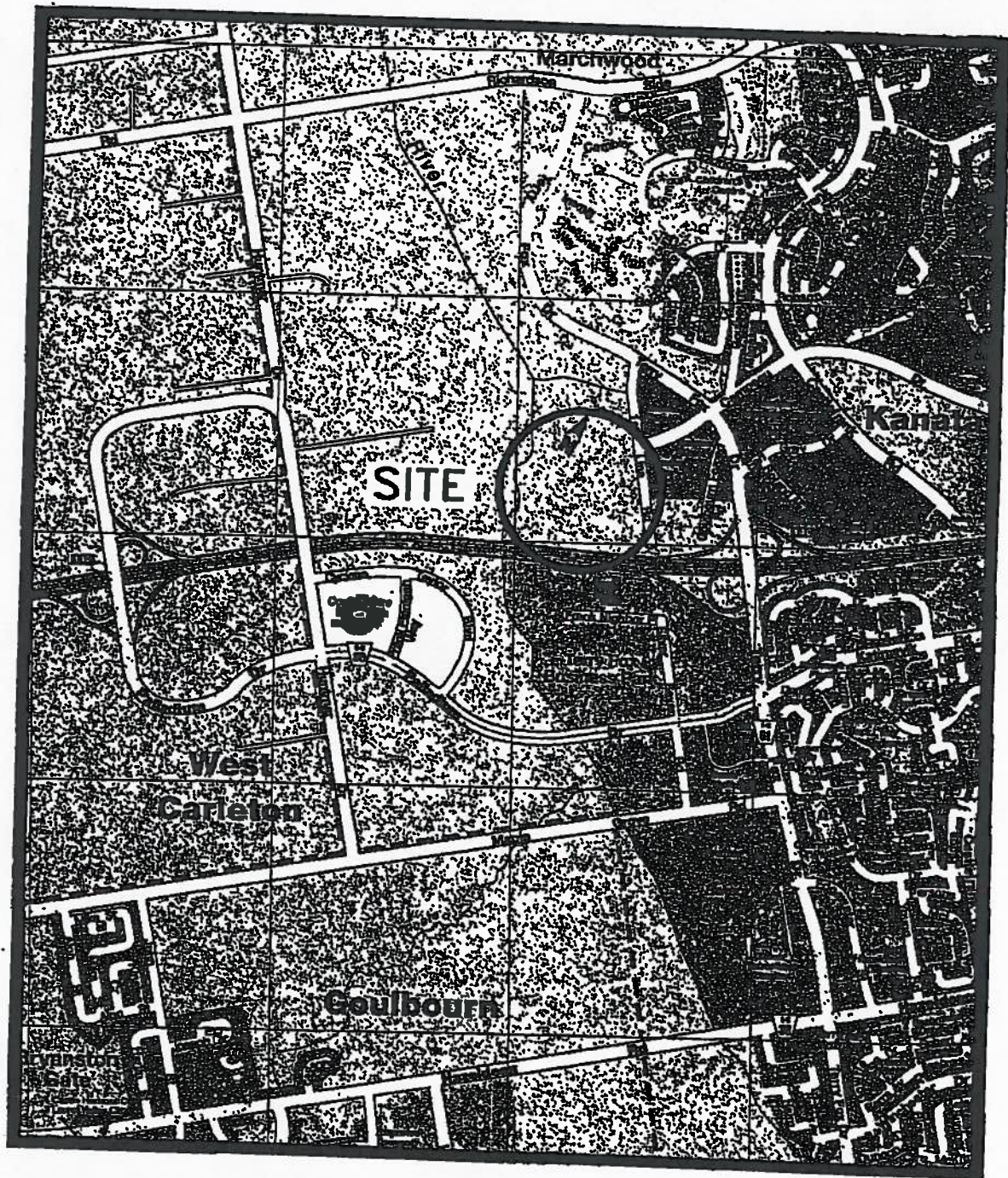


FIGURE 1
KEY PLAN

Appendix p131



JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

REMARKS

FILE NO.

G8689

BORINGS BY Backhoe

DATE 5 SEP 02

HOLE NO.

TP16

SOIL DESCRIPTION

STRATA PLOT

SAMPLE

DEPTH
(m)

ELEV.
(m)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

PIEZOMETER
CONSTRUCTION

GROUND SURFACE

TOPSOIL

0.10

FILL: Brown silty fine sand
with gravel

0.60

End of Test Pit

(TP dry upon completion)

TYPE

NUMBER

%
RECOVERY

N VALUE
or RQD

G

1

0-98.93

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed Δ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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



JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

REMARKS

FILE NO.

G8689

BORINGS BY Backhoe

DATE 5 SEP 02

HOLE NO.

TP16

SOIL DESCRIPTION

STRATA PLOT

SAMPLE

DEPTH
(m)

ELEV.
(m)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ **Water Content %**

**PIEZOMETER
CONSTRUCTION**

GROUND SURFACE

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0.10

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with gravel

G

1

End of Test Pit

0.60

(TP dry upon completion)

0-98.93

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SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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CONSOLIDATION TEST

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p' _o	-	Preconsolidation pressure of (maximum past pressure on) sample
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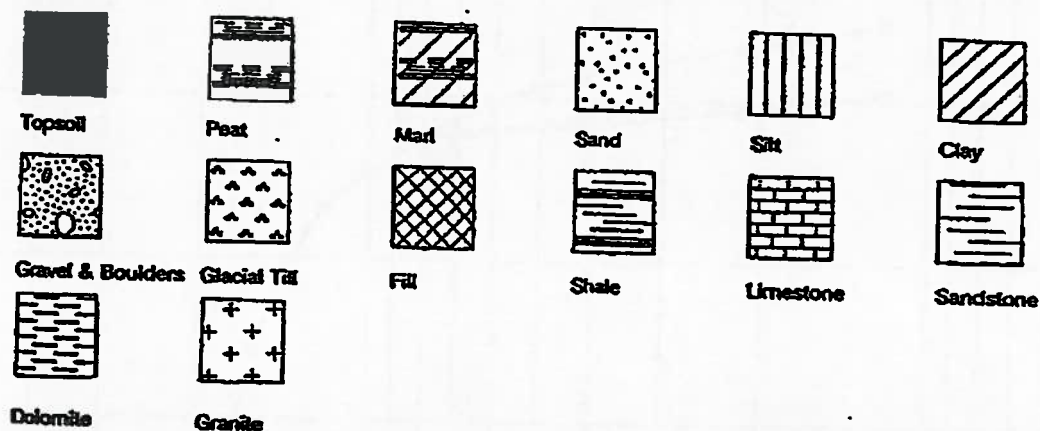
OC Ratio	Overconsolidation ratio =	p'_o / p'_o
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PERMEABILITY TEST

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SYMBOLS AND TERMS (continued)

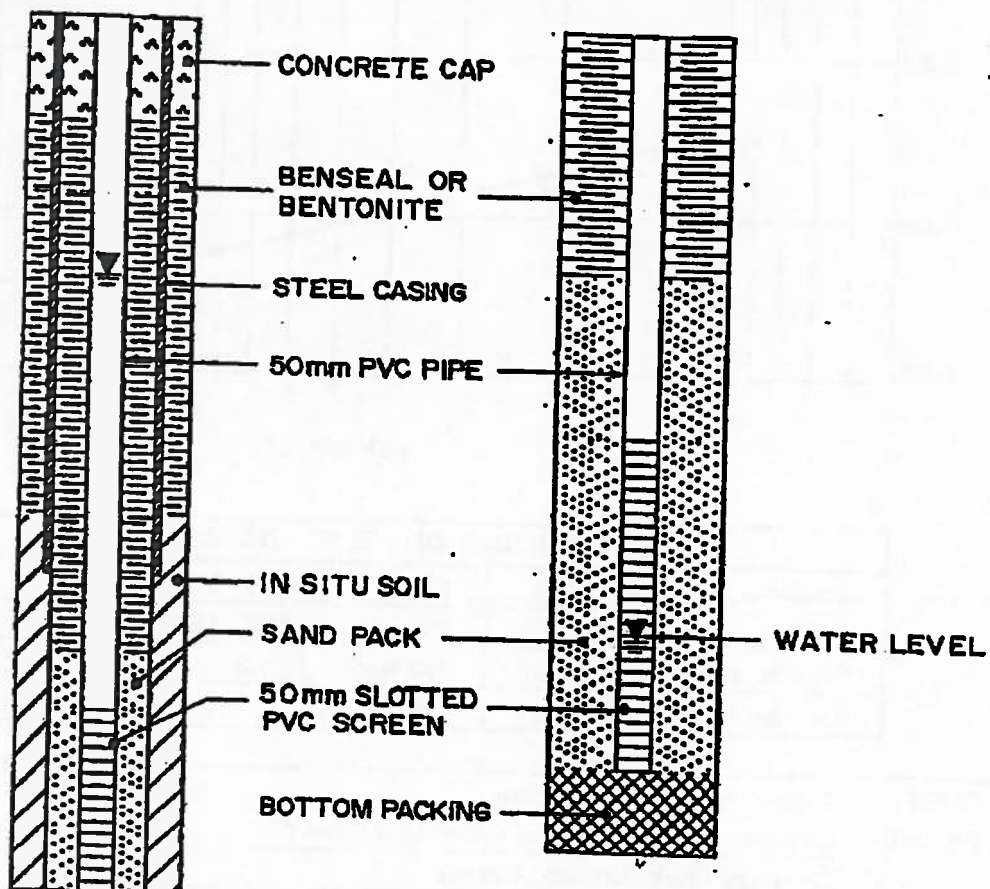
STRATA PLOT

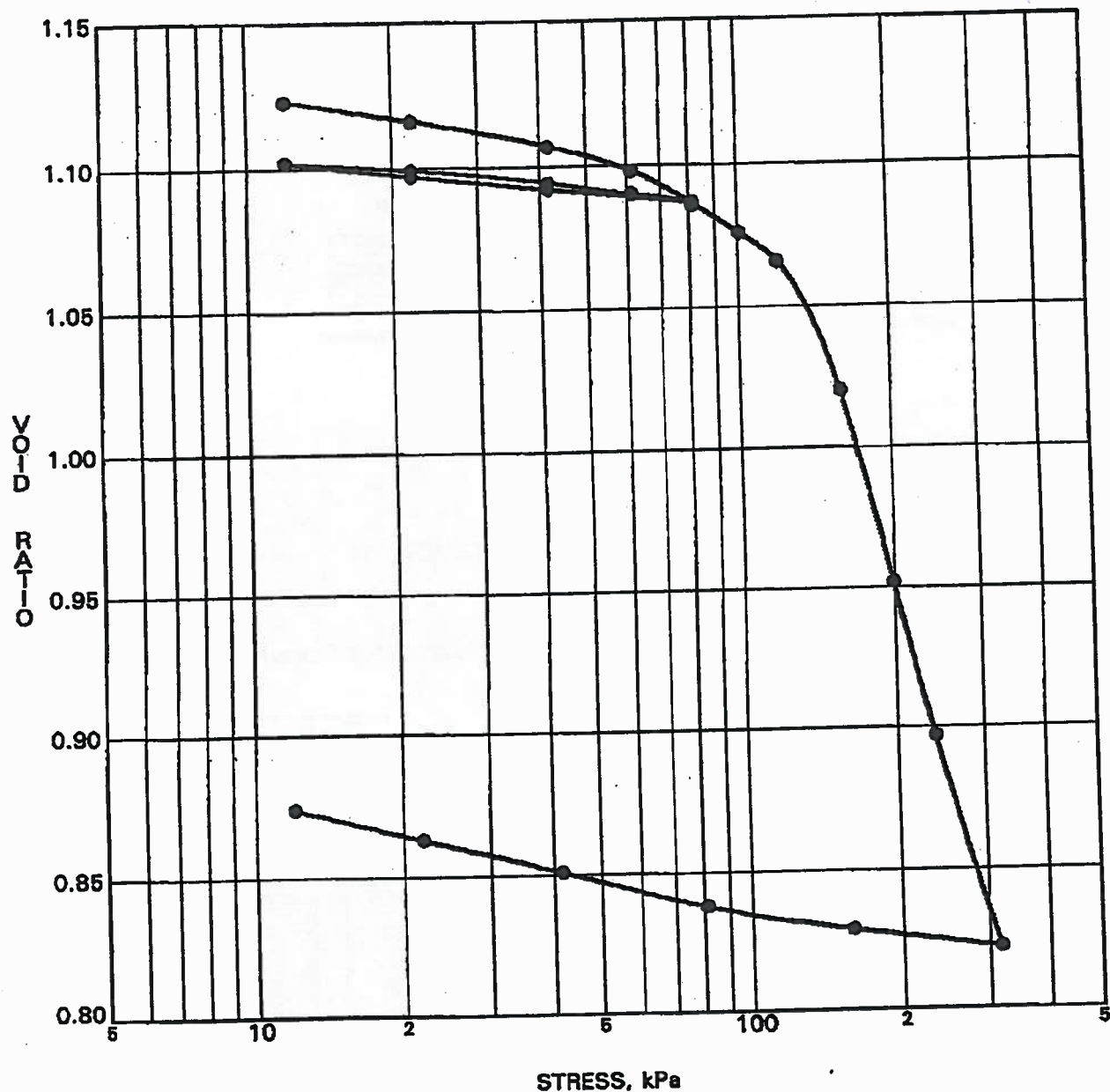


MONITORING WELL AND PIEZOMETER CONSTRUCTION

Monitoring Well Construction

Piezometer Construction





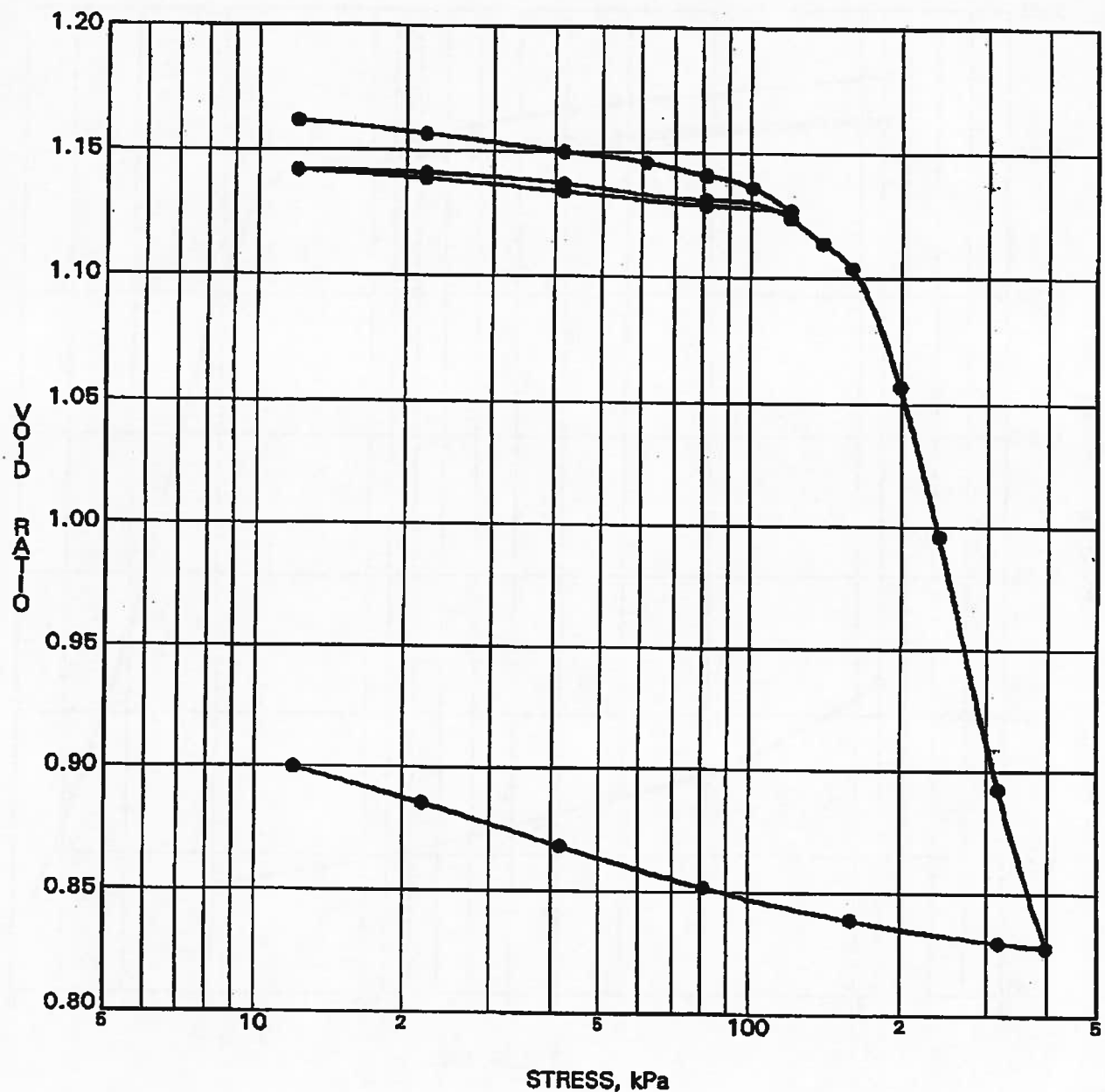
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Sample No.	TW 1	p'_c	132 kPa	C_c	0.653
Sample Depth	8.08 m	OC Ratio	1.3	W_o	40.5 %
Sample Elev.	88.98 m	Void Ratio	1.132	Unit Wt.	17.9 kN/m ³

CLIENT Taggart Realty Management
 PROJECT Geotechnical Investigation - First Line Road @
Campeau Drive, Ottawa, Ontario

FILE NO. G8689
 DATE 27/11/02



CONSOLIDATION TEST
JOHN D. PATERSON & ASSOCIATES LTD.
 Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



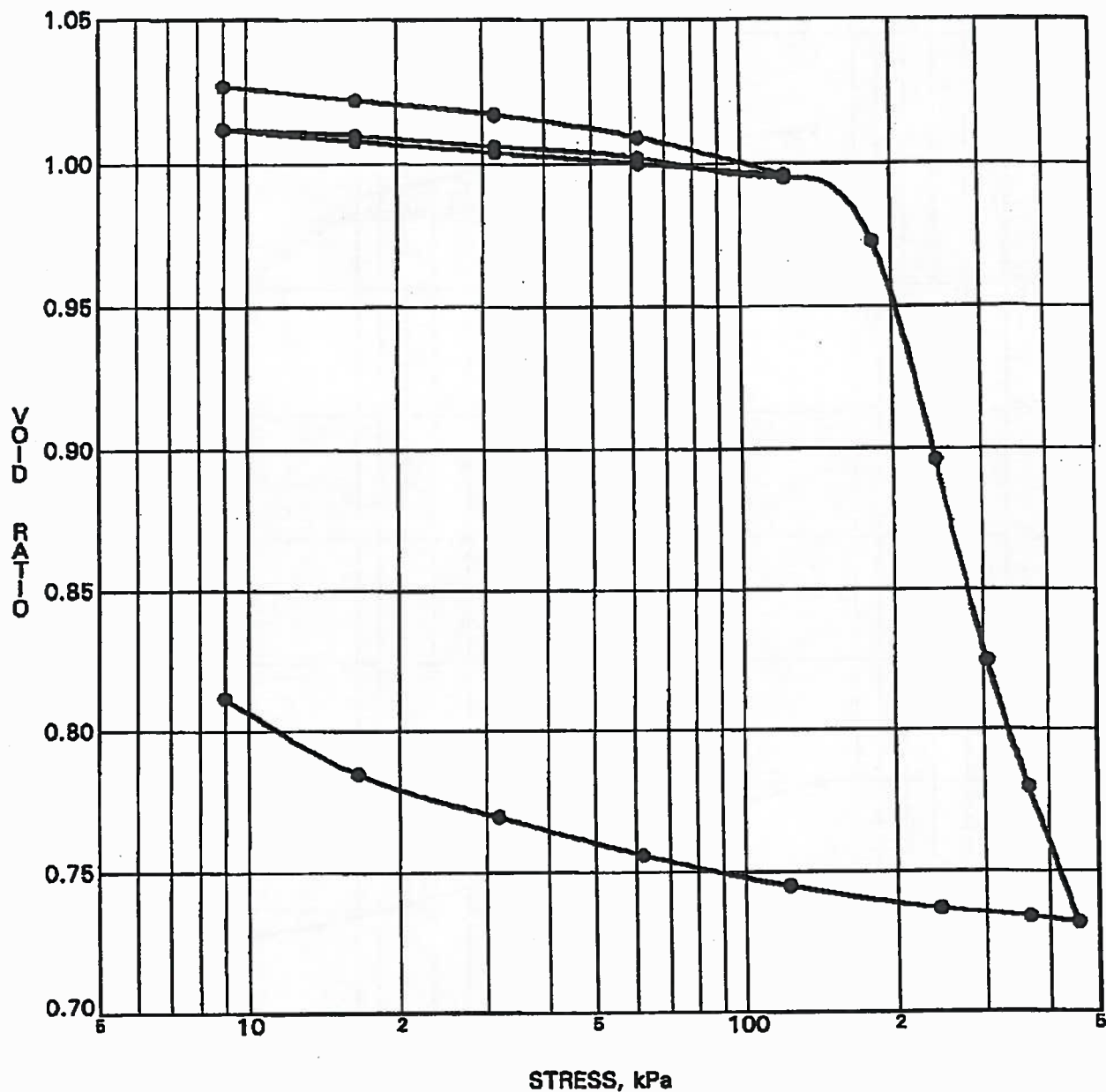
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Borehole No.	BH 2	p'_o	148 kPa	C_{cr}	0.020
Sample No.	TW 3	p'_c	175 kPa	C_c	0.781
Sample Depth	14.15 m	OC Ratio	1.2	W_o	42.2 %
Sample Elev.	82.91 m	Void Ratio	1.169	Unit Wt.	17.7 kN/m ³

CLIENT Taggart Realty Management
 PROJECT Geotechnical Investigation - First Line Road @
Campeau Drive, Ottawa, Ontario

FILE NO. G8689
 DATE 27/11/02



CONSOLIDATION TEST
JOHN D. PATERSON & ASSOCIATES LTD.
 Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 3	p'_o	124 kPa	C_{cr}	0.015
Sample No.	TW 4	p'_o	178 kPa	C_c	0.666
Sample Depth	9.60 m	OC Ratio	1.4	W_o	37.0 %
Sample Elev.	88.06 m	Void Ratio	1.029	Unit Wt.	18.3 kN/m ³

CLIENT Taggart Realty Management
 PROJECT Geotechnical Investigation - First Line Road @
 Campeau Drive, Ottawa, Ontario

FILE NO. G8689
 DATE 27/11/02



CONSOLIDATION TEST
JOHN D. PATERSON & ASSOCIATES LTD.
 Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7

Paracel Laboratories Ltd.**Order #: H1132****Certificate of Analysis**

Report Date: 09/23/02

Client: **J.D. Paterson and Associates**

Order Date: 09/16/02

Client Ref: 7223

Project: G8689

Sample Date: 09/04/02

Matrix: Soil

TP1 G3

Parameter

MDL

H1132.1

Chloride	5.0 ug/g	75
Sulphate	5.0 ug/g	55
pH	0.050 pH units	8.1
Resistivity	0.10 ohm.m	41

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING No. G8689-1, TEST HOLE LOCATION PLAN

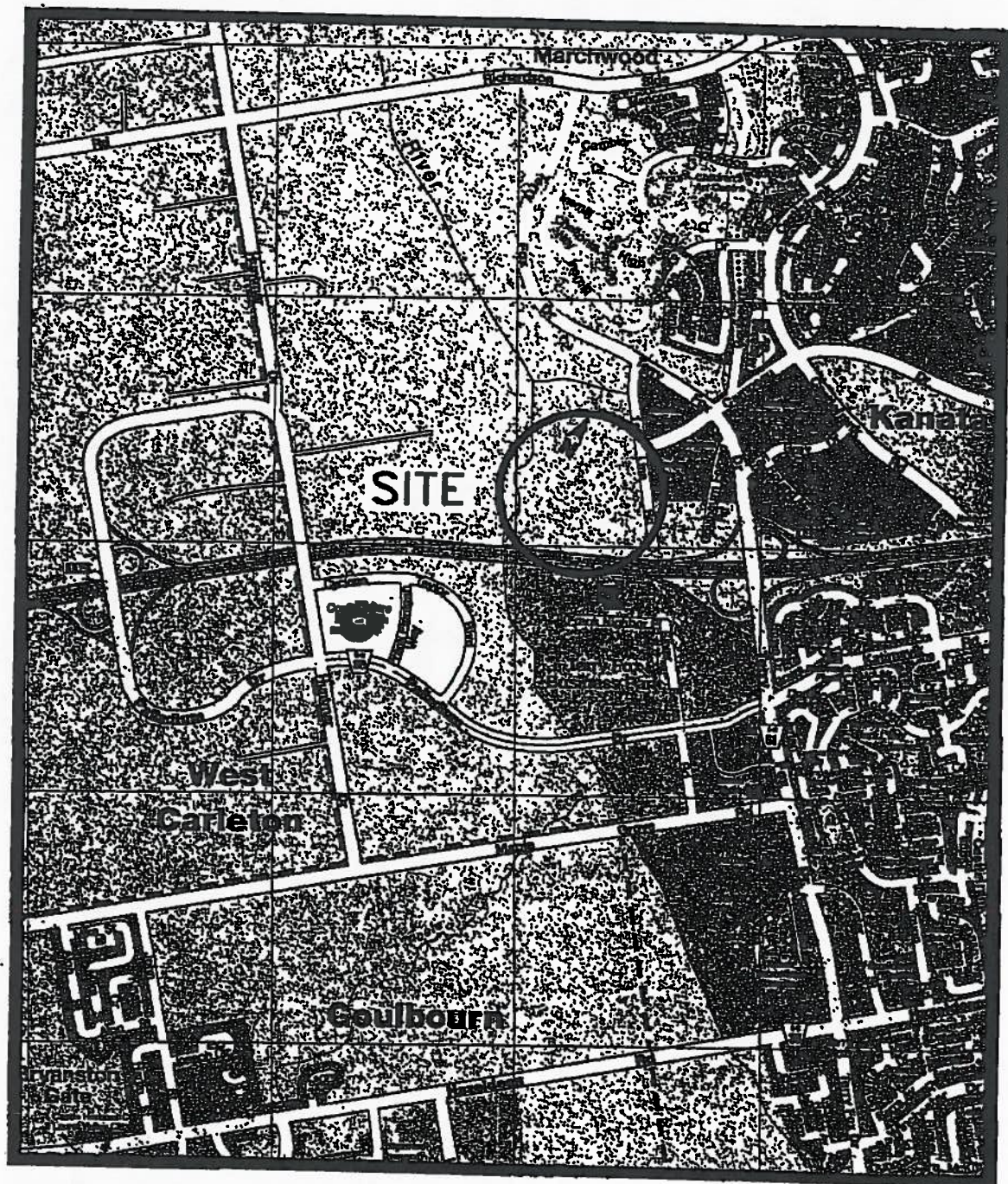
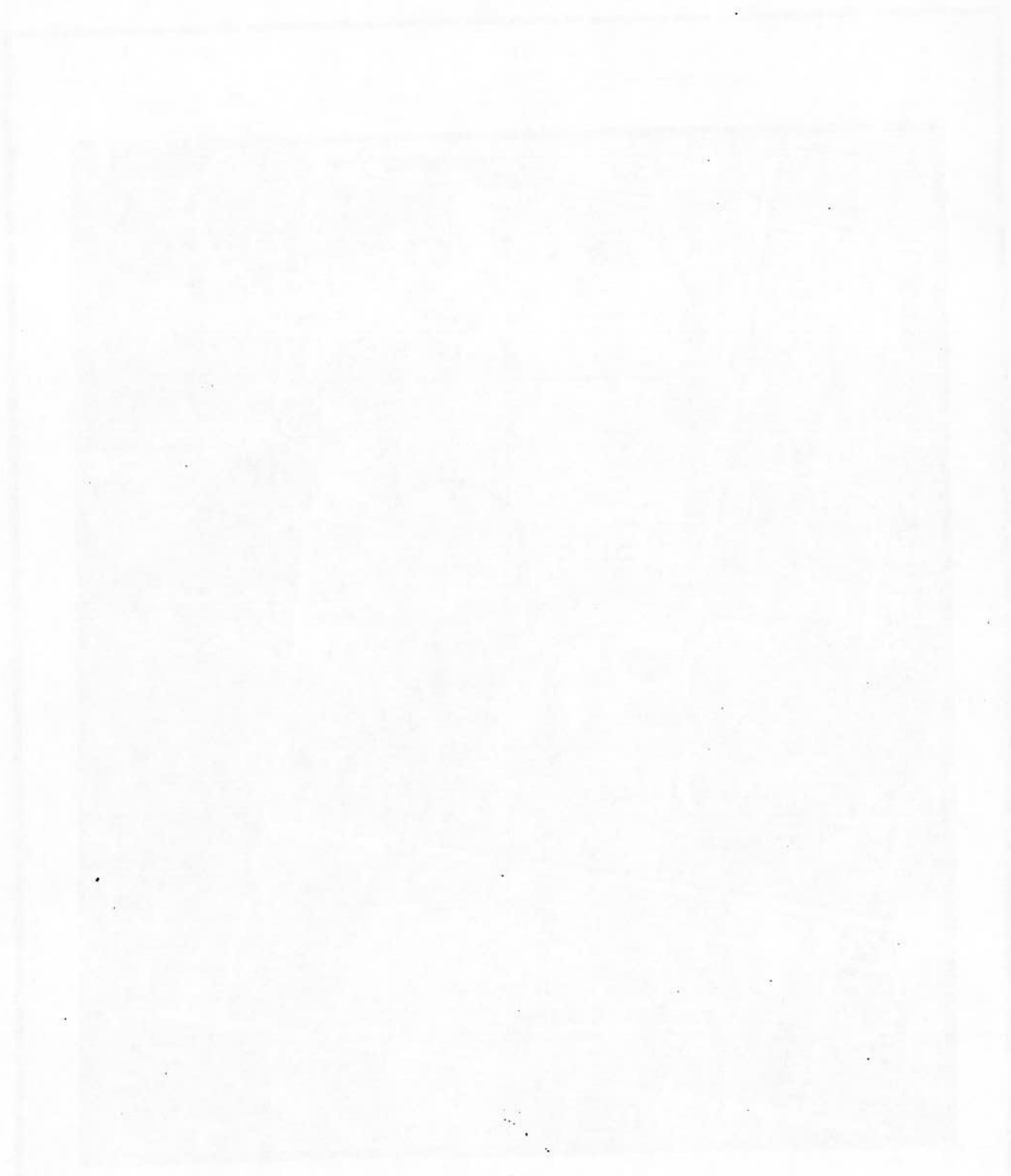


FIGURE 1
KEY PLAN



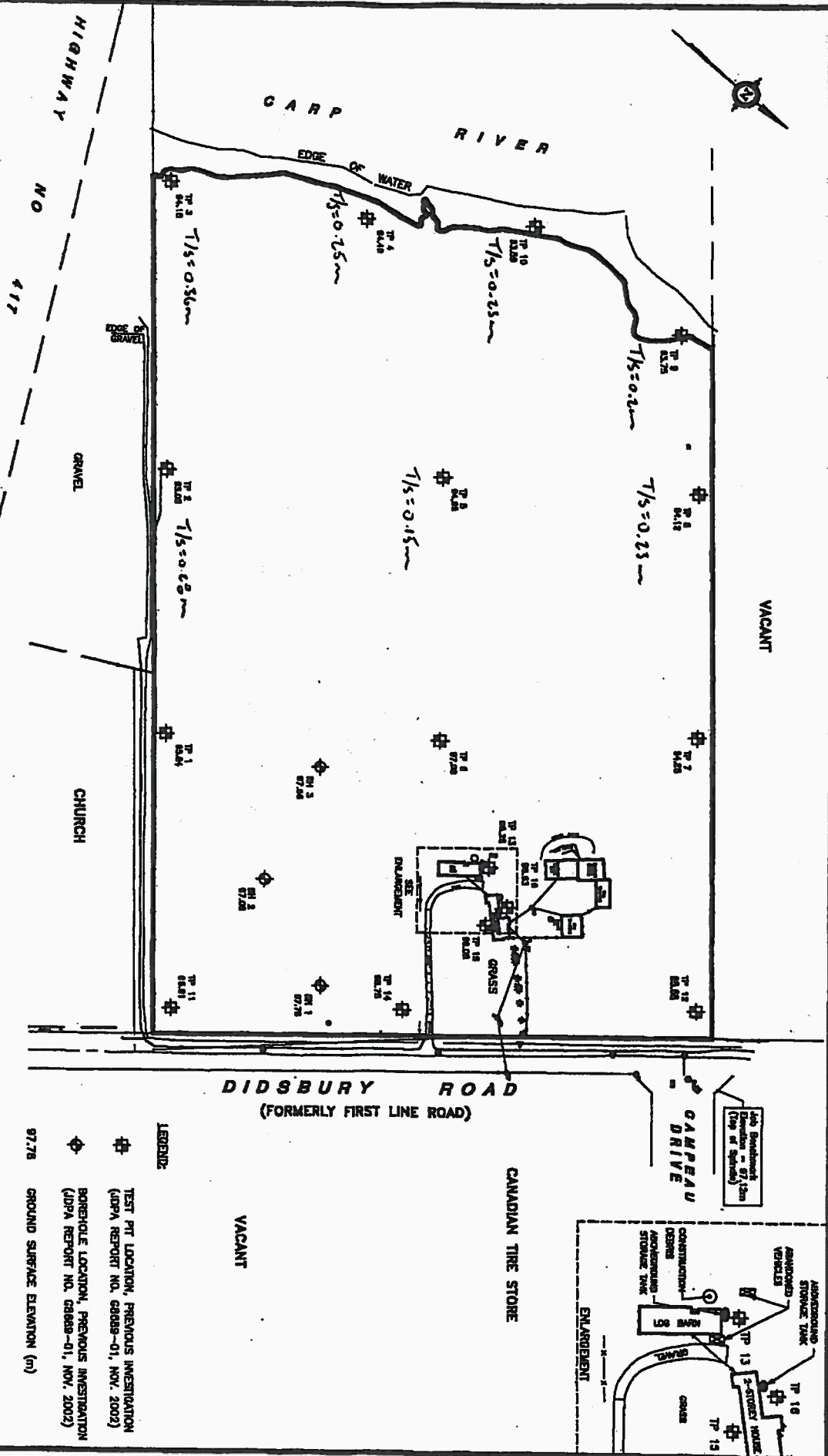
paterson group
 consulting engineers
 20 Concordia Court, Unit 1, Ottawa, Ontario K2E 7T7

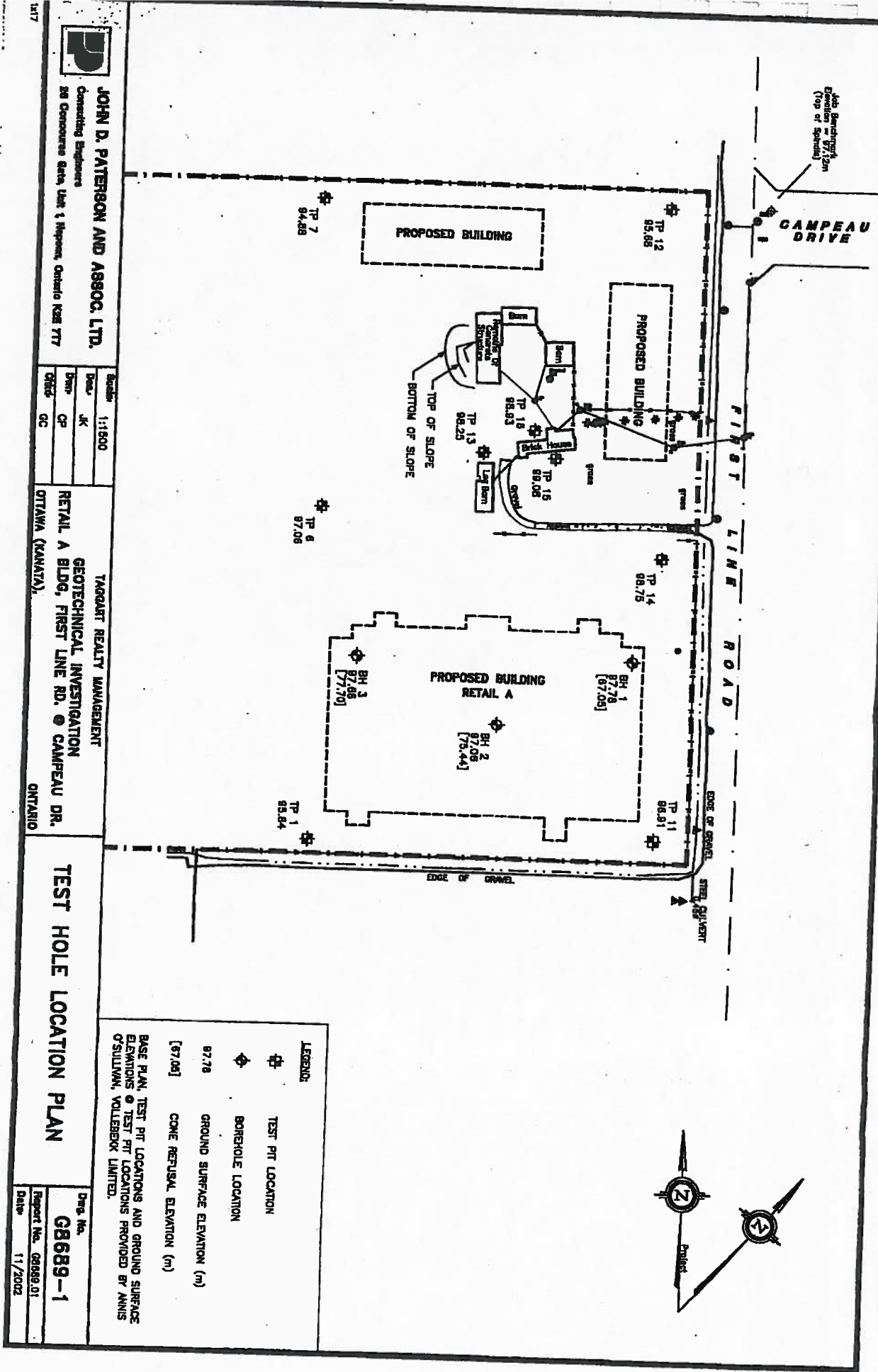
Scale: 1:2000
 Date: DC
 Drawn: LPO
 Checked: MSD

TAGCART REALTY MANAGEMENT
 PHASE I-ENVIRONMENTAL SITE ASSESSMENT
 DIDSBURY ROAD @ CAMPEAU DRIVE
 OTTAWA (K1A1A1), ONTARIO

SITE PLAN

Draw. No.: PE0147-1
 Report No.: PE0147-01
 Date: 03/2004





Toronto Inspection Ltd.

GEO-ENVIRONMENTAL CONSULTANTS

**REPORT ON
GEOTECHNICAL INVESTIGATION
PROPOSED RETAIL STORE AND GAS STATION
FIRST LINE ROAD & CAMPEAU ROAD
NORTH KANATA, ONTARIO**

**REPORT NO.: 03 LPL 782
REPORT DATE: APRIL 7, 2003**

**PREPARED FOR
MR. KEN HILDEBRANDT
LOBLAW PROPERTIES LIMITED
3563 LAKESHORE BLVD. WEST
TORONTO, ONTARIO
M8W 1P4**

**10 Konrad Crescent, Unit 16, Markham, Ontario . 3R 9X
Tel.: (905) 940-8509 Fax: (905) 940-8192**

Toronto Inspection Ltd.

GEO-ENVIRONMENTAL CONSULTANTS

110 KONRAD CRESCENT,
UNIT 16
MARKHAM, ONTARIO.
L3R 9X2
Tel.: 905-940-8509
Fax.:905-940-8192

April 7, 2003
Report No.: 03 LPL 782

Loblaw Properties Limited
3563 Lakeshore Boulevard West
Toronto, Ontario.
M8W 1P4

Attention: Mr. Ken Hildebrandt

**Re: Proposed Retail Store and Gas Station
First Line Road and Campeau Road
North Kanata, Ontario**

We are pleased to provide a report on geotechnical investigations carried out at the above site. The purpose of the investigations was to provide design and construction recommendations for the proposed retail store and a gas station.

A geotechnical investigation was carried out at the subject site on February 24, 2003 and consisted of eight sampled boreholes to depths varying from 3.5 to 8.1 m below the existing ground level. Boreholes Numbers 1 to 5 were located within the proposed retail building area, Borehole 6 was located within the proposed parking lot and Boreholes Numbers 7 and 8 were located in the location of the proposed gas station.

Topsoil varying in depth from 150 to 450 mm was encountered at the ground surface of the boreholes. The native subsoils below the topsoil consisted of sand, clayey silt and alluvium deposits.

Based on the standard penetration N-values, the sand was compact to very loose, generally in the loose to very loose range. The moisture content of the sand deposit varied from 8 to 10% above the water table and more than 20% in the saturated zone.

The clayey silt and the alluvium deposits were generally very soft. The moisture contents of the soil samples retrieved from these deposits ranged from 25% to more than 40%. Seven of the eight boreholes were terminated in the alluvium deposit at depths varying from 6.2 to 8.1 m below the existing ground level. Borehole Number 6 in the parking lot was terminated at a depth of 3.5 m below the existing ground level.

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Free water was encountered at seven of the eight boreholes at depths varying from 2.6 to 4 m below the existing ground level. Based on the field observations and the moisture content profile of the subsoils obtained from the boreholes, it is our opinion that there is a continuous ground water table at the subject site. The stabilised ground water table was, however, not established at the site.

We understand that the proposed development will consist of a single storey framed building without a basement in the south and a gas station in the north part of the site. The slab-on-grade elevations of the proposed structures were not known at the time of preparing this report.

Based on the subsoil information obtained at the borehole locations, it is our opinion that the native sand, clayey silt and the alluvium deposits are normally consolidated, with little or no preconsolidation pressures. Development loads in excess of the available preconsolidation pressure will result in unacceptable settlement. Therefore, the cumulative effects of any additional fill, slab-on-grade loading, footing pressures and the long term lowering of the ground water table must be taken into account for the determination of the foundation bearing pressures. Additional deeper boreholes will be needed to determine the depth of the very loose deposits at the site. In addition, unidirectional consolidation tests will have to be carried out on relatively undisturbed samples from the site to determine the available preconsolidation pressures in the subsoils.

With the limited subsoil information available at the site, the following are the preliminary foundation alternatives:

Preload the site to at least 150% of the total anticipated load (dead and live loads) of the proposed development. In order to speed up the consolidation process, vertical wick drains should be installed in a grid pattern, three metres or less on centres. The wick drains should extend to the hard strata, underlying the very soft clayey silt / alluvium deposits, at an estimated depth of 30 m below the existing ground level. This depth should be confirmed by additional deep boreholes at the subject site.

The preload area should extend approximately 3 m beyond the envelope of the proposed structures. The potential effect of the preload and the lowering of the water table on the adjacent properties must be taken into consideration.

Under the preload pressure, the settlement of the sand deposits will be almost instantaneous. However, the primary consolidation of the silty clay and the clayey silt deposits can take up to 6 months, depending on the permeability of these deposits. Field monitoring of the ground settlement and the excess pore water pressures in the subsoils will have to be carried out on weekly intervals during the initial stages. The monitoring frequency will be scaled back to

bi-weekly intervals after the first month. These monitoring results will be analysed to determine when the primary settlement has been achieved.

If the preload, below the founding elevation, is placed and compacted as an engineered fill, footings for the proposed buildings can be founded in the engineered fill, after the consolidation of the subsoils has been completed. A preliminary design bearing pressure of 150 kPa can be considered for footings founded in the engineered fill, compacted to at least 100% of its standard Proctor maximum dry density.

Use driven piles as the foundations for the proposed retail building. H piles or 250 mm diameter pipe piles, driven closed ended to a set of 20 blows for 25 mm penetration using an energy of 30000 ft pounds (41 kJ), can be designed for an allowable axial bearing capacity of 1000 kN. This capacity is based on the pipe piles having minimum wall thicknesses of 13 mm and being filled, after completion of driving, with 30 MPa concrete. The design bearing capacity of 1000 kN per pile has taken into account estimated dragdown forces of 250 kN.

All horizontal loads on the structure should be resisted by battered piles.

If any fill is added to the site, consideration should be given to using a structural slab in conjunction with the deep foundations.

The ground elevations at the borehole locations were not determined. The report will be updated after the topographic survey plan is available and additional subsoil investigation and lab testing, detailed in Section 5.0 of the report are carried out.

All perimeter foundations or grade beams for the proposed retail building and the Gas Station canopy will have to be placed below the frost penetration depth of 1.5 m below the outside grade. Interior column footings within the heated building can be placed at or below a depth of 1 m from the existing ground level.

The subsoil at the depths at the two boreholes (numbers 7 and 8) carried out in the area of the proposed gas station, consist of a loose sand deposit. Spread footings founded in the sand deposit should be designed for a bearing pressure of 100 kPa.

An alternative to spread / strip footings on native soil, the gas bar and kiosk structures can be founded on a concrete pad supported on the grade. It is our opinion that a conventional slab-on-grade construction, with at least 150 mm of granular A (OPSS form 1010) or 20 mm crusher run limestone

below the slab as a moisture barrier, will be feasible for the gas station.

Ground water condition can be anticipated during excavations deeper than 2.5 m below the ground level at the location of the gas station. Positive dewatering of the sand deposit will be necessary if excavation of more than 2.5 m is needed for the underground storage tanks.

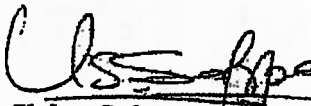
We trust that you will find this report complete within our terms of reference. If you have any questions regarding the above, please call this office.

Yours sincerely

TORONTO INSPECTION LTD.



Bennett C. Sun, P. Eng.
Senior Geotechnical Engineer



Upkar S. Sappal, P. Eng.
Principal Engineer



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ENCLOSURES:

Borehole Location Plan
Records of Boreholes 1 to 8
Subsurface Stratigraphy
Insulation of Subgrade

Drawing No. 1
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1.0 INTRODUCTION

Toronto Inspection Ltd. was retained by Loblaw Properties Limited to conduct a geotechnical investigation for the proposed retail store and a gas station at a site located on the west side of First Line Road and south of Campeau Road in North Kanata, Ontario.

The purpose of the investigation was to determine the subsoil and groundwater conditions affecting the design and construction of a single storey retail building without a basement and a gas station. In particular geotechnical data was to be provided for:

- General founding conditions
- Foundation design bearing pressures
- Construction recommendations
- Excavation recommendations

The report is provided on the basis of the above terms of reference and on the assumption that the design of the buildings will be in accordance with the applicable building codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, our office should be consulted to review the design and to confirm the recommendations and comments provided in the report.

2.0 INVESTIGATION PROCEDURE

The field work for the initial investigation was carried out on February 24, 2003 and consisted of five sampled boreholes (Numbers 1 to 5) within the location of the proposed retail store, in addition to two boreholes (Numbers 7 and 8) at the location of the proposed gas station. Borehole Number 6 was carried out within the proposed parking lot.

The locations of the previous and the current boreholes are shown on the appended Borehole Location Plan (Drawing No. 1).

The boreholes were advanced using a track mounted drill rig equipped with continuous flight solid stem augers supplied by a specialist drilling contractor, under the full time supervision of a member of our engineering staff.

Samples were taken at regular intervals using a split spoon sampler in conjunction with Standard Penetration Tests using a driving energy of 475 joules (350 ft. lbs). The samples were identified and logged in the field and were carefully bagged for later visual

identification and laboratory water content determination. All boreholes were terminated at depths of 3.5 to 8.1 m below the existing ground level.

Groundwater observations were made in the boreholes during and following the completion of drilling.

The ground elevations at the borehole locations were not determined. The report will be updated after the topographic survey plan is available and additional subsoil investigation and lab testing as detailed in Section 5.0 of the report are carried out.

3.0 SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets and Drawing 1 for details of field work, including soil classification, inferred stratigraphy, groundwater observations carried out during and on completion of drilling in the open boreholes and the results of the laboratory water content determinations.

Topsoil varying in depth from 150 to 450 mm was encountered at the ground surface at the borehole locations. The native subsoils below the topsoil consisted of sand, clayey silt and alluvium deposits.

3.1 Clayey Silt

Underlying the topsoil at Boreholes 2, 3 and 5, and the sand deposit at Borehole 4, a deposit of clayey silt was encountered. The lower limit of the clayey silt deposit was at a depth of 2.4 m at Boreholes 2, 3 and 5 and 3.2 at Borehole 4.

The clayey silt deposit was very fine to fine grained material of low plasticity to almost non plastic. Based on the standard penetration N-values, the clayey silt deposit was firm to very soft; generally in the soft to very soft range. Occasional fragments of sea shells were encountered within the clayey silt deposit.

The in-situ moisture content of the deposit varied from 20 to more than 40%, indicating moist to saturated conditions.

3.2 Sand

Underlying the topsoil at Boreholes 1, 4, 6, 7 and 8, and the clayey silt at Boreholes 2, 3 and 5, a deposit of very fine to fine grained sand was encountered. At Borehole 4, a lower deposit of sand was encountered below the clayey silt.

Based on the standard penetration N-values, the sand was compact to very loose, generally in the loose to very loose range. The moisture content of the sand deposit varied from 8 to 10% above the water table and more than 20% in the saturated zone.

Alluvium

With the exception of Borehole 6, an alluvium deposit was encountered below the sand at all borehole locations. The alluvium deposit consisted of clay and silt layers with traces of sea shells and occasional fine sand layers.

Based on the N-values, the alluvium deposit was generally very soft. The in-situ moisture content of the deposit was more than 30%; indicating saturated conditions.

With the exception of Borehole 6, all boreholes were terminated in this deposit at depths varying from 6.6 to 8.1 m below the existing ground level. The lower limit of the deposit was not determined.

Ground Water

Free water was encountered at the borehole locations during drilling and on completions at depths varying from 2.6 to 4 m below the existing ground level. Borehole Number 6 cave-in at a depth of 3.0 m on completion and ground water depth was not recorded.

4.0 RECOMMENDATIONS

Project Data

We understand that the proposed development will consist of a single storey framed retail building without a basement in the south part and a Gas Station in the north part of the site. The slab-on-grade elevations of the proposed structures were not known at the time of preparing this report.

Foundation Design

The subsoil data at the borehole locations indicate that the native sand, clayey silt and the alluvium deposits are normally consolidated, with little or no preconsolidation pressures. Development loads in excess of the available preconsolidation pressure will result in unacceptable settlement. Therefore, the cumulative effects of any additional fill, slab-on-grade loading, footing pressures and the long term lowering of the ground water table must be taken into account for the determination of the foundation bearing pressures. Additional deeper boreholes will be needed to determine the depth of the very loose deposits at the site. In addition, unidirectional consolidation tests will have to be carried out on relatively undisturbed samples from the site to determine the available preconsolidation pressures in the subsoils.

With the limited subsoil information available at the site, the following are the preliminary foundation alternatives:

- Preload the site to at least 150% of the total anticipated load (dead and live loads) of the proposed development. In order to speed up the consolidation process, vertical wick drains should be installed in a grid pattern, three metres or less on centres. The wick drains should extend to the hard strata, underlying the very soft clayey silt / alluvium deposits, at an estimated depth of 30 m below the existing ground level. This depth should be confirmed by additional deep boreholes at the subject site.

The preload area should extend approximately 3 m beyond the envelope of the proposed structures. The potential effect of the preload and the lowering of the water table on the adjacent properties must be taken into consideration.

Under the preload pressure, the settlement of the sand deposits will be almost

instantaneous. However, the primary consolidation of the silty clay and the clayey silt deposits can take upto 6 months, depending on the permeability of these deposits. Field monitoring of the ground settlement and the excess pore water pressures in the subsoils will have to be carried out on weekly intervals during the initial stages. The monitoring frequency will be scaled back to bi-weekly intervals after the first month. These monitoring results will be analysed to determine when the primary settlement has been achieved.

If the preload, below the founding elevation, is placed and compacted as an engineered fill, footings for the proposed buildings can be founded in the engineered fill, after the consolidation of the subsoils has been completed. A preliminary design bearing pressure of 150 kPa can be considered for footings founded in the engineered fill, compacted to at least 100% of its standard Proctor maximum dry density.

Use driven piles as the foundations for the proposed retail building. 250 mm diameter pipe piles, driven closed ended to a set of 20 blows for 25 mm penetration using an energy of 30000 ft pounds (41 kJ), can be designed for a net allowable axial bearing capacity of 1000 kN. This capacity is based on the pipe piles having minimum wall thicknesses of 13 mm and being filled, after completion of driving, with 30 MPa concrete. The design bearing capacity of 1000 kN per pile takes into account an estimated dragdown forces of 250 kN.

The pipe piles are displacement piles and substantial heave of immediate surroundings areas can be anticipated. This could also result in the uplift of the surrounding piles. It will, therefore, be necessary to monitor the neighbouring piles for uplift.

The displacement of H piles is much less than the pipe piles. The design bearing of HP 310 or equivalent H pile, driven to the above set criteria, is similar to the pipe pile.

The set of each pile must be confirmed by restriking the pile at least once. The restrike should be carried out 48 hours after the initial set to confirm that the design set is met.

All horizontal loads on the structure should be resisted by battered piles.

If any fill is added to the site, consideration should be given to using a structural slab in conjunction with the deep foundations.

Additional subsoil and ground water investigation will be needed to confirm the above recommendations.

All perimeter foundations or grade beams for the proposed retail building and the Gas Station canopy will have to be placed below the frost penetration depth of 1.5 m below the outside grade. Interior column footings within the heated building can be placed at or below a depth of 1 m from the existing ground level.

The subsoil at the depths at the two boreholes (numbers 7 and 8) carried out in the area of the proposed gas station, consisted of a loose sand deposit. Provided that no additional fill is placed on the existing grade, spread footings founded in the sand deposit can be designed for a bearing pressure of 100 kPa.

The depth of the footings for the canopy of the gas bar should be lowered to an elevation where the slope from the underside of the footings to the excavation of the underground gas tank farm is less than 1H : 1V.

An alternative to spread / strip footings on native soil, the gas bar and kiosk structures can be founded on a concrete pad supported on the grade. An alternative would be to place the gas bar and kiosk structures on a concrete pad supported on the grade. Typical details of the concrete pad construction are shown on Drawing No. 10.

4.3 Floor Slab Construction

Provided that there is no additional fill placed on the site, it is our opinion that a conventional slab-on-grade method, with at least 150 mm of granular A (OPSS form 1010) or 20 mm crusher run limestone below the slab as a moisture barrier, will be feasible at the site.

4.4 Earthquake Consideration

The Ontario Building Code requires that all buildings be designed to resist a minimum earthquake force V , as given in the following expression:

$$V = v.S.I.F.W$$

From a geotechnical point of view, the factors of importance, v and F , at this site can be taken as 0.1 and 1.5, respectively. These values should be reviewed by the Structural Engineer.

4.5 Excavation and Backfilling

All excavations should comply with the Ontario Occupational Health and Safety Act. Any excavation deeper than 1.2 m below the existing ground surface should be sloped back to a safe angle of less than 45° . For excavations deeper than 4 m, there is a possibility of base heave. In order to prevent base heave, the geometry of any excavation deeper than 4 m should be reviewed by this office.

We do not anticipate any serious ground water conditions for excavations of up to about 3 m below the existing ground surface. Below this level, positive dewatering of the sand layer will be necessary.

Bedding for the underground services within the clayey silt or the sand deposits should consist of Granular A or its equivalent. If wet clayey silt and sand layers are encountered at the base of excavation of the service trenches, clear stone bedding is recommended. The clear stone bedding should be separated from the wet subsoil by a geotextile filter fabric, Terrafix 270 R or its approved equivalent. The geotextile filter fabric should completely surround the clear stone to prevent loss of fines from the surrounding soils.

Organic free, on site excavated sand may be used for backfilling, provided it is dried sufficiently to the dry side of its optimum, prior to placement. The clayey silt material will be very sensitive to moisture changes and its use should only be considered if the construction is carried out during the dry season.

4.6 Pavement Design and Construction

Following grading, the subgrade should be proof rolled using heavy vibrating construction equipment. Any soft spots revealed by that process should be sub-excavated and replaced with an approved local or imported fill. The degree of compaction of the backfill should be 95% of the standard Proctor maximum dry density to 300 mm below the subgrade level. The upper 300 mm of the subgrade should be compacted to 98% of standard Proctor maximum dry density.

Provision must be made for the water to drain out of, and not collect in, the granular base courses for the pavement to function properly. Continuous perforated corrugated steel or plastic longitudinal sub-drains should be used to prevent the build-up of water in the roadway's or the parking lot's granular base courses. The pipes should be surrounded by a geotextile filter fabric as per Ontario Provincial Specifications Standard (OPSS 405). The sub-drains should be at least 800 mm below the top of the pavement. Backfill above the drain should comprise of free draining Granular "B" material or an equivalent. The sub-drains should be on a positive grade leading to frost-free sump or catch basins.

Catch basins and manholes should be backfilled with compacted OPSS Granular "B" material. Catch basins should be perforated just above the drain level and the holes should be screened with a filter fabric. This will help drain the pavement structure as well as alleviate the problem of differential movement of catch basins or manholes due to frost action.

5.0 ADDITIONAL INVESTIGATION

Additional subsoil and ground water investigation will be needed to confirm the recommendations provided in the report. The subsurface data at additional boreholes will be obtained using regular standard penetration sampling method and / or by driven dynamic cone penetration tests. Undisturbed samples of the clayey silt and any other cohesive soils encountered at the boreholes will be obtained using thin walled shelby tubes. These samples will be subjected in our laboratory to unidirectional consolidation tests to determine the preconsolidation pressure. Grain size distributions will also be carried out on representative soil samples obtained from the boreholes. The test results from the additional boreholes will be used to determine the foundation alternatives, anticipated settlement of the buildings under surcharge and foundation loads and the approximate time frame for consolidation under surcharge loads.

6.0 GENERAL STATEMENT OF LIMITATION

The comments and recommendations presented in this report are based on the subsoil and ground water conditions encountered at the borehole locations, indicated in the Borehole Location Plan, and are intended for the guidance of the design engineer.

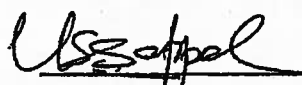
We consider this report to be representative of the subsurface conditions at the subject property. The soil and the ground water conditions between and beyond the borehole locations may differ from those encountered at the time of our investigation. Any contractor bidding on, or undertaking the works, should decide on their own investigations and interpretations of the ground water and the subsoil conditions between the borehole locations. Any use and / or the interpretation of the data presented in this report, and any decisions made on it by the third party are the responsibility of the third party. Toronto Inspection Ltd.'s responsibility is limited to the accurate interpretation of the soil and ground water conditions prevailing in the location investigated by us and accept no responsibility for the loss of time and damages, if any, suffered by the third party as a result of decisions or actions based on this report.

We trust that you will find this report complete within our terms of reference. Should you have any questions regarding the information provided, or when we may be of service to you during the construction phase, please contact this office.

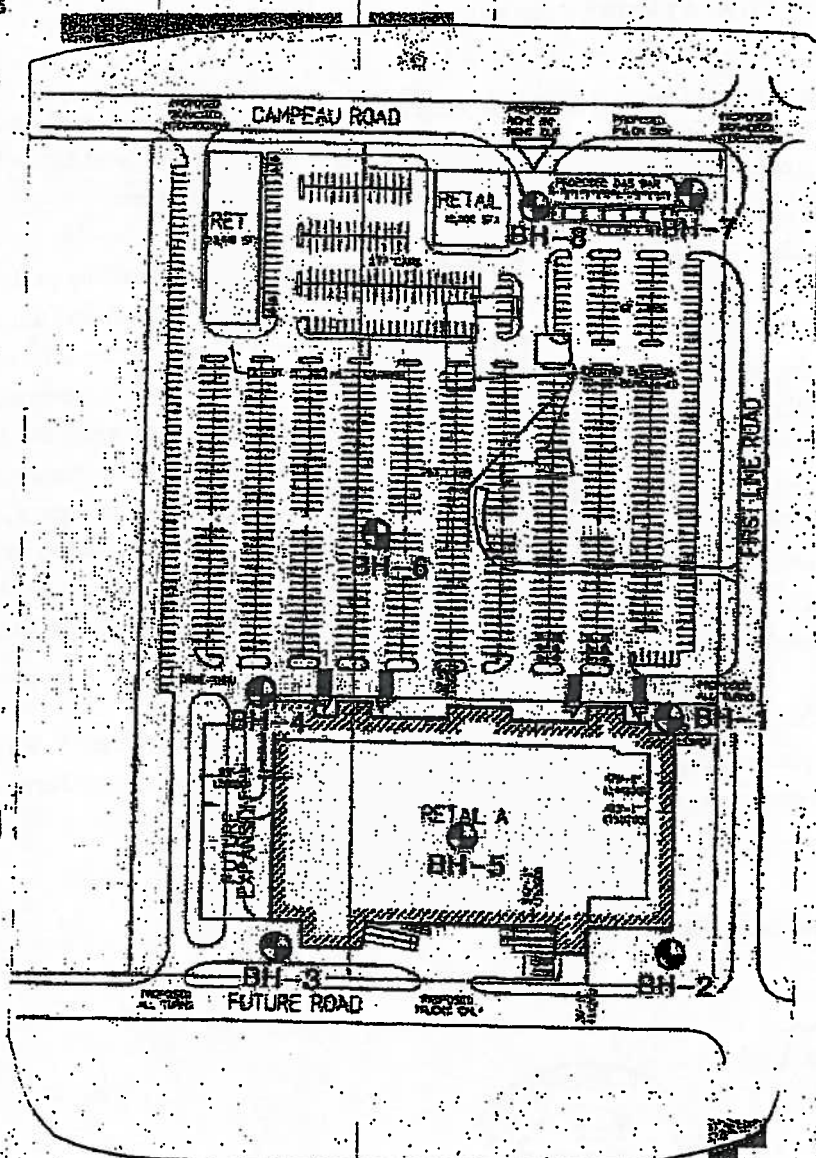
Yours very truly,

Toronto Inspection Ltd.


Bennett C. Sun, P. Eng.
Senior Geotechnical Engineer


Upkar S. Sappal, P. Eng.
Principal Engineer





LEGEND

● Borehole Location

NOT TO SCALE

TORONTO INSPECTION LTD.

REPORT NO.:
03 LPL 782

TITLE: **BOREHOLE LOCATION PLAN**

DRAWING NO.:

LOCATION: Proposed Retail Store & Gas Station - First Line Road & Campeau Road, North Kanata

1

Log of Borehole 1

Dwg No. 2

Project: Proposed Development

Sheet No. 1 of 1

Location: First Line Road & Campeau Road, North Kanata

Date Drilled: 24/02/03

Auger Sample

Headspace Reading (ppm)

SPT (N) Value

Natural Moisture

Drill Type: Truck mounted

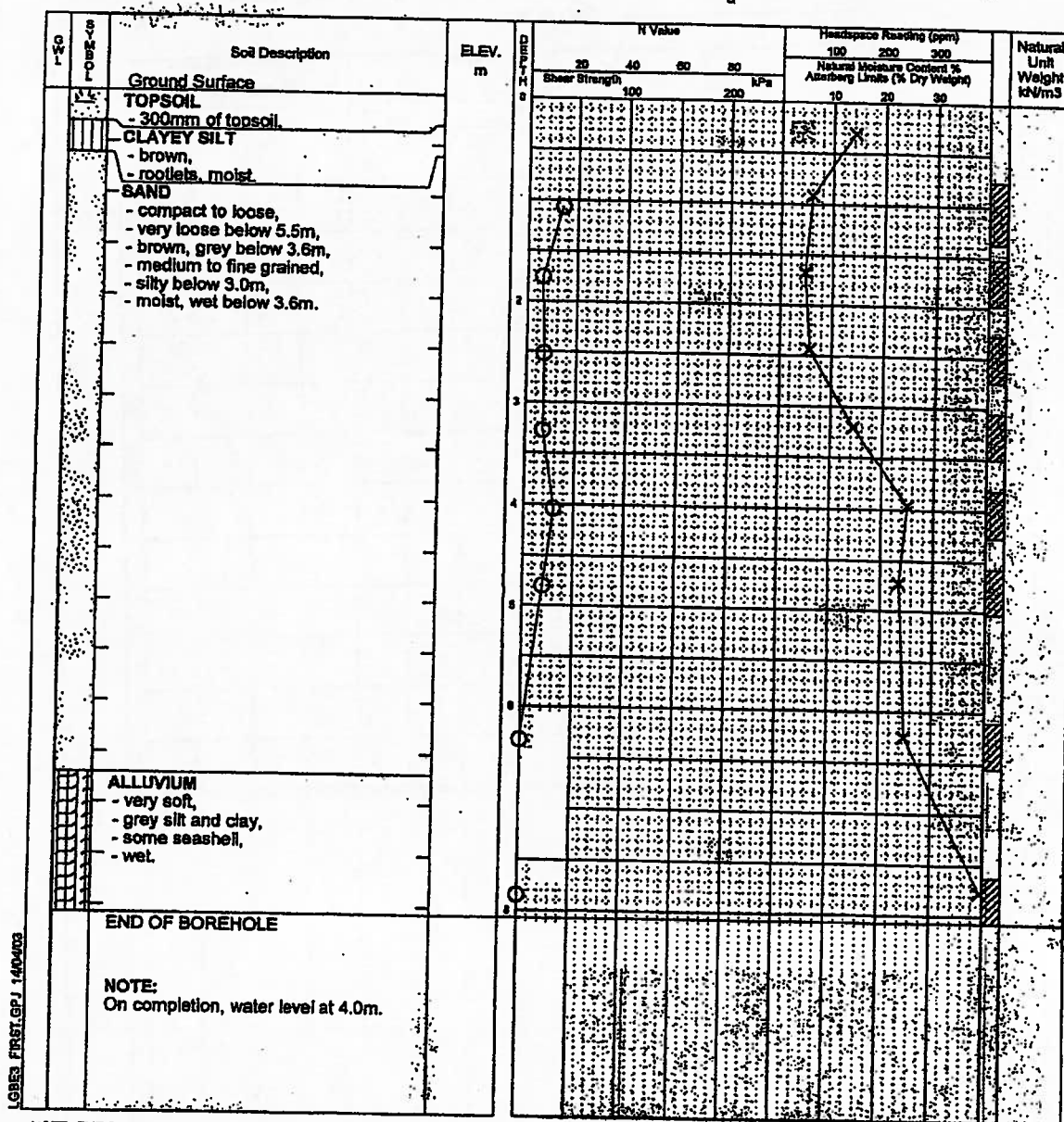
Dynamic Cone Test

Plastic and Liquid Limit

Datum:.. Ground Surface

Field Vane Test

Panostromer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project No. 03 LPL 782

Log of Borehole 2

Dwg No. 3

Proposed Development

Sheet No. 1 of 1

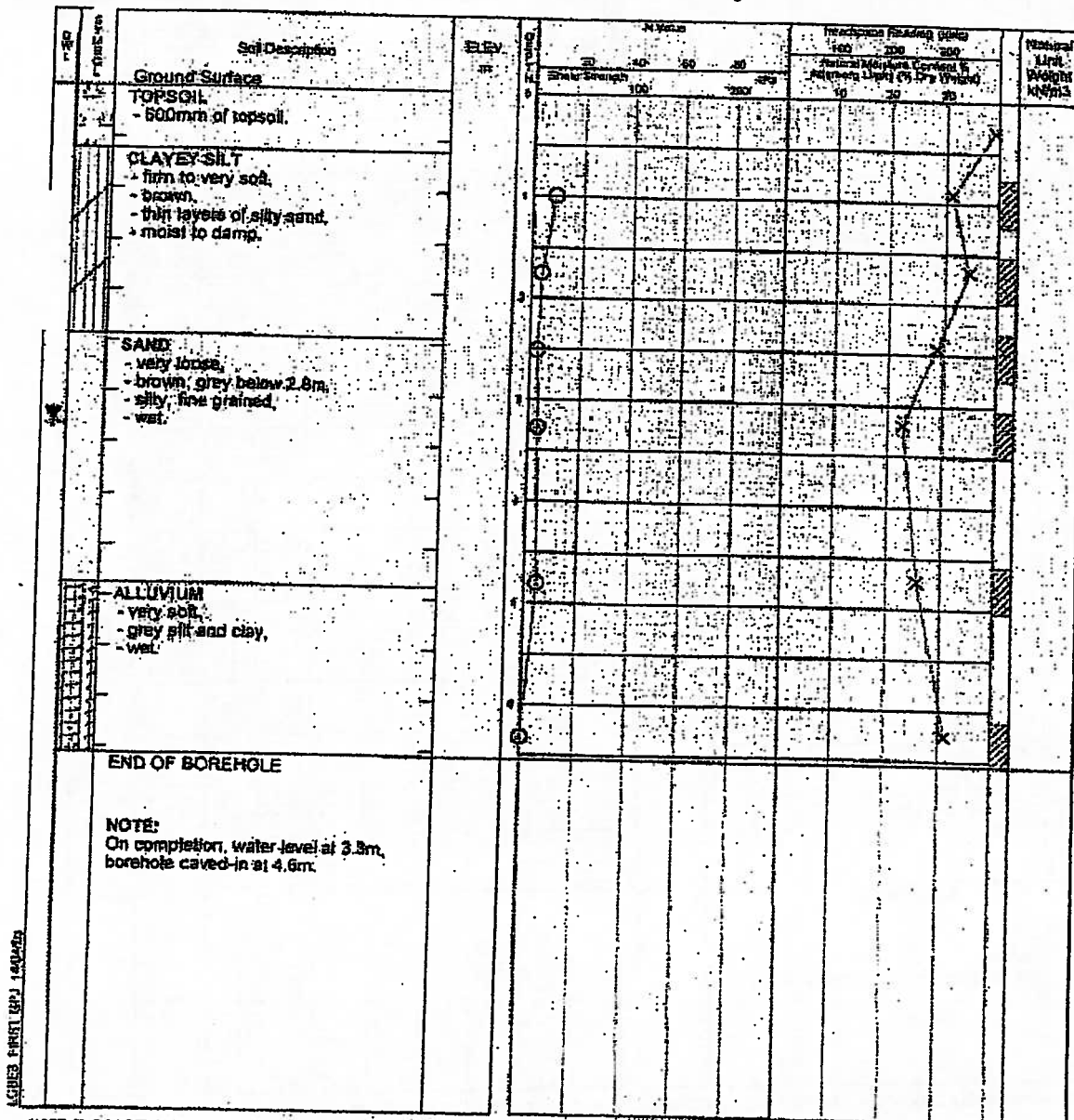
Location: First Line Road & Campeau Road, North Kanata

Date Drilled: 24/02/03

Drill Type: Truck mounted

Auger Sample ☒
 SPT (N) Value ☒
 Dynamic Cone Test ☐
 Shelby Tube ☐
 Field Vane Test ☐

Headspace Reading (ppm) ☐
 Natural Moisture ☒
 Plastic and Liquid Limit ☐
 Unconfined Compression ☐
 % Strain at Failure ☐
 Penetrometer ☐



Log of Borehole 3[illegible]

Dwg No. 4

Location: First Line Road & Campeau Road, North Kanata

Sheet No. 1 of 1

Date Drilled: 24/02/03

Drill Type: Truck mounted

Datum: Ground Surface

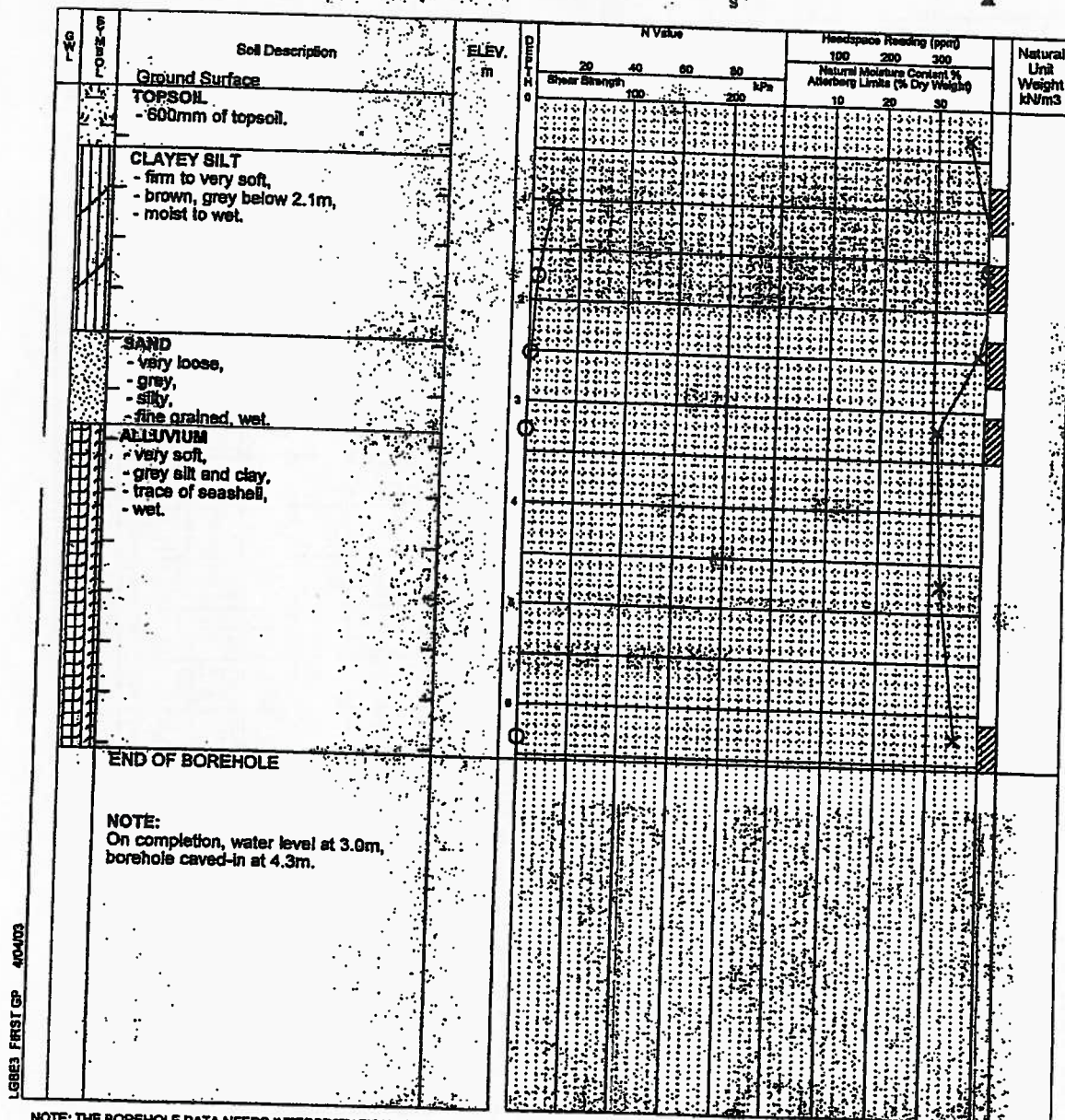
Auger Sample

SPT (N) Value

Dynamic Cone Test

Shelby Tube

Field Vane Test



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

[illegible]

Project No. 03 LPL 782

Log of Borehole 4

Dwg No. 5

Project: Proposed Development

Sheet No. 1 of 1

Location: First Line Road & Campeau Road, North Kanata

Date Drilled: 24/02/03

Auger Sample

Headspace Reading (ppm)

Drill Type: Truck mounted

SPT (N) Value

Natural Moisture

Datum: Ground Surface

Dynamic Cone Test

Plastic and Liquid Limit

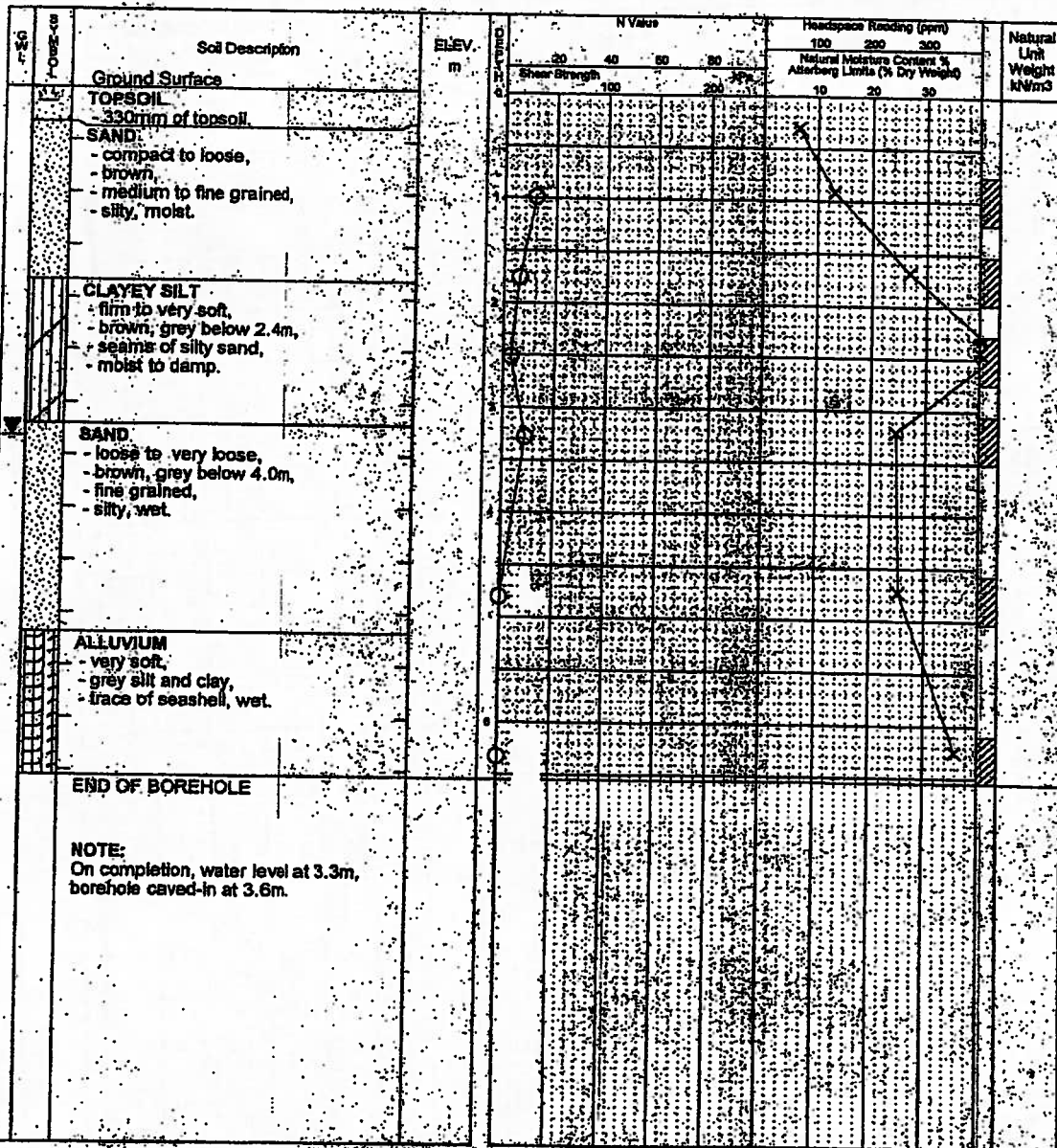
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project No. 03 LPL 782

Log of Borehole 5

Project: Proposed Development

Dwg No. 6

Location: First Line Road & Campeau Road, North Kanata

Sheet No. 1 of 1

Date Drilled: 24/02/03

Drill Type: Truck mounted

Datum: Ground Surface

Auger Sample

SPT (N) Value

Dynamic Cone Test

Shelby Tube

Field Vane Test

Headspace Reading (ppm)

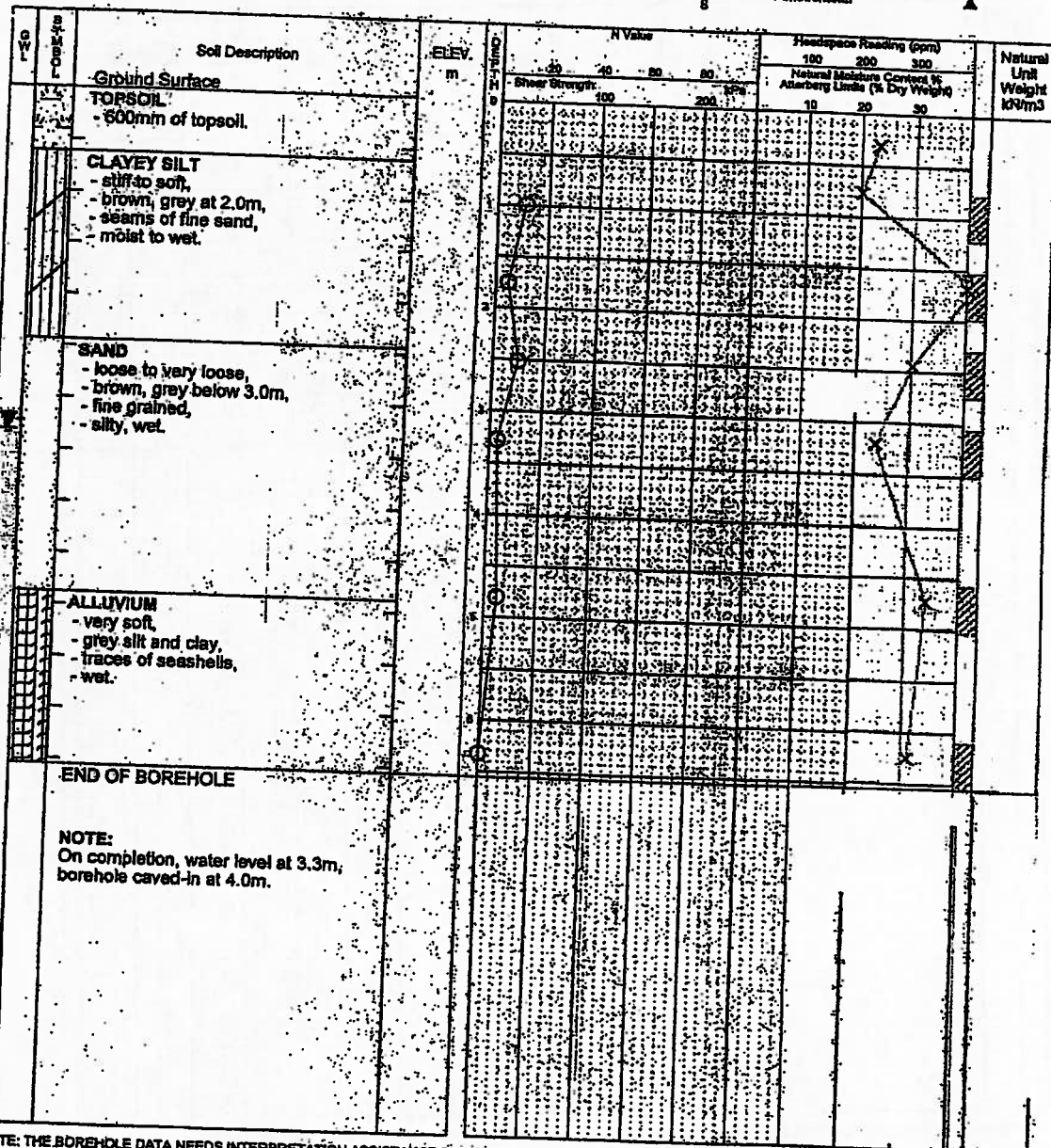
Natural Moisture

Plastic and Liquid Limit

Unconfined Compression

% Strain at Failure

Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project 03 LPL 782

Log of Borehole 6

Dwg No. 7

Project: Proposed Development

No. 1 1

Location: First Line Road & Campeau Road, North Kanata

Date Drilled: 24/02/03

Auger Sample

☒

Natural Moisture

Drill Type: truck mounted

SPT Values

☐ ☒

and Liquid Limit

Drift: Ground Surface

Dynamic Cone Test

Unclassified Compression

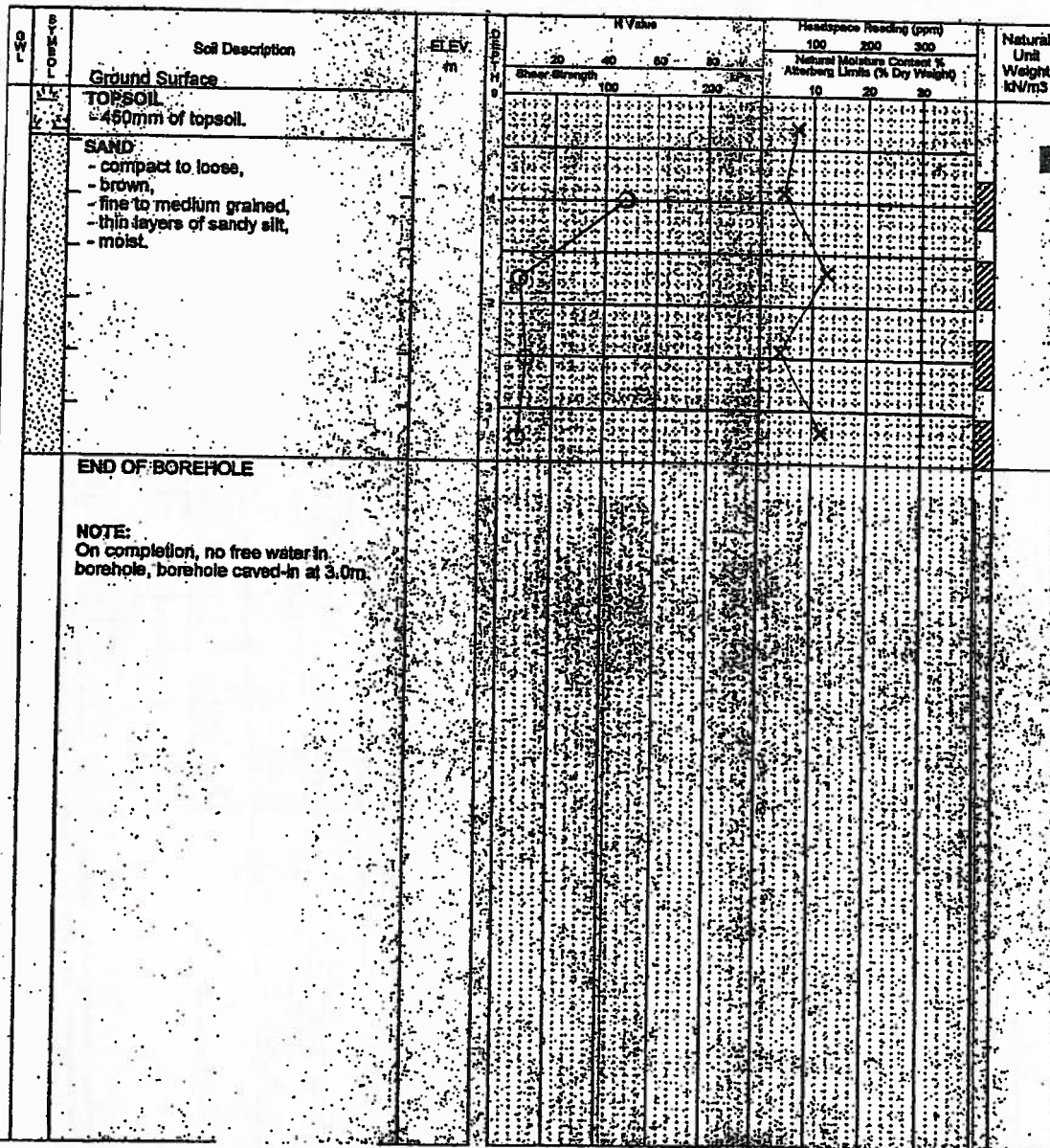
Tube

in Situ at Failure

Field Vane Test

☒

Remoulded



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE

THROUGHOUT

BEFORE USE BY OTHERS

To onto nspection Ltd

Time	Water Level (m)	Depth to Cave (m)

Project No. 03 LPL 782

Log of Borehole 7

Dwg No. 8

Project: Proposed Development

Sheet No. 1 of 1

Location: First Line Road & Campeau Road, North Kanata

Date Drilled: 24/02/03

Auger Sample

Headspace Reading (ppm)

Drill Type: Truck mounted

SPT (N) Value

Natural Moisture

Datum: Ground Surface

Dynamic Cone Test

Plastic and Liquid Limit

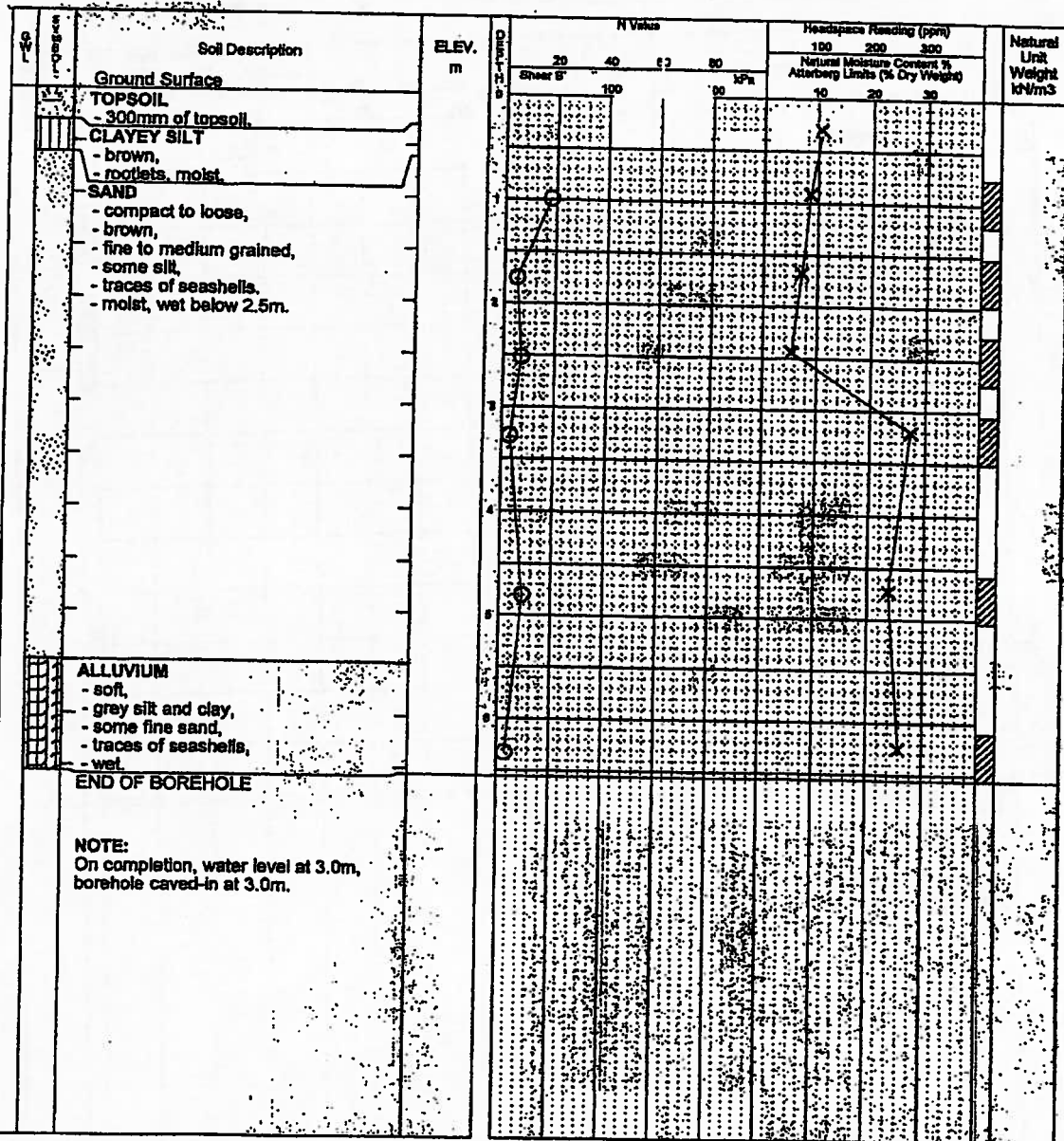
Shelby Tube

Unconfined Compression

Field Vane Test

% Strain at Failure

Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project No. 03 LPL 782

Log of Borehole 8

Project: Proposed Development

Dwg No. 9

Location: First Line Road & Campeau Road, North Kanata

Sheet No. 1 of 1

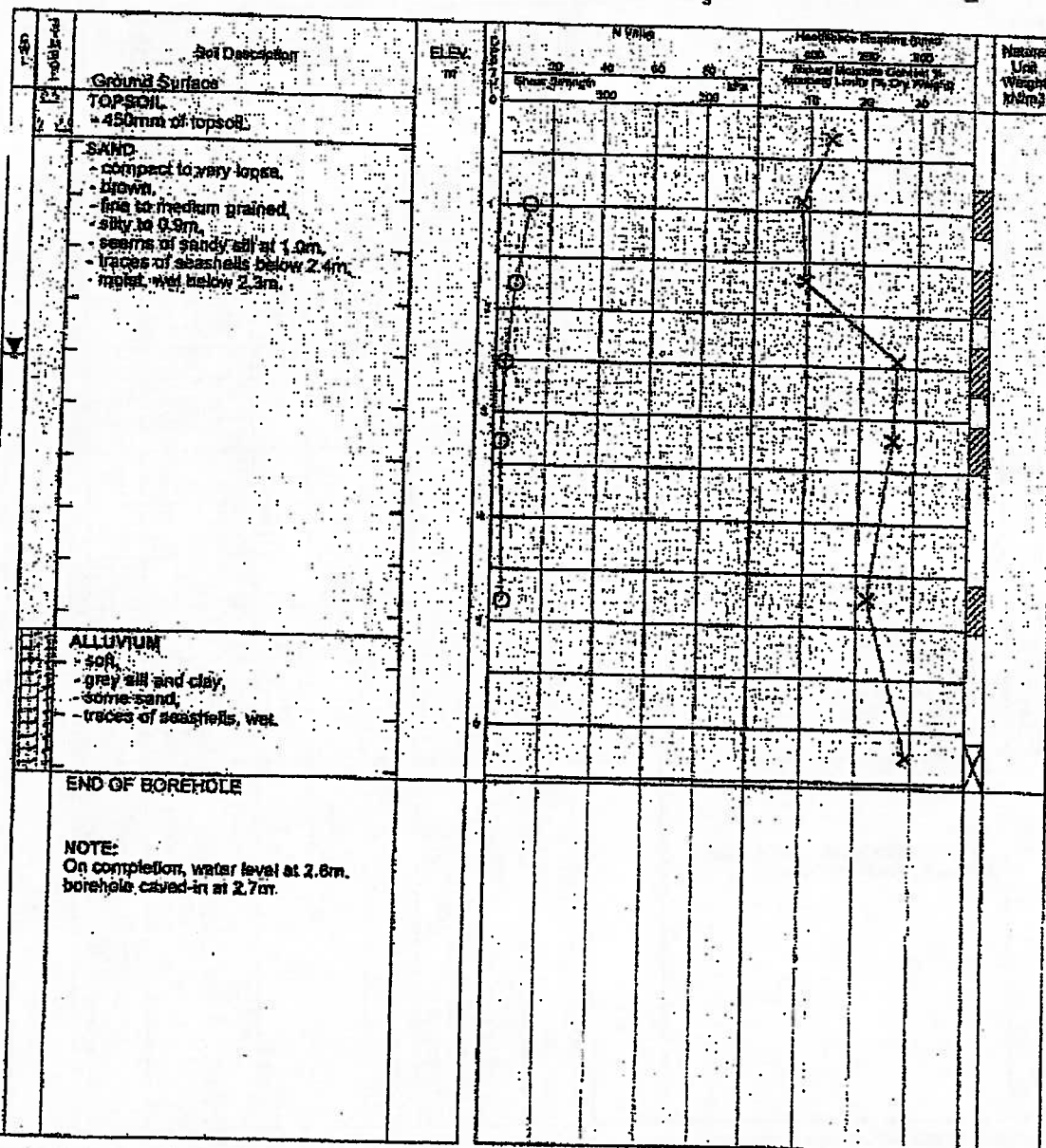
Date Drilled: 24/02/03

Drill Type: Truck mounted

Datum: Ground Surface

Super Sample
SPY (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

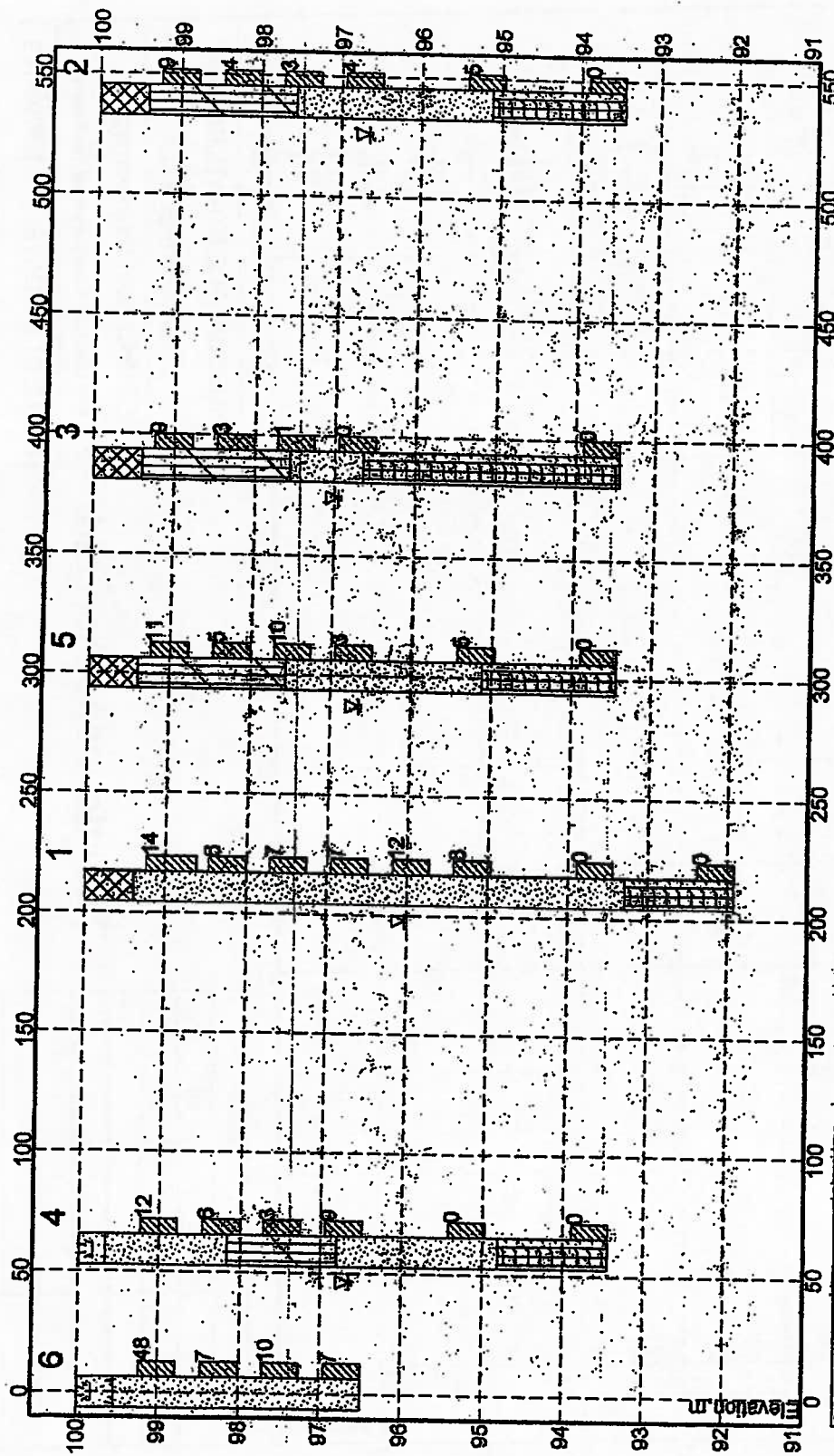
Headspace Reading (ppm)
Natural Moisture
Plastic and Liquid Limit
Unconfined Compression
% Strain at Failure
Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

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Time	Water Level (m)	Depth to Cave (m)



Borehole No	Elev.	Depth
1	100.0	8.1
2	100.0	8.6
3	100.0	8.8
4	100.0	8.8
5	100.0	8.6
6	100.0	3.5

SUBSURFACE STRATIGRAPHY

Section BHs 6, 4, 1, 5, 3 & 2

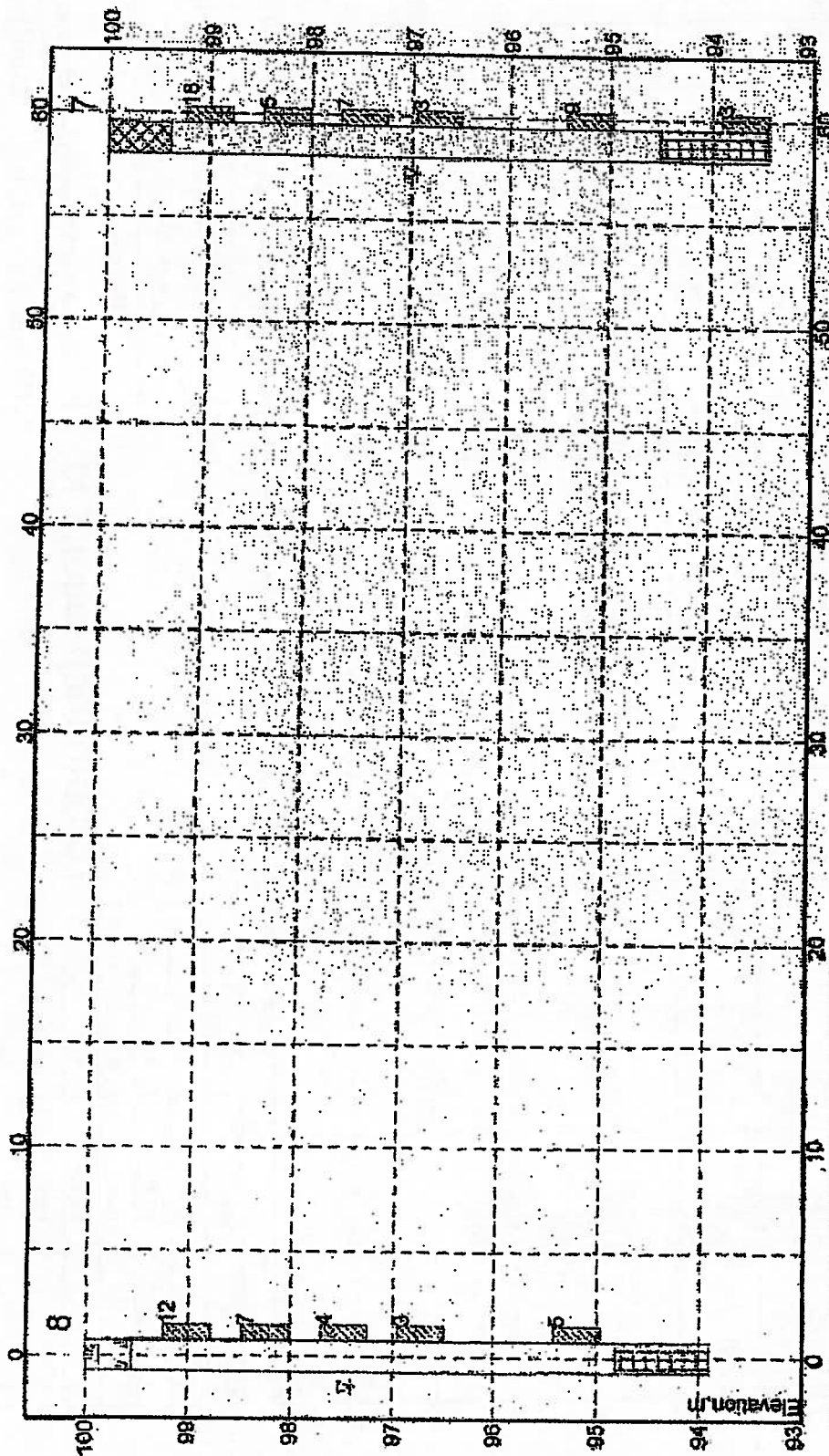
Proposed Development

First Line Road & Campeau Road, North Kanata

PROJECT # DATE DRAWING

03 LPL 782 Mar 03 10

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SUBSURFACE STRATIGRAPHY

Section BHs 8 & 7

Proposed Development

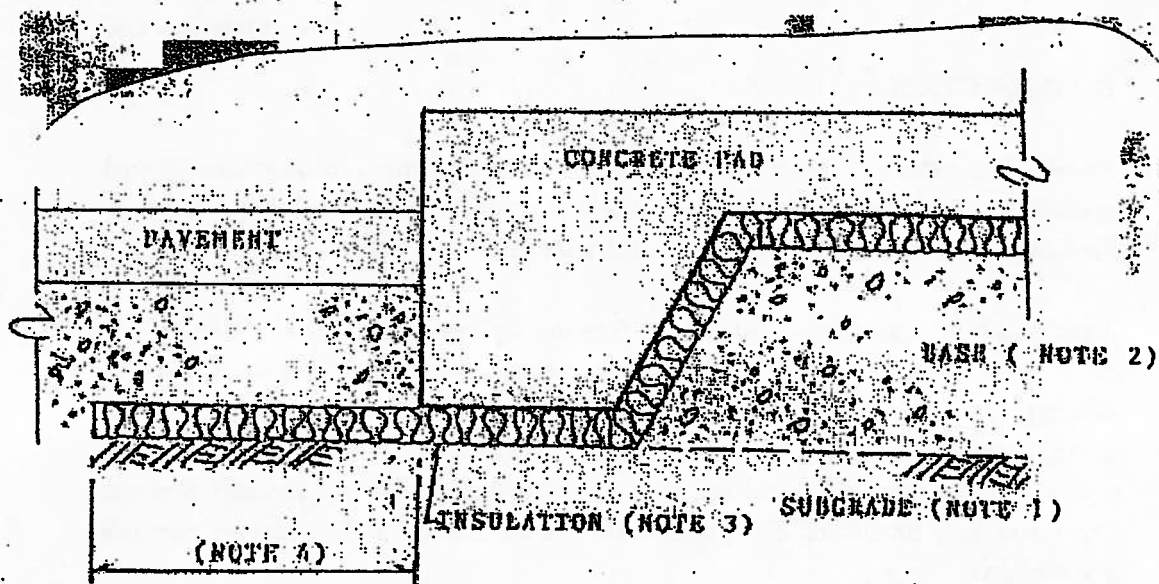
First Line Road & Campeau Road, North Kanata

PROJECT # DATE DRAWING

03 LPL 782 Mar 03 11

Toronto Inspection Ltd.

Borehole No	Elev.	Depth
7	100.0	6.6
8	100.0	6.1



Description:

1. Subgrade material should be free of organics or compressible topsoil, compacted to at least 98% of its Standard Proctor maximum dry density (SPMDD).
2. Base should be Granular A or equivalent, compacted to 100% SPMDD.
3. Rigid insulation of Styrofoam or equivalent, minimum thickness of 50mm.
4. Insulation should extend at least 1.2m beyond the pad in all directions.

NOT TO SCALE

TORONTO INSPECTION LTD.

REPORT NO.:
03 LPL 782

TITLE: **INSULATION OF SUBGRADE**

DRAWING NO.:

LOCATION: Proposed Retail Store & Gas Station - First Line Road & Campeau Road, North Kanata

12



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1.0 INTRODUCTION

Toronto Inspection Ltd. was retained by Loblaw Properties Limited to conduct an additional geotechnical investigation for a proposed retail store at Didsbury Road (formerly named as First Line Road) and Campeau Drive in Kanata, Ontario.

A previous investigation was conducted by *Toronto Inspection Ltd.* (Report No. 03LPL782 dated April 2003), with eight boreholes extending to depths of 3.5m to 8.1m from the existing grade level. Copies of the borehole logs and the borehole location plan are attached in Appendix A of this report. The investigation revealed that the subsoil consisted of compact to very loose sand overlying a soft to very soft clayey silt and silty clay deposits. The lower limit of the soft clay deposit was not established in the previous borehole investigation.

The additional borehole investigation was to determine the depth of the soft clay strata and to collect relatively undisturbed soil samples from the soft clay for consolidation tests. In addition, in-situ shear vane tests were also conducted to estimate the shear strength of the clay deposit.

This report is provided on the basis of the above terms of reference and on an assumption that the design of structure will be in accordance with the applicable building codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, our office should be consulted to review the design and to confirm the recommendations and comments provided in the report.

This report superseded the previous investigation report (Report No. 03LPL782 dated April 2003).

2.0 SITE CONDITION

The site, approximately 16 acres in area, is located at the southwest quadrant of First Line Road and Campeau Road in North Kanata. At the time of investigation, the site was a farm land, with farm houses, barns and scattered trees on site. The existing ground surface of the site was grass covered. Some soil mounds were evident beside the swale along the east boundary. The existing site gradient drops by approximately 2m towards the west to northwest.

3.0 INVESTIGATION PROCEDURE

The field work for the investigation was carried out on September 9, 2005 and consisted of drilling two sampled boreholes (BH-101 and BH-102) at the locations shown on Drawing No. 1. The boreholes were advanced using a truck mounted drill rig, equipped with continuous flight hollow stem augers and sampling rods, supplied by a specialist drilling contractor.

Soil samples were retrieved from the boreholes at regular intervals using a split spoon sampler in conjunction with Standard Penetration Tests using a driving energy of 475 joules (350 ft-lbs), to depths of 6.6m and 8.1m from grade. Below the sampled depths, in-situ shear vane tests were performed to estimate the undrained shear strength of the cohesive subsoil and thin walled shelby tube samples of the soft clay were collected for a consolidation test in the laboratory. The soil samples were identified, logged and packed carefully in the field for later visual identification and testing.

In addition to Standard Penetration Tests, dynamic cone penetration tests were performed at the borehole locations, from the ground surface to depths of 38.1m and 26.6m from grade, to determine the consistency of the subsoils.

Groundwater observations were made in the boreholes during and upon the completion of drilling.

The ground elevations at the borehole locations were determined in the field using the "Top of Fire Hydrant" located at the northeast intersection of Campeau Drive and Didsbury Road, as a temporary bench mark (TBM).

The geodetic elevation of 97.12m for the TBM was obtained from the Site Grading Plan prepared by Stantec Consulting Ltd.

4.0 SUMMARISED SITE AND SUBSURFACE CONDITIONS

Reference is made to the appended Borehole Location Plan (Drawing No. 1), Logs of Boreholes (Drawing Nos. 2 to 5) for details of field work, including soil classification, inferred stratigraphy, and groundwater observations carried out during the drilling of the boreholes. The current boreholes revealed that the subsoils, below the surficial topsoil,



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generally consisted of a sand, overlying a silty clay deposit. Brief descriptions of the subsoil encountered at the current borehole locations, were as follows:

4.1 Topsoil

A layer of topsoil was contacted at the ground surface of the boreholes. The thickness of topsoil, at the borehole locations, was 250mm.

4.2 Sand

Underlying the topsoil at the location of boreholes, a sand deposit was encountered. The deposit was weathered at the upper portion and contained rootlets to depths of 0.3m to 0.6m from grade. The sand was generally fine to medium grained, with thin layers of sandy silt, extending to depths of 4.0m and 6.7m from the existing ground level. Some sea shells were present in the sand deposit, at a depth of 2.4m from grade at BH-101 location, indicating the sand might be a marine deposit.

Based on the Standard Penetration N-values, in the range of 3 to 12 blows per 0.3m penetration, the deposit was compact to loose. The moisture content of the sand deposit varied from 5 to more than 20%, indicating a saturation zone below the depth of 2 to 3m from grade.

The bulk unit weight of the sand deposit was estimated to be 19 kN/m^3 and increased to 21 kN/m^3 in a saturated condition.

4.3 Silty Clay

A silty clay deposit was encountered below the sand at the borehole locations. The deposit was cohesive, of low to medium plasticity and consisted of imbedded sea shells and trace of dark stains/organic matters upto a depth of 7.5m to 9m from grade. The unit weight of the clay deposit was estimated to be 16 kN/m^3 .

Atterberg Limits tests, conducted on a clay sample obtained from the location of BH101, at a depth of 10.9m, were as follows:

Plastic Limit: $w_p = 21\%$

Liquid Limit: $w_L = 33\%$

Plasticity Index = $w_L - w_p = 12\%$



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The results, plotted on the Plasticity Chart (Figure 1), indicated that the deposit can be classified as inorganic clay of medium plasticity.

The in-situ moisture content of the deposit was more than 30%, indicating saturated conditions and above the liquid limit of the soil.

The Standard Penetration N-values obtained in the clay deposit was less than 2 blows per 0.3m penetration. Based on the in-situ shear vane tests, conducted in the clay deposit at depths of 7.9m to 11.0m, the estimated undrained shear strength (c_u) of the clay deposit, varied from 40 kPa to 88 kPa.

The lower limit of the soft clay deposit was not established within the sampled depths. However, dynamic cone penetration tests were performed to estimate the lower extent of the soft deposit. Based on the dynamic cone penetration results, it is our opinion that the soft deposit extended to depths of at least 16m to 18m from grade, where the blow counts increased gradually.

A consolidation test was carried out on a relatively undisturbed sample of the clay, obtained using a thin wall Shelby tube from BH101, at a depth of 11.0m from grade. The results, shown on Figure 2, indicated that the deposit, at the test location, has a pre-consolidated pressure (p_c') of about 130 kPa (2,600 psf), around 30 kPa over the existing effective overburden pressure.

4.4 Groundwater

Groundwater seepage was evident in the boreholes during the drilling operation. Upon the completion of drilling, free water was recorded at depths of 2.1m and 5.4m from grade, at BH-101 and BH-102 locations, respectively.

Based on the field observation and the moisture content profile of the subsoil samples, it is our opinion that groundwater is apparent in the sand deposit at the saturation level of approximately 2.1m to 3.0m from grade level.

The groundwater level may fluctuate according to the seasonal conditions.



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5.0 RECOMMENDATIONS

We understand that the proposed retail will be a single storey framed structure without a basement, with paved parking lot and driveways at street level. The site development will include site grading by cut and fill operation. The finished floor level will be 96.6m, ie. a cut of about 1.6m at the southeast portion and a fill of about 0.8m at the northwest portion of the proposed building. Based on the subsoil conditions, as encountered at the borehole locations, our recommendations are as follows:

5.1 Site Grading

During the site preparation, the contractor must allow for removal of topsoil and any deleterious fill from the areas of development before the cut and fill operation. The soil description and depth of fill, shown on the Borehole Logs, are specific depths at the borehole locations only. In areas of the existing building and underground utilities, additional fill material may be encountered. Since the depth of organic fill can vary considerably at the site, the contractor bidding for the project must determine the depths and the profile of the fill material by test pits to accurately define the volume of organic fill to be removed from the site.

Compressible material, consisting of silty clay with relatively high organic content, will not be suitable for reuse in areas where future settlement cannot be tolerated. This material will have to be disposed off-site or reused in landscaped areas, subject to approval by the landscape architect.

Due to the current difference on the ground elevation, the proposed re-development of the site will require a cut and fill operation. Within the fill areas such as the parking lot, the ground will be subject to surcharge loads resulting in differential settlement. Assuming a unit weight of 20 kN/m³ for the uplift material, the anticipated long term settlement (not including the immediate settlement anticipated in the sand layer) is estimated as follows:

GRADE RAISE	IMPOSED FILL LOAD	ESTIMATED SETTLEMENT
0.5m	10 kPa	5 mm
1.0m	20 kPa	10 mm
1.5m	30 kPa	15 mm

5.2 Foundations

The silty clay subsoil, below the sand crust at the site, is slightly over-consolidated with an estimated overstress of 30 kPa from the existing overburden pressure. The building can be supported on conventional footings only if the proposed grade raise is kept to a minimum.

Based on the shear resistance of the subsoil, the allowable bearing pressures for conventional footings founded on the sand deposit at this site will have to be limited to 72 kPa (1,500 p.s.f.).

The settlement analysis indicate that with a maximum grade raise of 0.8m, the anticipated live and dead load of 7 kPa on the slab-on-grade and footings designed for a bearing pressure of 72 kPa, founded at a depth of 1.8m from grade, will result in an estimated settlement of 40mm, a differential settlement of approximately 35mm across the building.

If the allowable settlement is to be limited to 25mm, a light weight fill (Expanded Polystyrene Geofoam or equivalent) may be used for uplifting part of the building site at the northwest portion of the proposed building, to reduce the surcharge load onto the soft deposit. The Geofoam has a unit weight of about 0.1 kN/m³. For 0.2m grade raise with Geofoam, and 0.6m with conventional fill, the anticipated settlement, under the similar slab-on-grade and foundation loading conditions, will be about 20mm, compared to 40mm with regular fill material. The Geofoam can be applied in any part of the building area where the grade raise is more than 0.6m from the existing grade.

The Geofoam should be placed and installed as per manufacturer's specifications. In addition, the Geofoam should be so installed to resist any uplift pressure from the possible hydrostatic pressure in the ground. It is necessary to place the Geofoam at the lowest level, with the conventional fill on top for uplift resistance. Subdrains should also be installed at a level below the Geofoam to drain any groundwater build up. Alternatively, an anchoring system may be installed to hold the Geofoam in place.

All the perimeter footings, grade beams and exterior footings which will be exposed to seasonal freezing condition should be placed covered with at least 1.8m of soil or



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equivalent thermal insulation for frost protection. Interior column footings within heated buildings can be placed at a depth of 1 m from the existing ground level.

It should be noted that the above recommendations for the foundations have been analysed by *Toronto Inspection Ltd.* from the information obtained from the present and previous boreholes. Field inspection should be provided by *Toronto Inspection Ltd.* to verify that the soils at the site are as interpreted from the borehole information provided. If the soil conditions are found to vary from those shown on the borehole logs, the design and construction recommendations might have to be modified in the field.

5.3 Slab-on-grade Construction

The floor slab of the proposed building, founded on conventional footings, can be designed and constructed as a conventional slab-on-grade method. Prior to site grading, the building subgrade should be proof-rolled using a heavy roller in the presence of the soils engineer. Any new fill below the subgrade level should be free of organics, and should be compacted in shallow lifts not exceeding 300 mm to 95% Standard Proctor maximum dry density.

A bedding consisting of at least 150 mm of Granular A (OPSS Form 1010) or 20mm crusher run limestone, is recommended as a moisture barrier under a light to medium loaded floor slab. The bedding should be compacted to at least 100% Standard Proctor Maximum Dry Density (SPMDD).

5.4 Earthquake Consideration

The Ontario Building Code requires that all buildings be designed to resist a minimum earthquake force V , as given in the following expression:

$$V = v.S.I.F.W$$

From a geotechnical point of view, the factors of importance, v and F , at this site can be taken as 0.10 and 1.3, respectively. These values should be reviewed by the Structural Engineer.



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5.5 Excavation and Backfill

All excavations should comply with the Ontario Occupational Health and Safety Act. Excavation should be sloped back to a safe angle of less than 45°. For excavation into the clay deposit or below the saturated level, the slope of excavation should be flattened to a safe condition.

A continuous ground water table apparently exists at the site within the sand deposit. The sand deposit will have to be dewatered before any excavation can be carried out into and below the water level within this deposit. It is our opinion that filtered sumps might be adequate for shallow excavations to within 2m to 3m from grade. Provision should be made in the construction budget to install a system of vacuum well points to lower the water table for excavations deeper than a depth of 3m below the existing ground level.

Bedding for the underground services, including catch basins and manholes, within the sand deposit, should consist of OPSS Granular A, 20mm crusher run limestone, or equivalent. If the subsoil at bottom of trenches consists of saturated sand or wet clay, the bedding in the service trenches should consist of 20mm clear limestone and a geotextile filter fabric should be used to separate the clear stone bedding from the base and the sides of the excavation. The geotextile filter fabric must surround the clear stone bedding completely.

Permanent lowering of water table in the sand and the lower silty clay deposit could result in long term settlement due to additional load of the soft clay deposit. If the sewer inverts extend below the existing water table or into the lower silty clay deposit, this office should be consulted to provide recommendations to prevent the lowering of the water table in the long run.

The sand material removed from the site may be reused for backfilling. However, the in-situ moisture content of the clay material was much higher than their optimum moisture content and this material is not suitable for backfilling the trenches in their current high moisture content state. Provided that the material can be dried out to or below the optimum moisture content, the use of the on-site excavated clay material should be limited to backfill locations where future settlement will be of little consequence.



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Topsoil and other compressible fill removed from the site may be reused in landscape areas, subject to the approval of the landscape architect.

Backfill around catch basins, manholes, and narrow trenches should consist of imported granular material. The backfill should be compacted using a smaller vibratory equipment. Catch basins should be perforated just above the drain level and the holes should be screened with a filter fabric. This will help in draining the pavement structure as well as alleviate the problem of differential movement of catch basins or manholes due to frost action.

5.6 Pavement Construction

We understand that the proposed parking lot will be raised slightly from the existing grade level. The settlement associated with the grade raise has been discussed in Section 5.1 of this report.

Provided that the uplift fill material over the subgrade consists of free draining soils, the Standard Loblaw's pavement design should be followed for the light and heavy duty parking and access roads.

If a frost susceptible material (fine sand, silt or clay) is used to uplift the subgrade, an additional thickness of subbase of upto 450mm should be utilized for the light duty pavement and 600mm for the heavy duty driveway.

The subgrade preparation of the entire pavement area should include removal of the existing vegetation and compressible fill, followed by proof rolling using a heavy roller. Any soft spots revealed by the proof rolling should be subexcavated and replaced with an approved dry material and compacted to at least 98% of its SPMDD.

Provision should be made for the water to drain out and not collect in the granular base courses for the pavement to function properly. If the subgrade consisted of a frost susceptible material, continuous perforated, OPSS 405, longitudinal drains, minimum diameter of 100mm, should be used as subdrains. The subdrains should be at least 1.1m below the pavement level and installed on a positive gradient to allow for a free flow of water. The backfill above the drains should comprise of free draining Granular B or its equivalent and should be continuous with the granular subbase of the pavement.



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6.0 GENERAL STATEMENT OF LIMITATION

The comments and recommendations presented in this report are based on the subsoil and ground water conditions encountered at the borehole locations, indicated in the borehole location plan, and are intended for the guidance of the design engineer. Although we consider this report to be representative of the subsurface conditions at the subject property, the soil and the ground water conditions between and beyond the borehole locations may differ from those encountered at the time of our investigation and may become apparent during construction. Any contractor bidding on, or undertaking the works, should decide on their own investigation and interpretations of the groundwater and the soil conditions between the borehole locations.

Any use and / or the interpretation of the data presented in this report, and any decisions made on it by the third party are responsibility of the third parties. The responsibility of *Toronto Inspection Ltd.* is limited to the accurate interpretation of the soil and ground water conditions prevailing in the locations investigated and accepts no responsibility for the loss of time and damages, if any, suffered by the third party as a result of decisions or actions based on this report.

Any legal actions arising directly or indirectly from this work and/or *Toronto Inspection Ltd.*'s performance of the services shall be filed no longer than two years from the date of *Toronto Inspection Ltd.*'s substantial completion of the services. *Toronto Inspection Ltd.* shall not be responsible to the client for lost revenues, lost of profits, cost of content, claims of customers, or other special indirect, consequential or punitive damages.

To the fullest extent permitted by law, the client's maximum aggregate recovery against *Toronto Inspection Ltd.*, its directors, employees, sub-contractors and representatives, for any and all claims by clients for all causes including, but not limited to, claims of breach of contract, breach of warranty and /or negligence, shall be the amount of the fee paid to *Toronto Inspection Ltd.* for its professional services rendered with respect to the particular site which is the subject of the claim by the client.

Yours very truly,
TORONTO INSPECTION LTD.

Bennett C. SUB, P. Eng.
Senior Geotechnical Engineer

Upkar S. Sappal, P. Eng.
Principal Engineer



Toronto Inspection Ltd.

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DRAWINGS

Borehole Location Plan
Logs of Boreholes

Drawing No. 1
Drawing Nos. 2 to 5

FIGURES

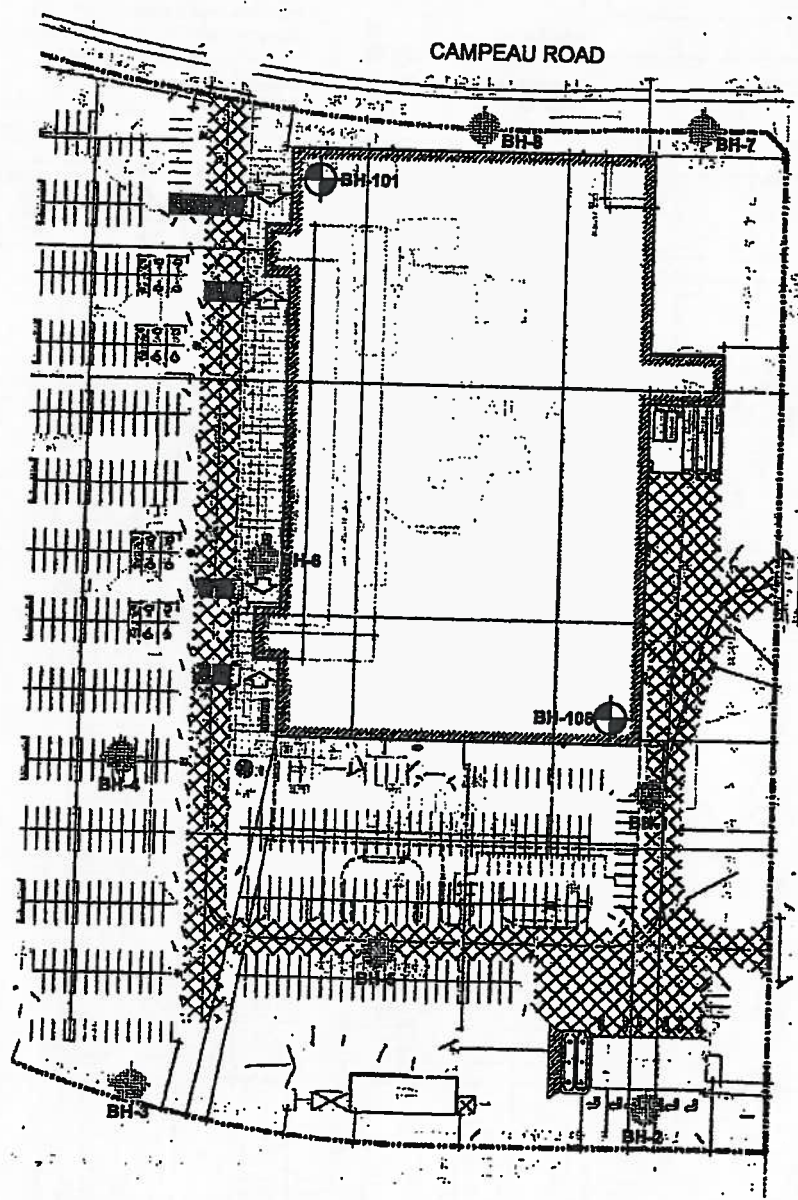
Atterberg Limit
Consolidation Test

Figure No. 1
Figure No. 2

APPENDIX

Previous Logs and Location Plan of Boreholes

Appendix A



EXISTING CANADIAN TIRE

LEGEND:



Borehole Location



Previous Borehole Location

NOT TO SCALE

TIL Toronto Inspection GEO-ENVIRONMENTAL CONSULTANTS 110 Konrad Crescent, Unit 16, Markham, Ontario L3R 9X2 Tel: 905-940 8509 Fax: 905-940 8192	TITLE: Borehole Location Plan
	LOCATION: Campeau Drive & Didsbury Road, Kanata, Ontario
	PROJECT NO. 1475-05-G-LPL-A DRAWING NO. 1

Project No. 1475-05-G-LPL-A

Log of Borehole 101

Dwg No. 2

Project: Proposed Development

Sheet No. 1 of 1

Location: First Line & Campeau Road, North Kanata, Toronto, Ontario

Date Drilled: 09/09/05

Auger Sample

SPT (N) Value

Dynamic Cone Test

Shear Tube

Field Vane Test

Headspace Reading (ppm)

Natural Moisture

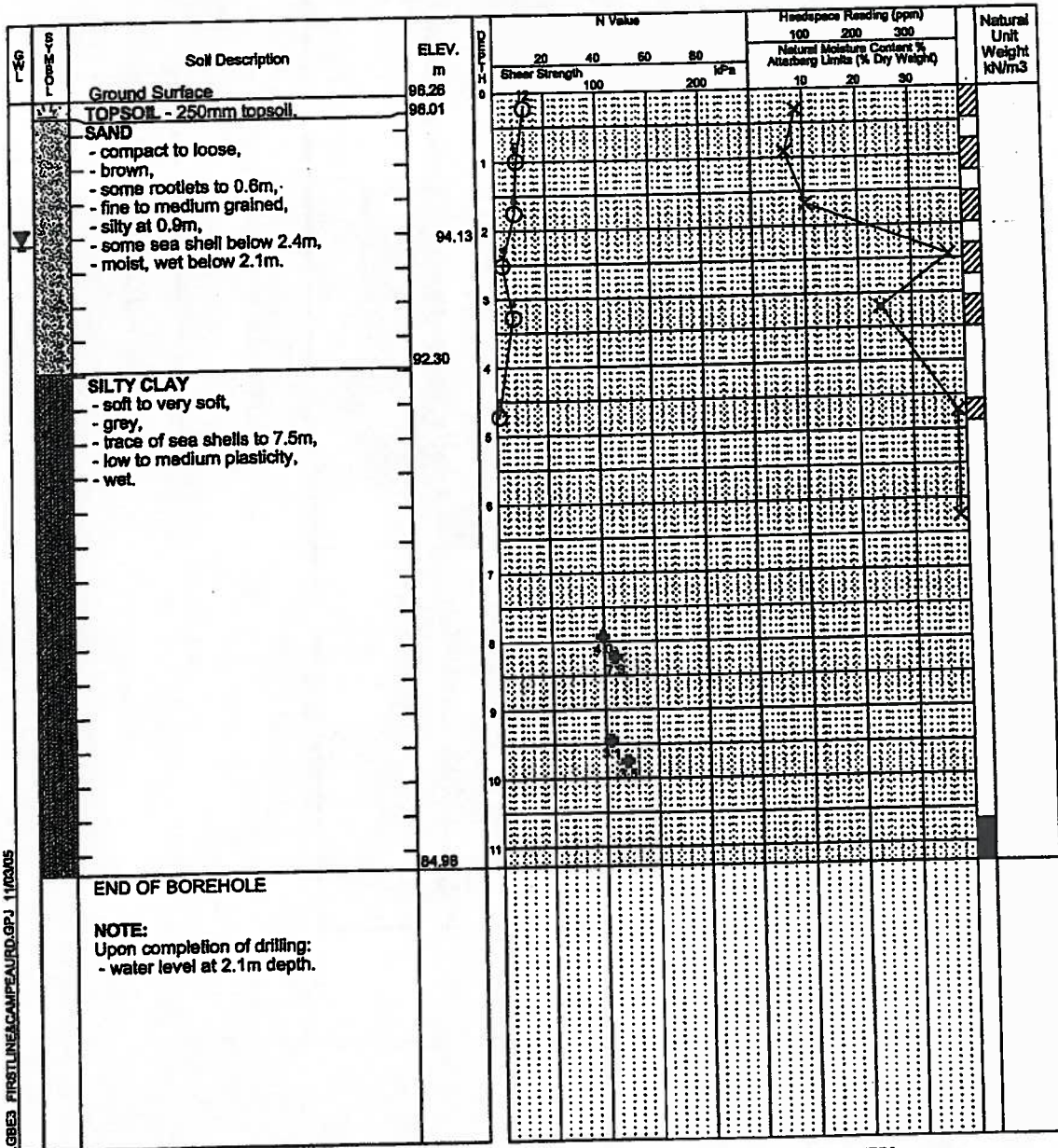
Plastic and Liquid Limit

Unconfined Compression

% Strain at Failure

Penetrometer

Datum: Geodetic



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

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Time	Water Level (m)	Depth to Cave (m)

Project No. 1475-05-G-LPL-A

Log of Borehole 101A

Project: Proposed Development

Dwg No. 3

Location: First Line & Campeau Road, North Kanata, Toronto, Ontario

Sheet No. 1 of 1

Date Drilled: 09/09/05

Drill Type: Truck Mounted Drill Rig

Datum: Geodetic

Auger Sample

SPT (N) Value

Dynamic Cone Test

Shelby Tube

Field Vane Test

Headspace Reading (ppm)

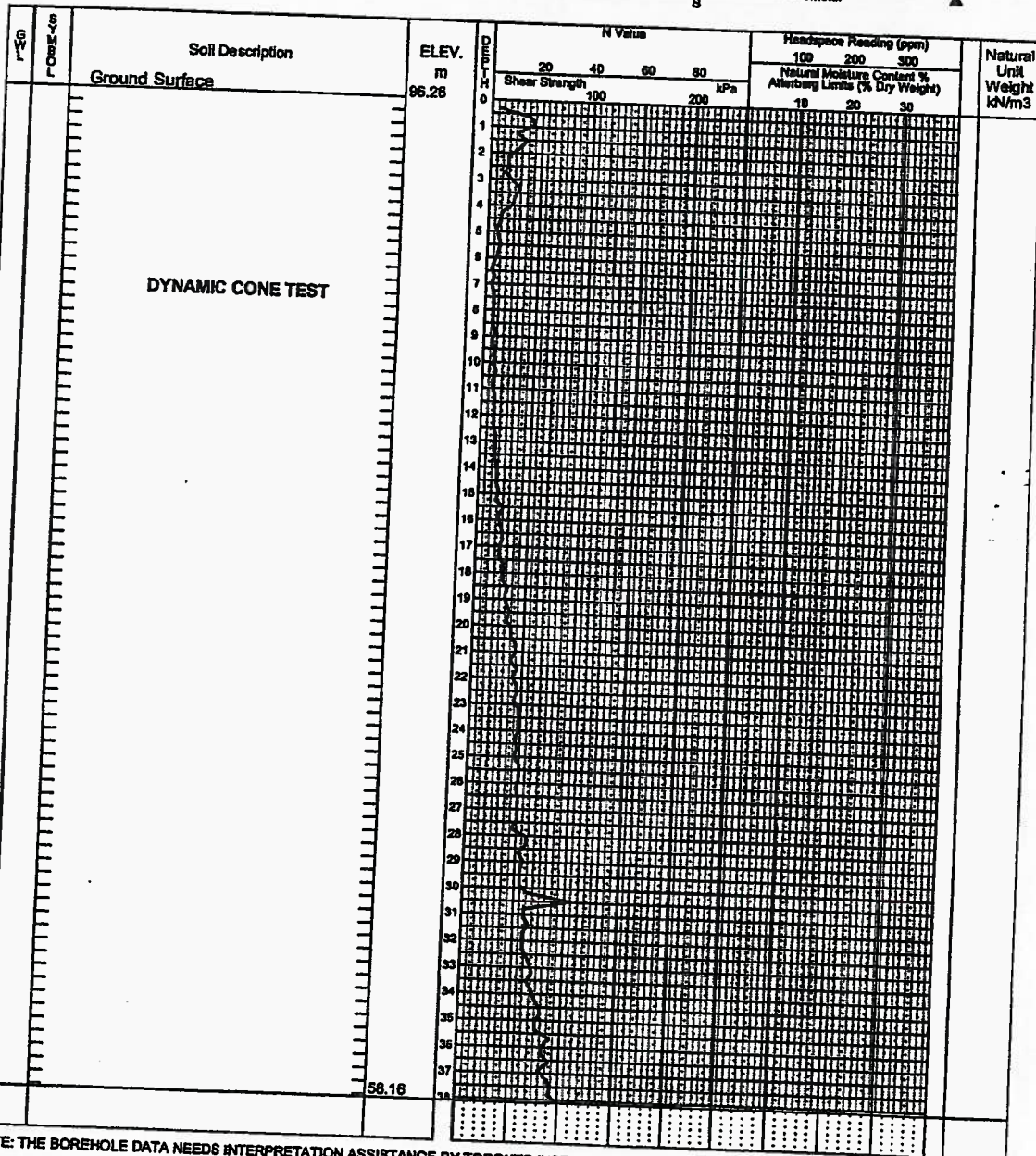
Natural Moisture

Plastic and Liquid Limit

Unconfined Compression

% Strain at Failure

Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Project No. 1475-05-G-LPL-A

Log of Borehole 102

Dwg No. 4

Project: Proposed Development

Sheet No. 1 of 1

Location: First Line & Campeau Road, North Kanata, Toronto, Ontario

Date Drilled: 09/09/05

Auger Sample



SPT (N) Value

Dynamic Cone Test

Shelby Tube

Field Vane Test

Headspace Reading (ppm)

Natural Moisture

Plastic and Liquid Limit

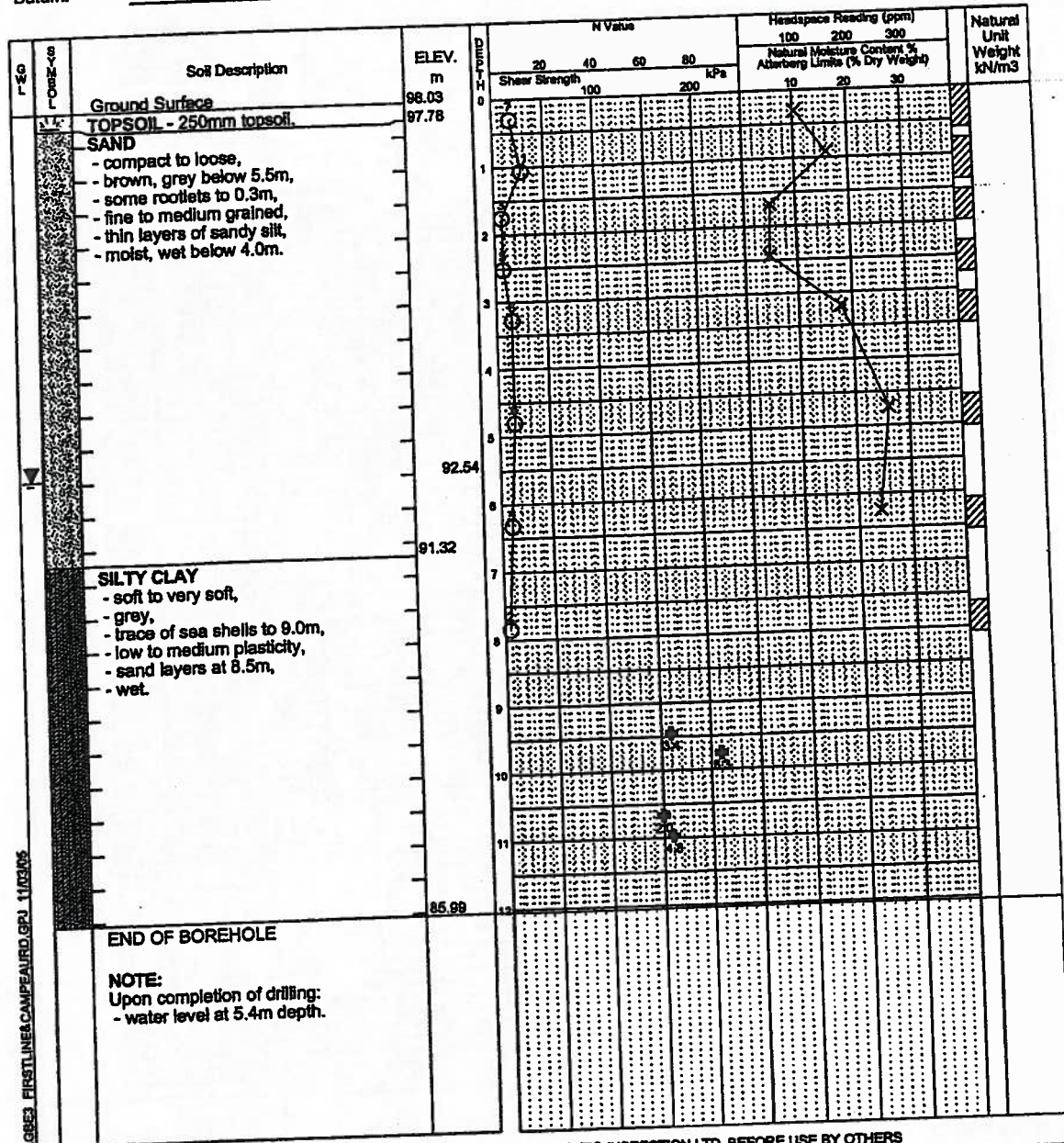
Unconfined Compression

% Strain at Failure

Penetrometer

Drill Type: Truck Mounted Drill Rig

Datum: Geodetic



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

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Time	Water Level (m)	Depth to Cave (m)

Project No. 1475-05-G-LPL-A

Log of Borehole 102A

Project: Proposed Development

Dwg No. 5

Location: First Line & Campeau Road, North Kanata, Toronto, Ontario

Sheet No. 1 of 1

Date Drilled: 09/09/05

Auger Sample ☒

Headspace Reading (ppm) ☒

Drill Type: Truck Mounted Drill Rig

SPT (N) Value ☒

Natural Moisture ☒

Datum: Geodetic

Dynamic Cone Test ☒

Plastic and Liquid Limit ☒

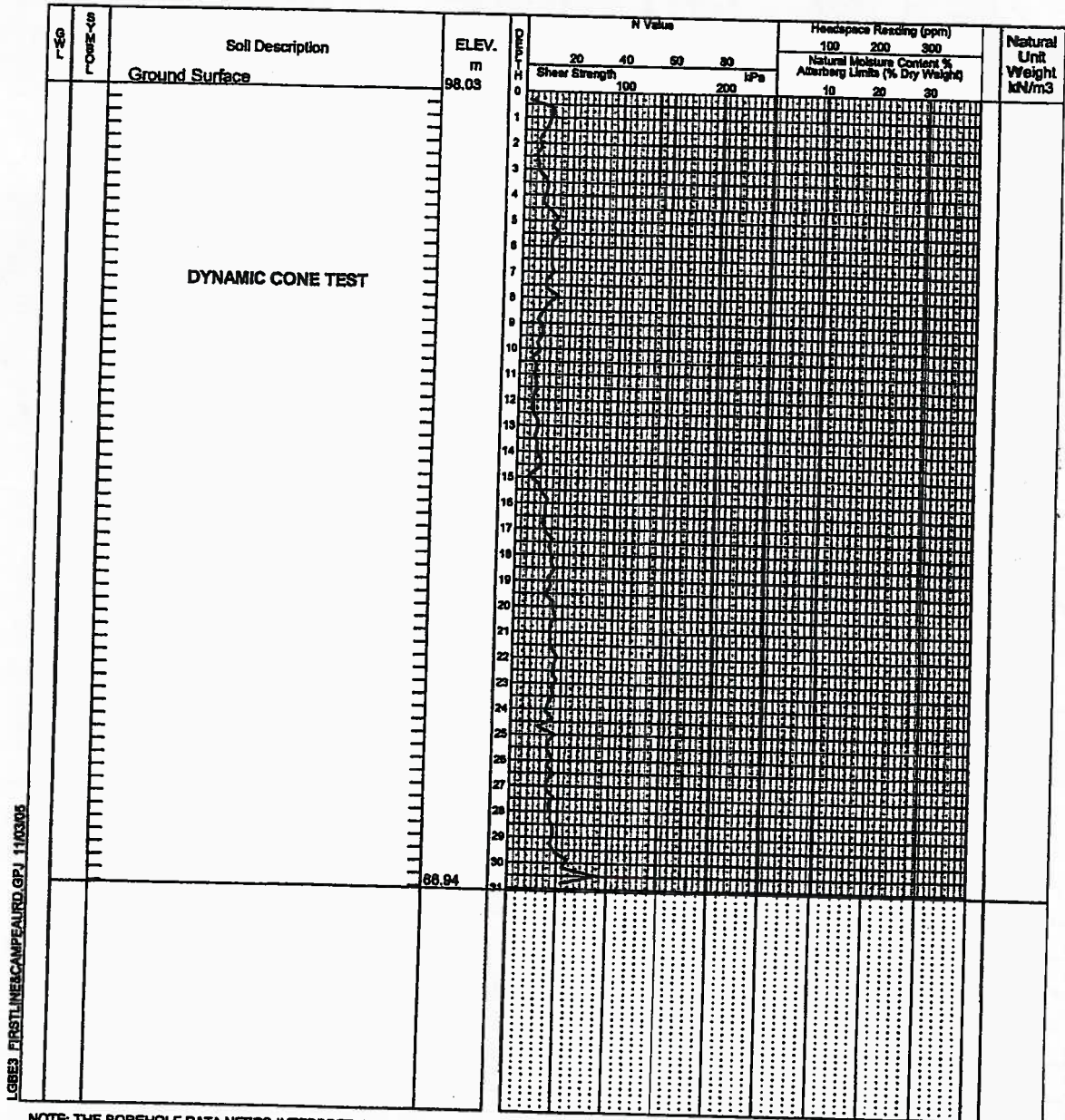
Shelby Tube ☒

Unconfined Compression ☒

Field Vane Test ☒

% Strain at Failure ☒

Penetrometer ☒



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Time	Water Level (m)	Depth to Cave (m)

patersongroup

Consulting Engineers

December 15, 2006
File: PG0940-LET.01

Taggart Group of Companies
c/o Stantec Consulting
1505 Laperriere Avenue
Ottawa, Ontario
K1Z 7T1

28 Concourse Gate, Unit 1
Ottawa, Ontario
Canada K2E 7T7
Tel: (613) 226-7381
Fax: (613) 226-6344

Geotechnical Engineering
Environmental Engineering
Hydrogeology
Geological Engineering
Materials Testing
Building Science

www.patersongroup.ca

Attention: **Mr. Matt Wingate**

Subject: **Geotechnical Review**
Proposed Stormwater Management Pond
Commercial Development - Kanata West
Didsbury Road at Campeau Drive - Ottawa

Dear Sir,

Further to your request and authorization, Paterson Group (Paterson) conducted a geotechnical review of available geotechnical information for the proposed stormwater management pond (SWMP) contemplated for the aforementioned site.

The field program for this site was carried out on September 4th and 5th, 2002. At that time, sixteen (16) test pits were advanced to depths ranging from 0.5 to 4 m below existing grade. The test pit locations were distributed across the site in a manner to provide general coverage of the property. Test pits were also placed within and in close proximity to the proposed SWMP. The locations of the test pits are presented on Drawing PE0147-1 - Site Plan.

1.0 Surface Conditions

The site is bordered to the west by the Carp River and to the east by the Didsbury. The north boundary coincides approximately with the extension of Campeau Drive while the south property will be intersected by the future transit way extension. At the time of this report, the north property was also vacant.

The subject property slopes gently from east to west towards the Carp River, and is mostly grass covered. A maximum difference in grade of 5.5 m was measured between the ground surface elevation at the test pit locations. There are scattered trees along the south property line. Several barns and a two storey dwelling with a basement level exist on east portion of the site (closer to Didsbury Road). A marsh area exists along the west property limit, close to the Carp River.

2.0 Subsurface Profile

The soil profile at this site varies across the site and generally consists of topsoil overlying discontinuous layers of silty clay, clayey silt, silty sand, and/or sandy silt. Fill was observed below the topsoil at TPs 15 and 16. Reference should be made to the Soil Profile & Test Data sheets enclosed for the details of the soil profiles encountered at each test hole location.

Topsoil

Topsoil ranging in thickness from 25 to 380 mm, was observed at ground surface at all test pit locations. No peat layer was observed in the test pits along the marsh area at the west limit of the property.

Fill

Brown sand to silty sand with trace of gravel and brick pieces was encountered below the topsoil layer at TPs 13, 14, 15, and 16. The fill extends to a depths varying between 0.5 and 1.8 m. The fill layer was not fully penetrated at TPs 15 and 16.

Silty Sand, Sand

A brown silty sand layer was encountered below the topsoil at TPs 1, 3, 6, 7, 9, 10, 11, and 12. This layer extends to depths ranging from 0.4 to 1.7 m below existing grade. Silty sand and/or sand layers are also encountered further below grade interbedded with sandy silt, silty clay and/or clayey silt.

Sandy Silt

Sandy silt extending to a depth of 0.9 and 0.6 m, was encountered below the topsoil at TPs 4 and 5, respectively. Sandy silt was also encountered at greater depths at TPs 1, 3, 6, 9, 10, and 11.

Clayey Silt, Silty Clay

Clayey silt and/or silty clay was encountered below the topsoil layer at TP 2 and 8. Clayey silt and/or silty clay was also encountered below the sand, silty sand and sandy silt at all test pit locations except TP 13 to 16.

Generally, the upper portion of this layer has been weathered to a stiff brown crust. The thickness of this weathered crust range from 0.2 m to 1.9 m. No brown crust was encountered at TPs 4 and 5. In-situ shear vane field testing carried out in this portion of the weathered crust yielded results ranging from about 85 to 110 kPa. These values are indicative of a stiff to very stiff consistency.

A grey clayey silt to silty clay layer was encountered at TPs 1, 2, 3, 4, 5, 8, 9 and 10 at depths varying between 2.2 and 3.5 m below grade. Imbedded white sea shells and trace organic matter was noticed in this layer at all test pit locations except for TPs 1, 9 and 10. In-situ shear vane field testing carried out in this portion of the clayey silt to silty clay yielded results ranging from about 15 to 40 kPa. These values are indicative of a soft to firm consistency.

3.0 Groundwater

The level of groundwater inflow into the excavation was measured at each test pit location on September 4 and 5, 2002. The measured groundwater levels are shown on the Soil Profile & Test Data sheets.

The groundwater levels vary between 1.2 and 3.6 m below existing grade. Test Pits 13, 14, 15 and 16 were dry upon completion. The groundwater levels are not considered to be representative of stabilized groundwater levels as they were taken in test pits left open for only a short period of time.

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

4.0 Geotechnical Assessment

Based on the grading plan issued by Stantec Consulting for the proposed development, it is our understanding that the proposed SWMP will consist of the following:

- ☐ Pond bottom elevation 90.25 m
- ☐ 10 year event water elevation 93.35 m
- ☐ Normal water elevation 92.25 m
- ☐ Top elevation of armour stone retaining wall (at pond) 93.80 m
- ☐ Top elevation of armour stone retaining wall (further from pond) 95.00 m

The subject site is considered to be suitable for the proposed SWMP. It is our understanding that the proposed SWMP will be located in an area of silty clay or sandy silt soils extending beyond the full depth of the pond. The surface elevation of the proposed SWMP will range from 94 to 95 m and will be excavated to an elevation of approximately 90.25 m. The permanent water level within the proposed SWMP will be at an elevation of 91.25 m.

From a geotechnical perspective, the long term groundwater level was evaluated at an elevation of 90.8 m below the existing grade. Therefore, it is expected that the construction of the proposed SWMP will not lower the long term groundwater table at the pond location.

The excavation side slopes of the proposed SWMP will intersect the silty sand and sand layers that are relatively pervious. It is expected that these sandy layers will be located above the permanent water level of 91.25 m. Consideration can be given to placing a clay liner in these areas or even rip-rap to provide protection to the pond side slopes during high water situations.

The proposed armour stone retaining walls will be constructed along the east portion of the pond and will be founded most likely in the silty sand layer. An allowable bearing capacity available at the proposed founding elevation of the retaining wall will range from 75 to 90 kPa. A detailed geotechnical review of the armour stone design will be issued at a later date.

We trust that this information is satisfactory.

Paterson Group Inc.



Carlos P. Da Silva, P. Eng.



Attachments:

- ☐ Soil Profile and Test Data sheets - Test Pits 1 to 16
- ☐ Soil Profile and Test Data sheets - Boreholes 1 to 3
- ☐ Drawing PE0147-1- Site Plan
- ☐ Drawing PG0940-1- Test Hole Location Plan

Report Distribution:

- ☐ Stantec (3 copies for distribution)
- ☐ Paterson Group (1 copy)



JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

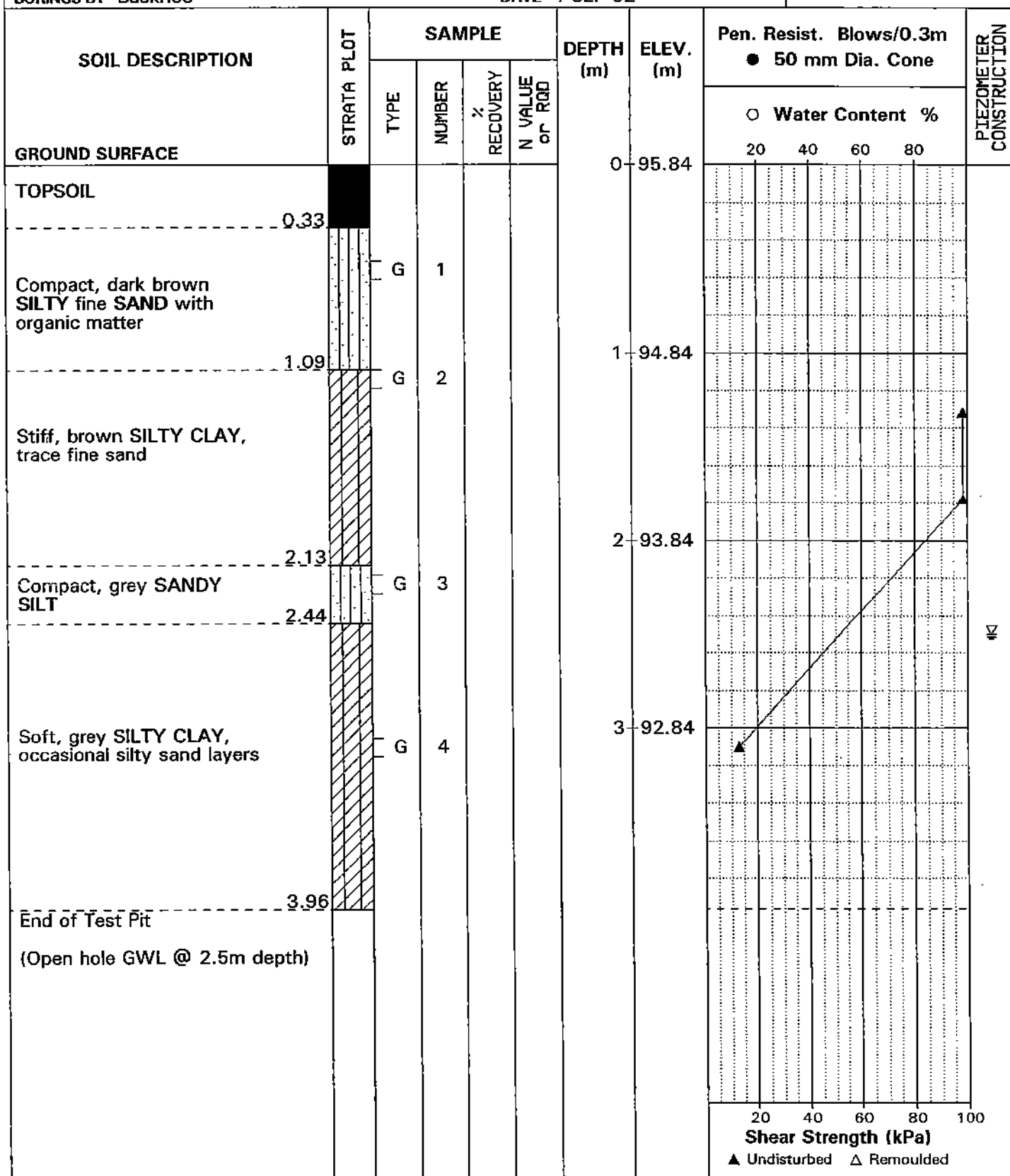
REMARKS

HOLE NO.

TP 1

BORINGS BY Backhoe

DATE 4 SEP 02



SOIL PROFILE & TEST DATA

Geotechnical Investigation

First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

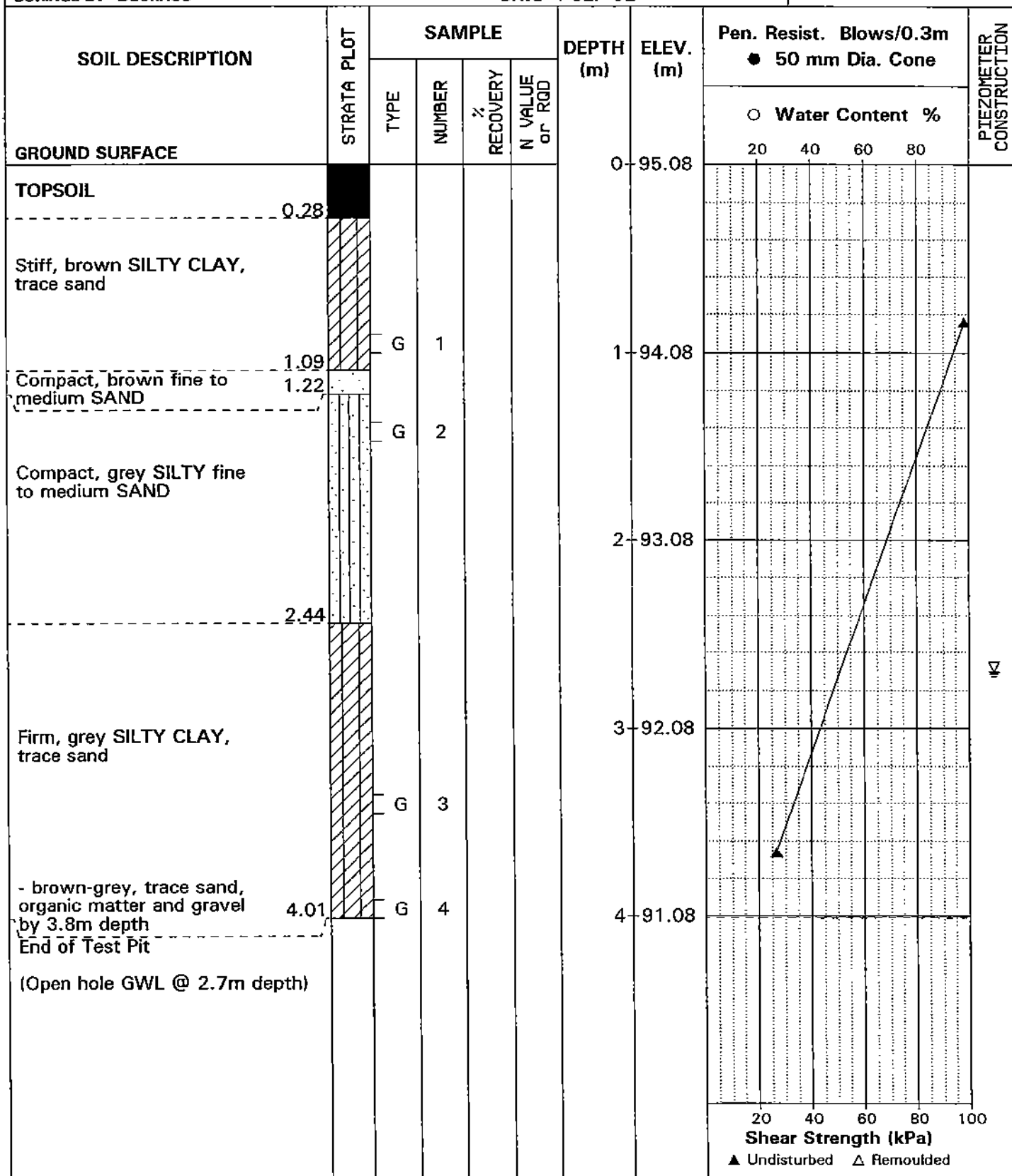
REMARKS

HOLE NO.

TP 2

BORINGS BY Backhoe

DATE 4 SEP 02





First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATE 4 SEP 02

Appendix p206

**JOHN D. PATERSON & ASSOCIATES LTD.**

Consulting Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

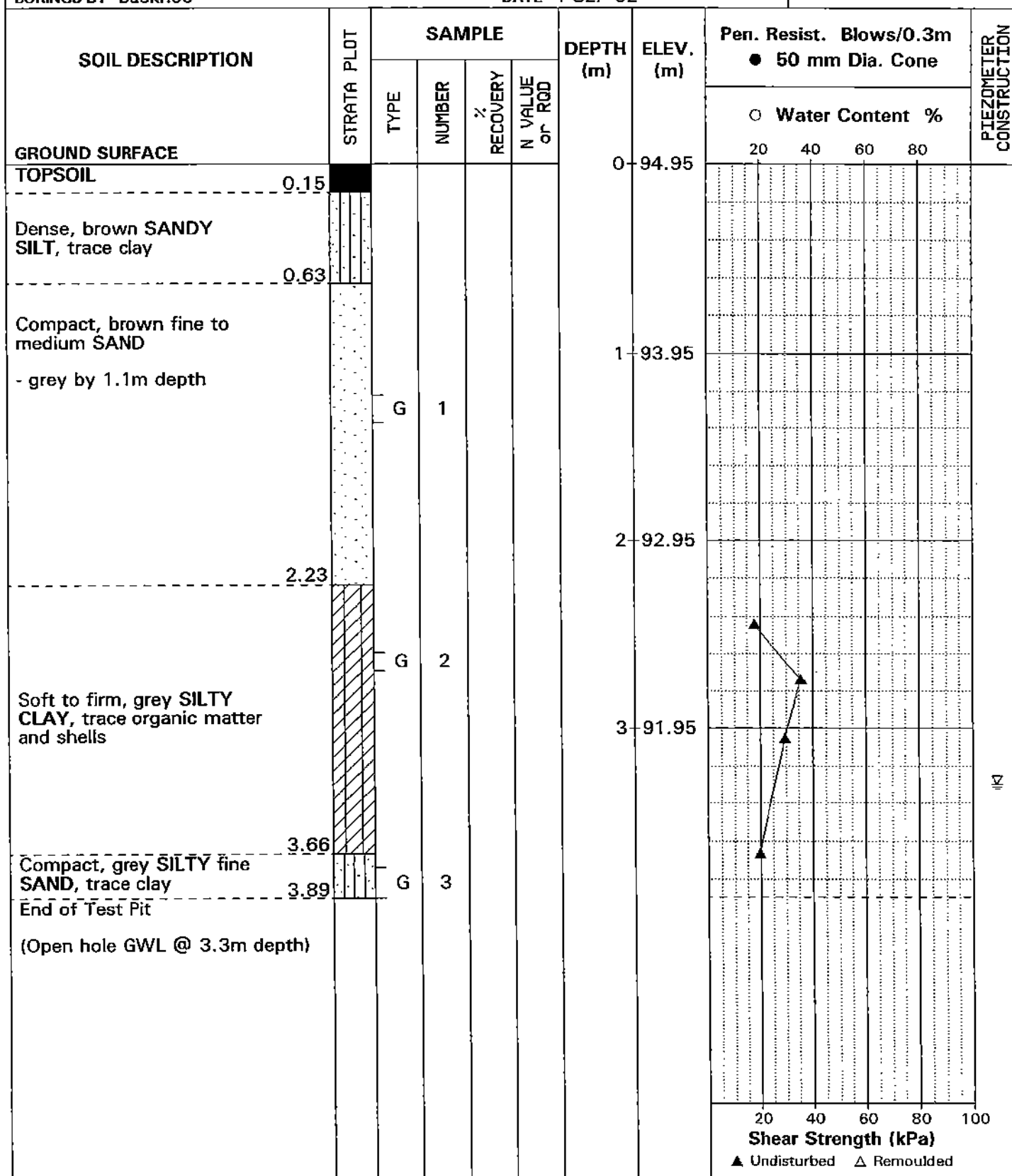
REMARKS

HOLE NO.

TP 5

BORINGS BY Backhoe

DATE 4 SEP 02



SOIL PROFILE & TEST DATA

**Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO. **G8689**

REMARKS

HOLE NO. TP 7

BORINGS BY Backhoe

DATE 4 SEP 02

[illegible]



JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

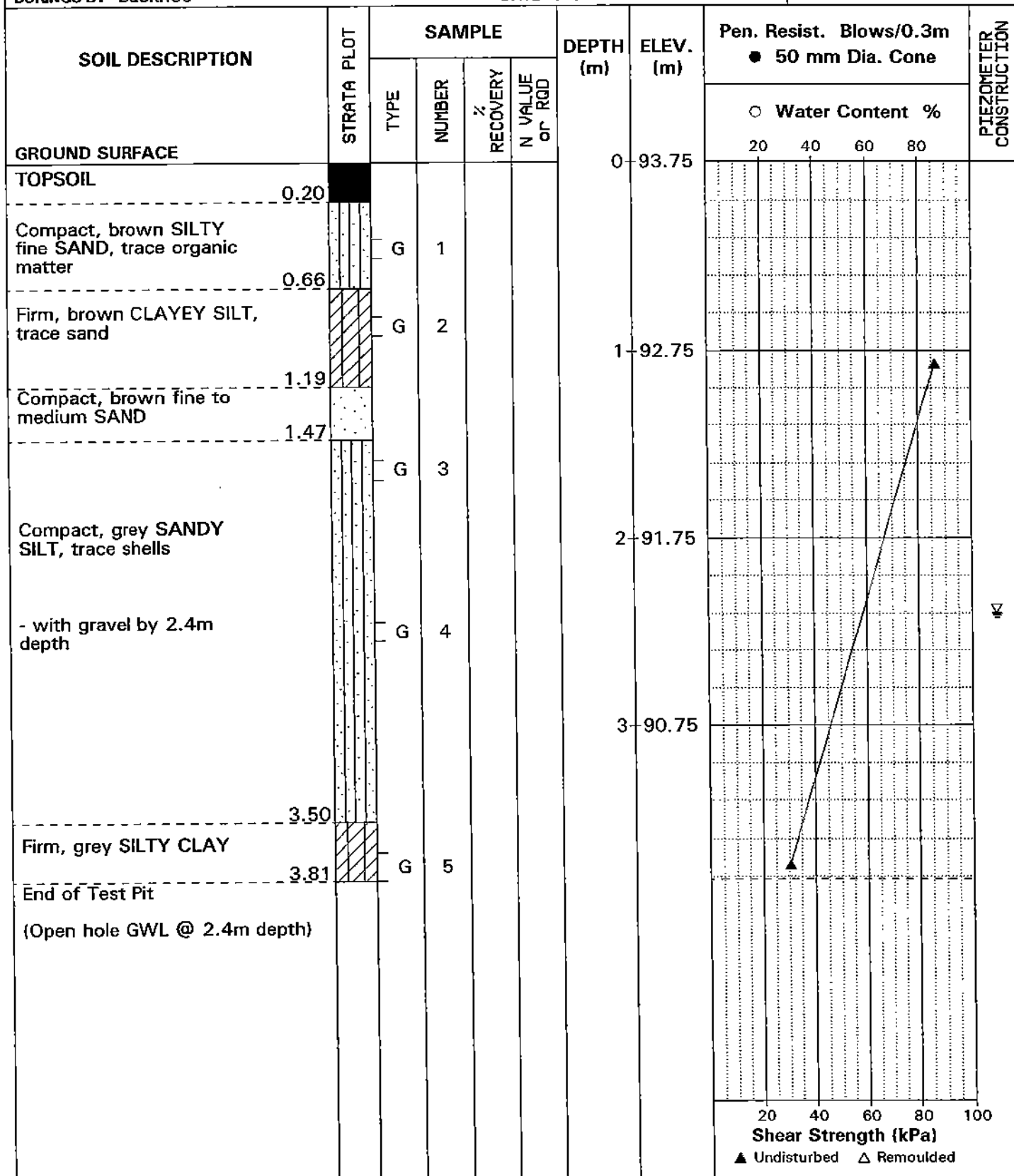
REMARKS

HOLE NO.

TP 9

BORINGS BY Backhoe

DATE 5 SEP 02



**JOHN D. PATERSON & ASSOCIATES LTD.**

Consulting Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

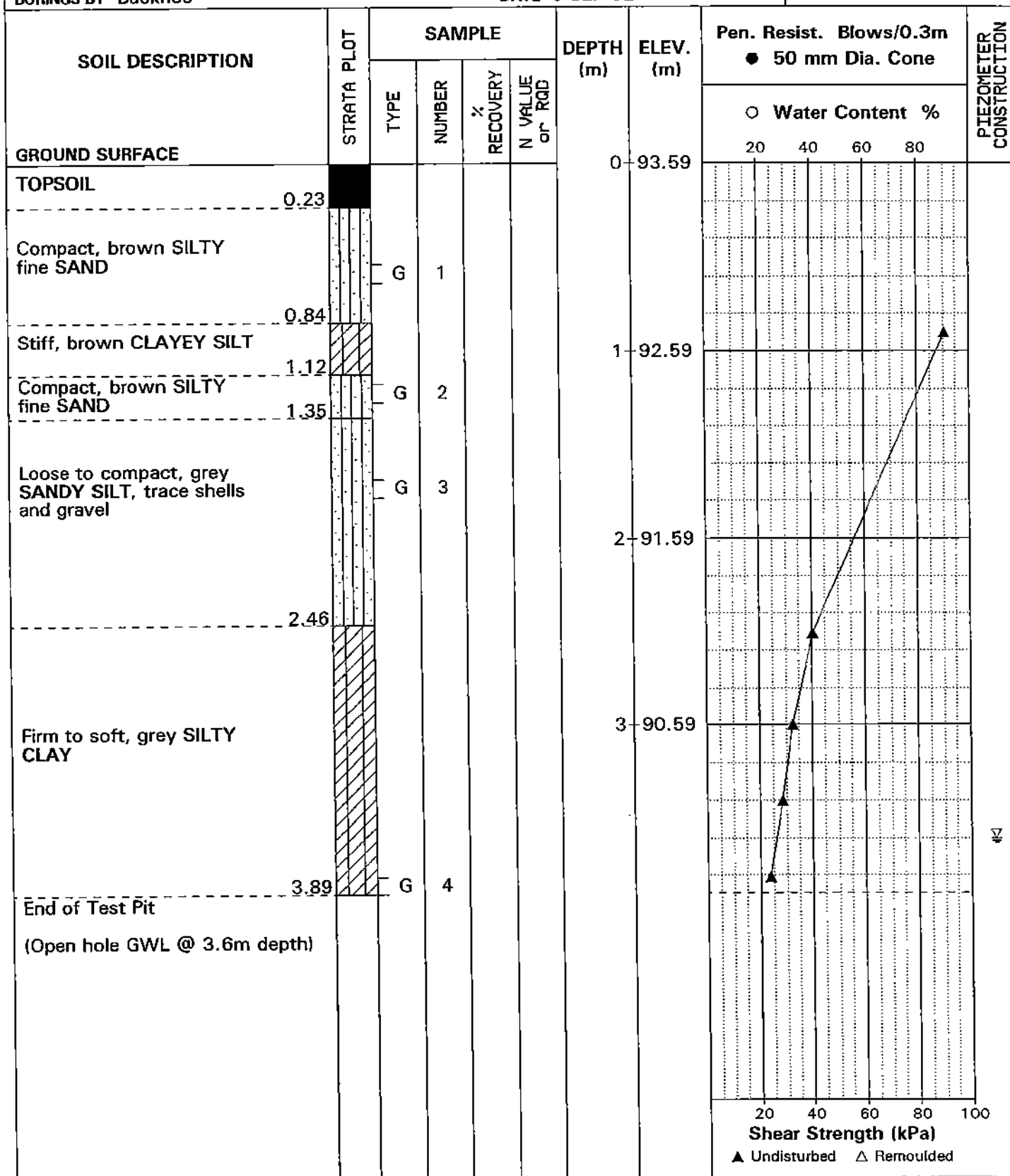
REMARKS

HOLE NO.

TP10

BORINGS BY Backhoe

DATE 5 SEP 02



**JOHN D. PATERSON & ASSOCIATES LTD.**

Consulting Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

REMARKS

HOLE NO.

TP11

BORINGS BY Backhoe

DATE 5 SEP 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	96.91					
TOPSOIL												
0.25												
Compact, light brown												
0.36												
SILTY fine SAND												
			</									

JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

First Line Road @ Campeau Drive

Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

REMARKS

FILE NO.

G8689

HOLE NO.

TP12

BORINGS BY Backhoe

DATE 5 SEP 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	95.68					
TOPSOIL												
----- 0.38 -----												
Compact, brown SILTY fine SAND	G	1				1	94.68					
----- 1.73 -----												
Firm, brown SILTY CLAY	G	2				2	93.68					
----- 1.90 -----												
Compact, brown medium to coarse SAND with shells	G	3				3	92.68					
----- 3.89 -----												
End of Test Pit												
(Open hole GWL @ 2.2m depth)												

Shear Strength (kPa)
 ▲ Undisturbed △ Remoulded

SOIL PROFILE & TEST DATA

**Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO. **G8689**

REMARKS

HOLE NO. TP13

BORINGS BY Backhoe

DATE 5 SEP 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	98.25					
TOPSOIL												
0.30												
Loose, brown SILTY fine SAND, occasional gravel (possible fill)		G	1									
		G	2			1	97.25					
1.83												
Compact, brown fine to medium SAND		G	3			2	96.25					
2.59												
End of Test Pit (TP dry upon completion)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded



JOHN D. PATERSON & ASSOCIATES LTD.

Consulting Engineers

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation

First Line Road @ Campeau Drive

Ottawa (Kanata), Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO.

G8689

REMARKS

HOLE NO.

TP14

BORINGS BY Backhoe

DATE 5 SEP 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %				
								20	40	60	80	
GROUND SURFACE						0	98.75					
TOPSOIL	0.18											
FILL: Brown silty fine sand with interbeds of hay/grass	0.76											
		G	1			1	97.75					
		G	2									
Compact, brown SILTY fine SAND						2	96.75					
						3	95.75					
		G	3									
End of Test Pit (TP dry upon completion)	3.60											
								20	40	60	80	100
								Shear Strength (kPa)				
								▲ Undisturbed △ Remoulded				

28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

**Geotechnical Investigation
First Line Road @ Campeau Drive
Ottawa (Kanata), Ontario**

DATUM Ground surface elevations provided by Annis, O'Sullivan and Vollebakk Limited.

FILE NO. **G8689**

REMARKS

HOLE NO. **TP15**

BORINGS BY Backhoe

DATE 5 SEP 02

[illegible]

SOIL PROFILE & TEST DATA

Geotechnical Investigation

Campeau Drive Extension at the Carp River
Ottawa (Kanata), Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

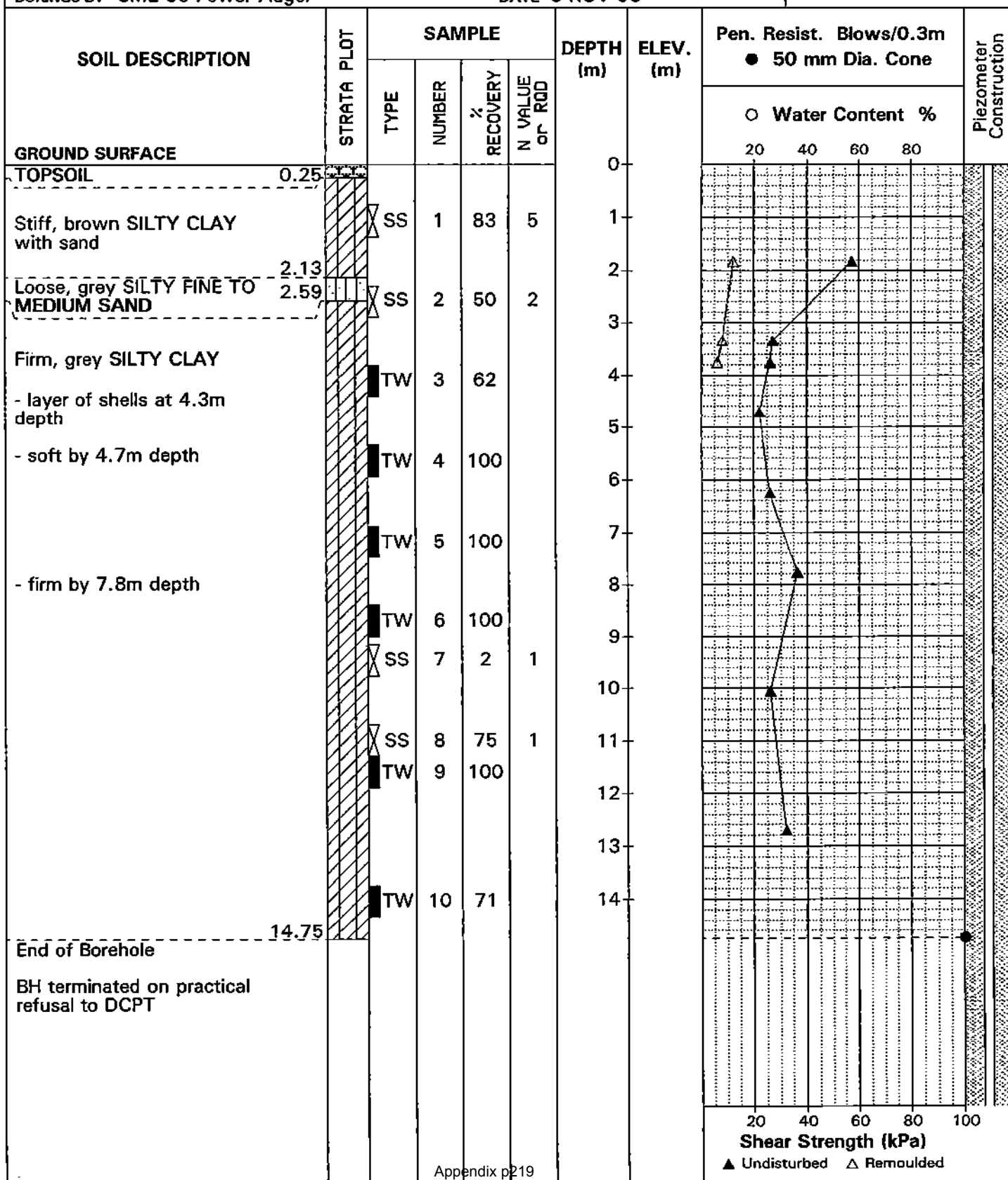
DATE 9 NOV 06

FILE NO.

PG0909

HOLE NO.

BH 1



SOIL PROFILE & TEST DATA

Geotechnical Investigation

Campeau Drive Extension at the Carp River
Ottawa (Kanata), Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

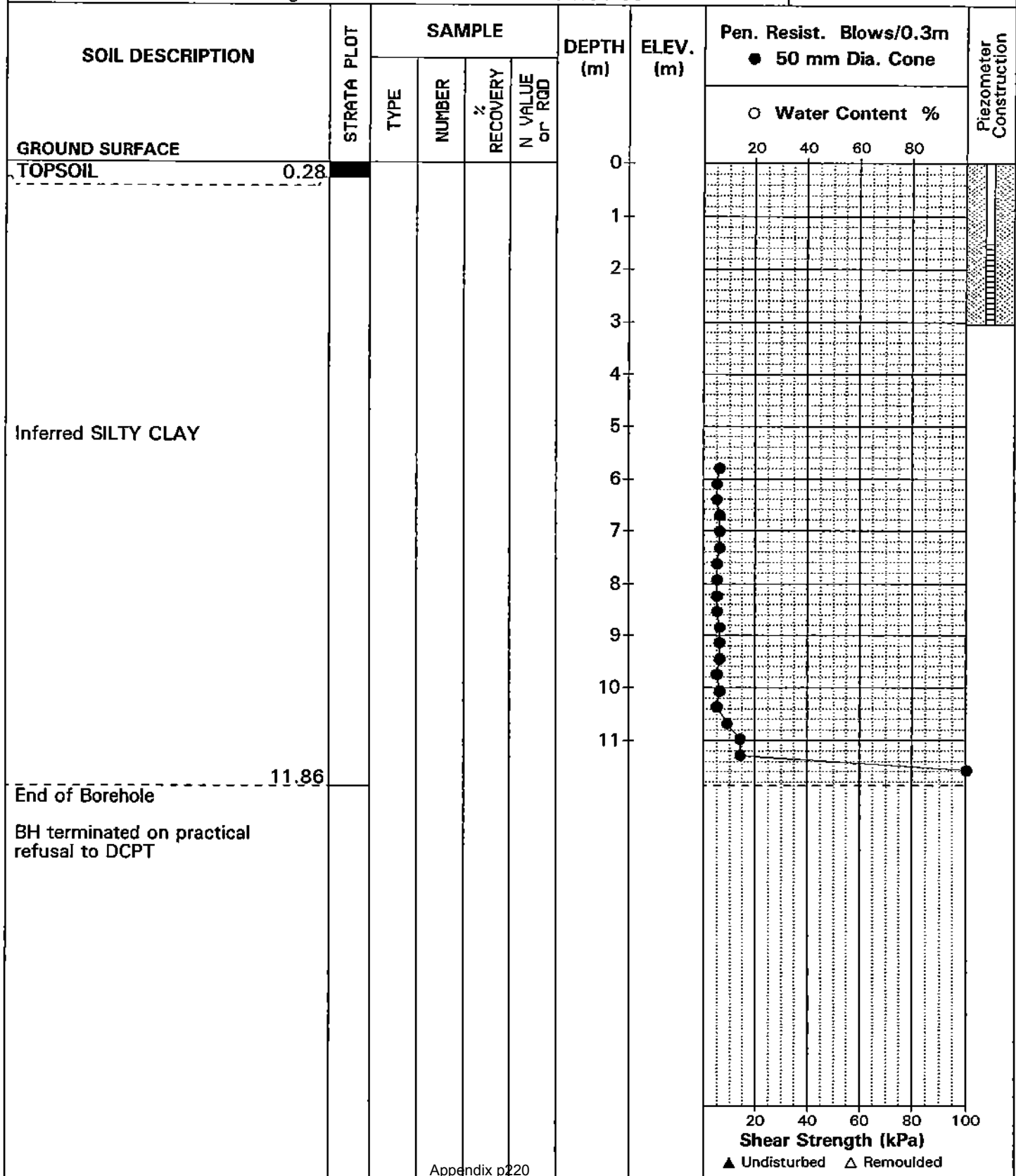
DATE 9 NOV 06

FILE NO.

PG0909

HOLE NO.

BH 2



SOIL PROFILE & TEST DATA

Geotechnical Investigation

Campeau Drive Extension at the Carp River
Ottawa (Kanata), Ontario

DATUM

FILE NO.

PG0909

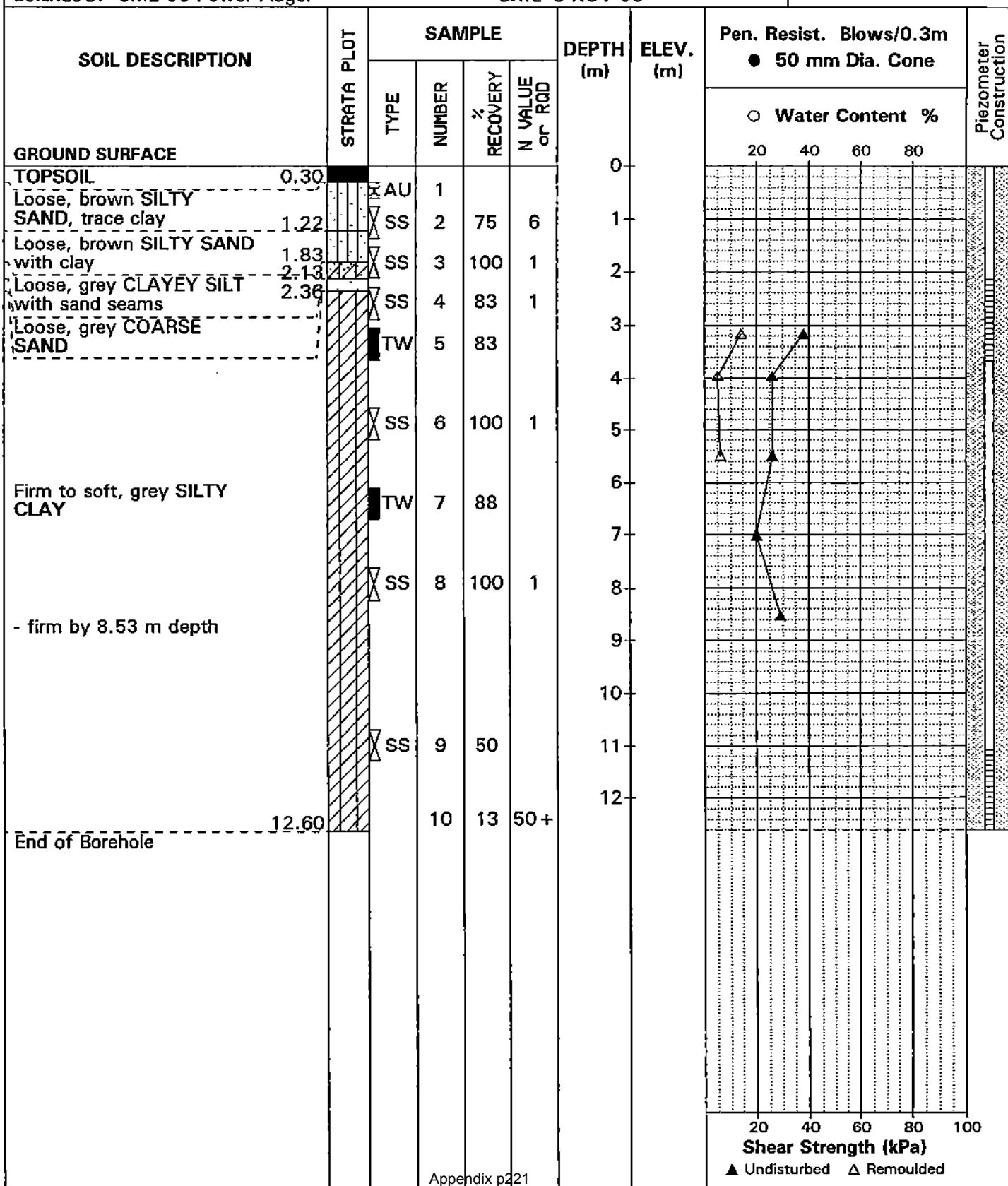
REMARKS

HOLE NO.

BH 3

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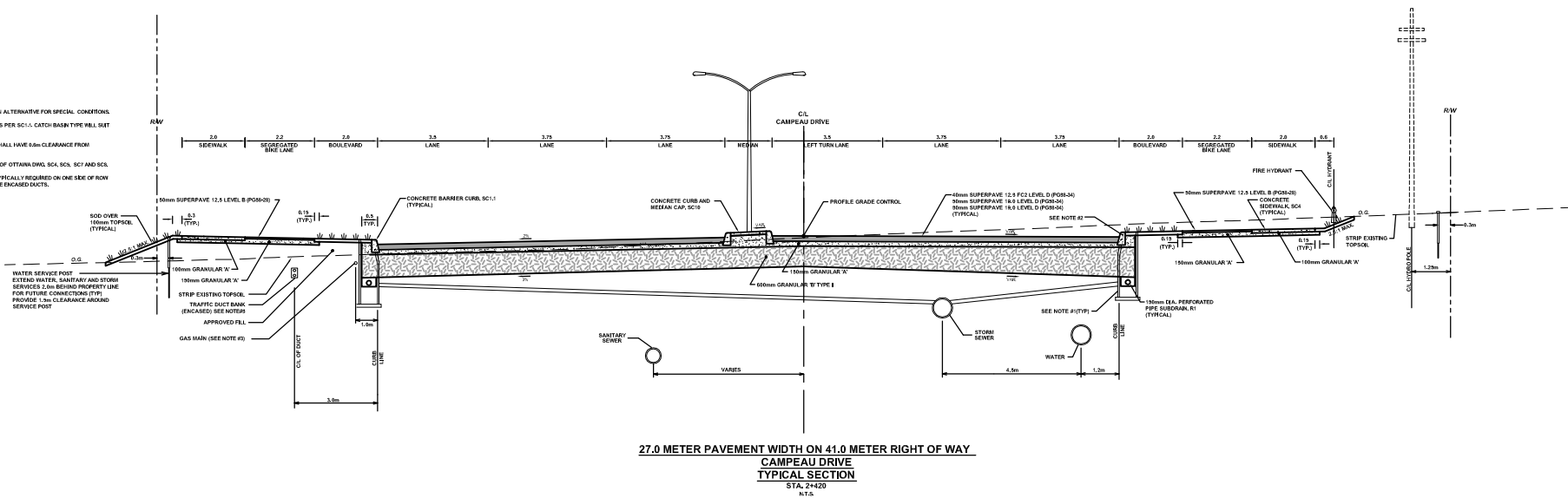
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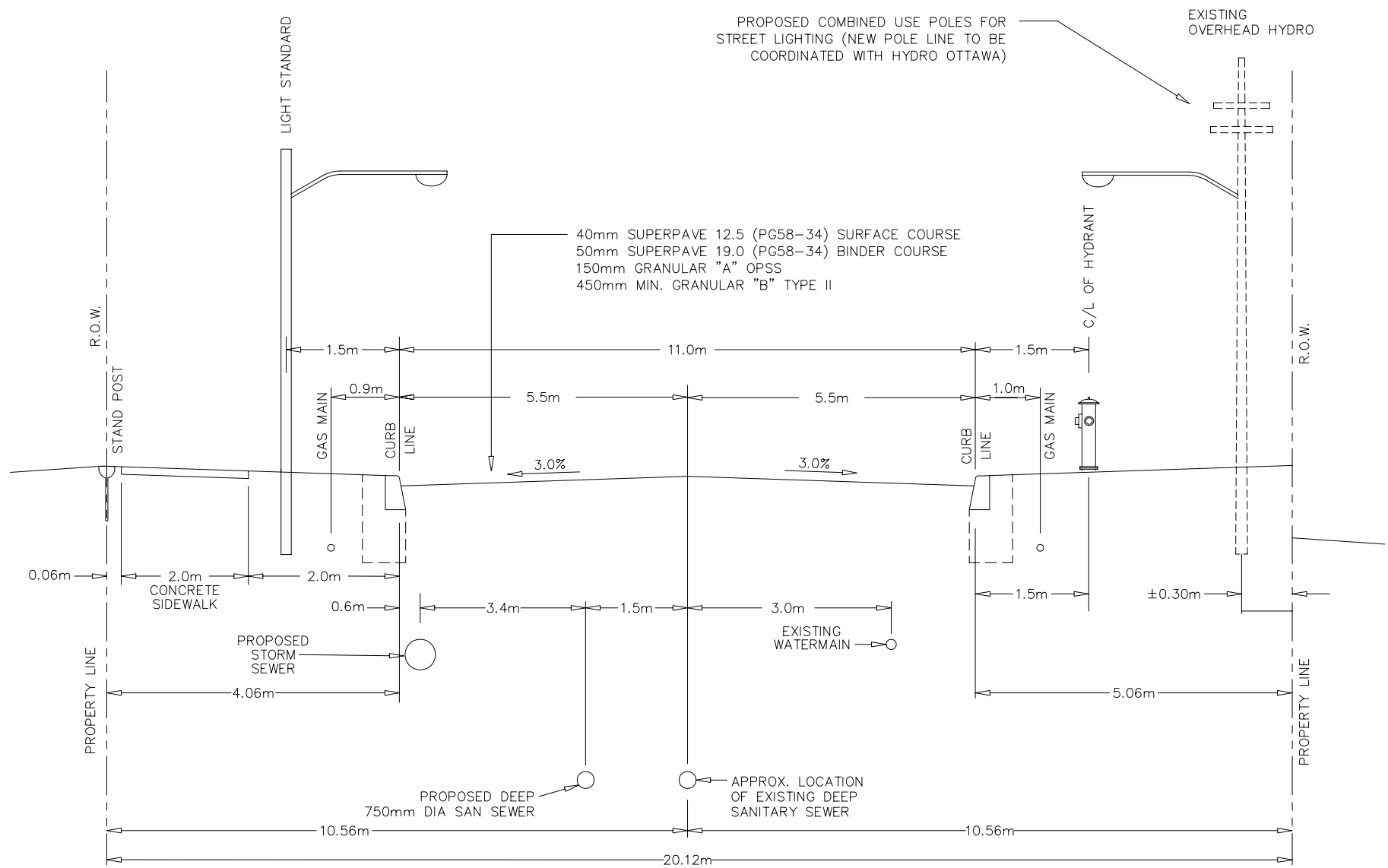


Appendix E

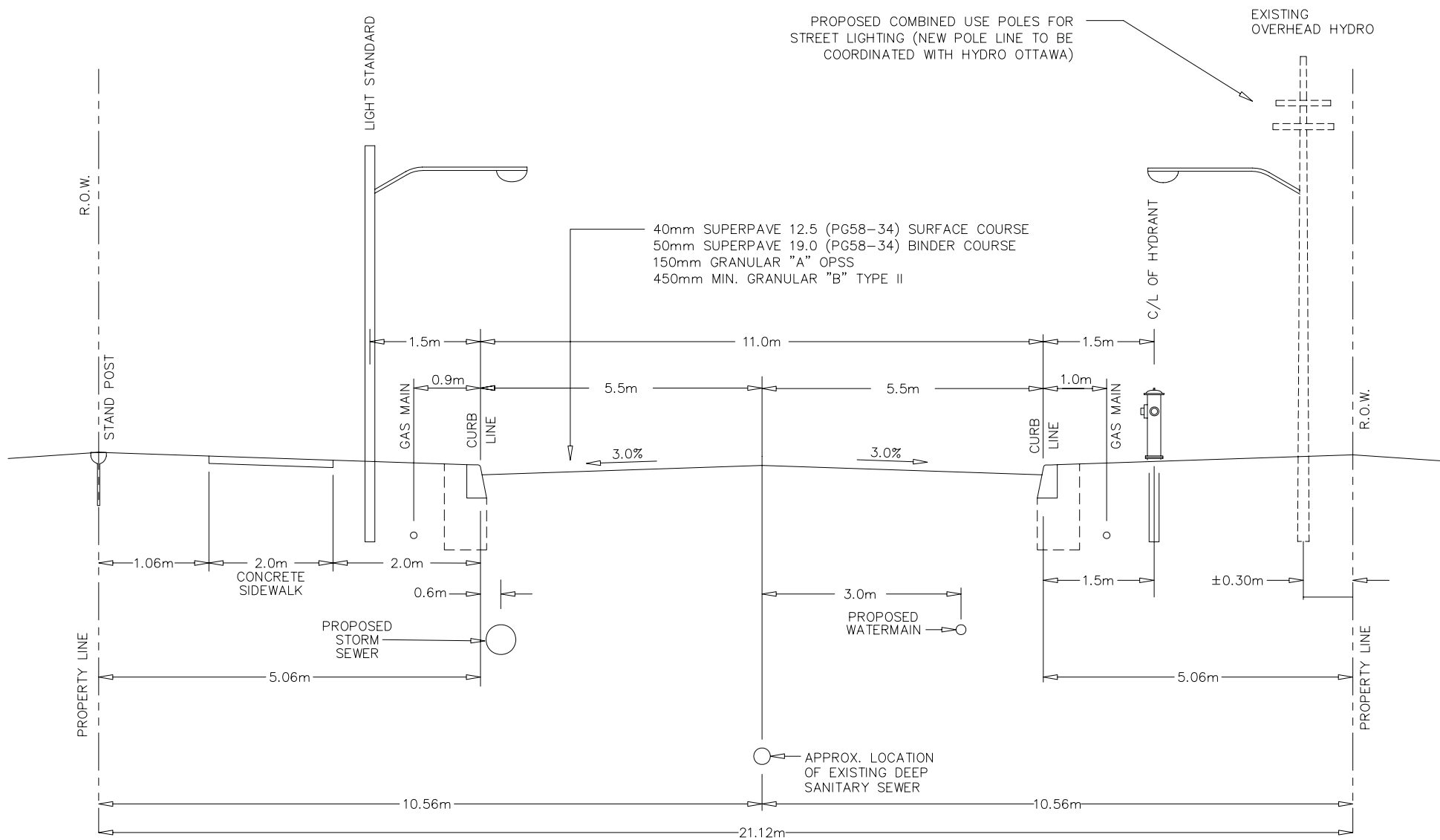
Road Cross Sections

1. CURB INLET CATCH BASIN MAY BE USED AS AN ALTERNATIVE FOR SPECIAL CONDITIONS.
2. CONCRETE CURBS SHALL BE BARRIER TYPE AS PER SCS-5. CATCH BASIN TYPE WILL SAT CURB DESIGN.
3. AT CATCH BASIN LOCATIONS THE GAS MAIN SHALL HAVE 0.6m CLEARANCE FROM STRUCTURES.
4. SIDEWALK SHALL BE INSTALLED AS PER CITY OF OTTAWA DNG, SC4, SC5, SC7 AND SC8.
5. TRAFFIC DUCT BANK (COVERED) TYPICALLY TYPICALLY REQUIRED ON ONE SIDE OF ROW ONLY. PROVIDE 1.0m COVER ON ALL CONCRETE ENCASED DUCTS.



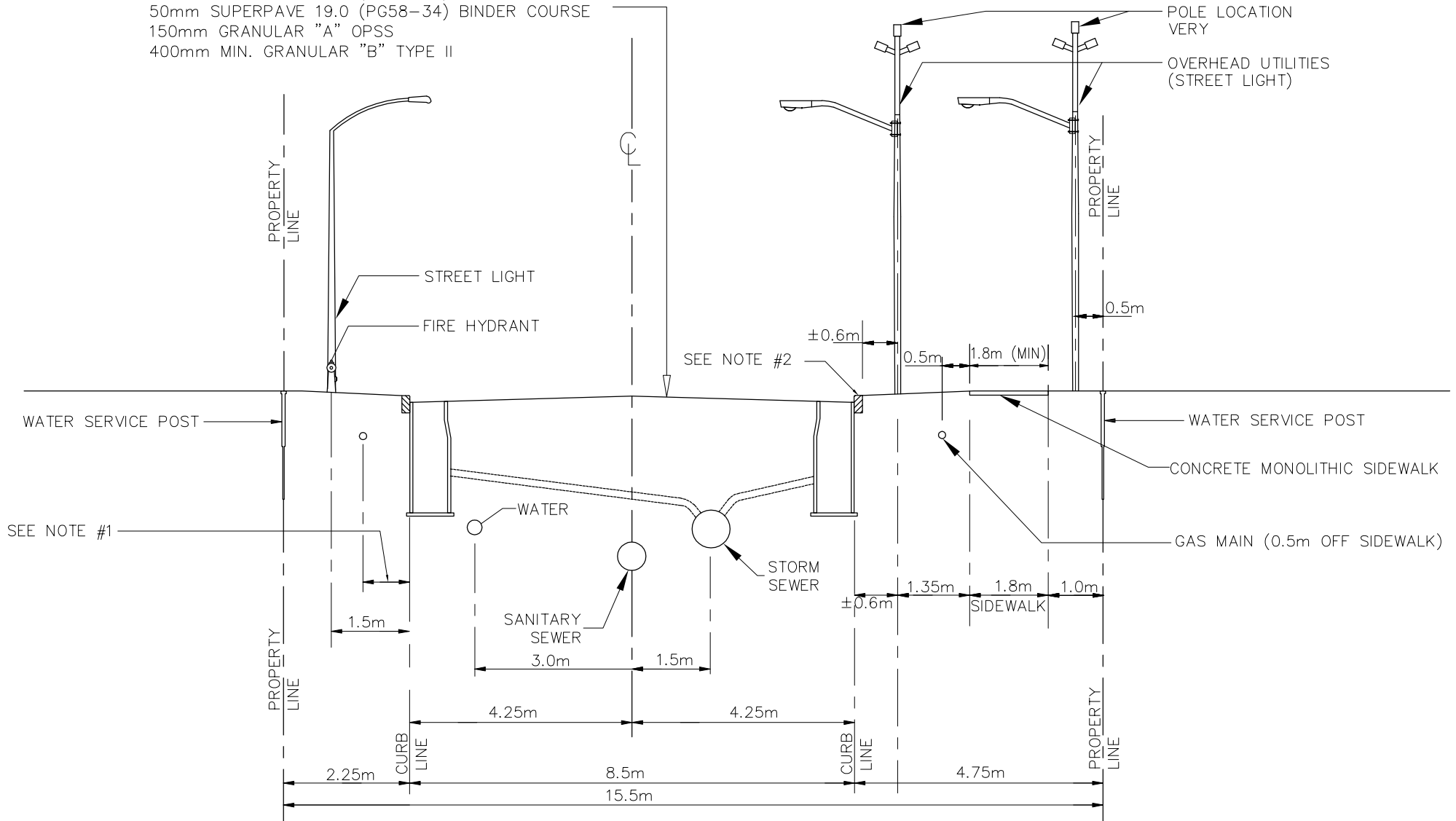


**11.0 METRE PAVEMENT WIDTH ON 20.12 METRE RIGHT OF WAY
 DIDSBURY ROAD
 NORTH OF CAMPEAU DRIVE
 (FOR ILLUSTRATION OF HORIZONTAL UNDERGROUND SERVICE LOCATIONS ONLY)**



**11.0 METRE PAVEMENT WIDTH ON 21.12 METRE RIGHT OF WAY
 DIDSBUY ROAD
 SOUTH OF CAMPEAU DRIVE
 (FOR ILLUSTRATION OF HORIZONTAL UNDERGROUND SERVICE LOCATIONS ONLY)**

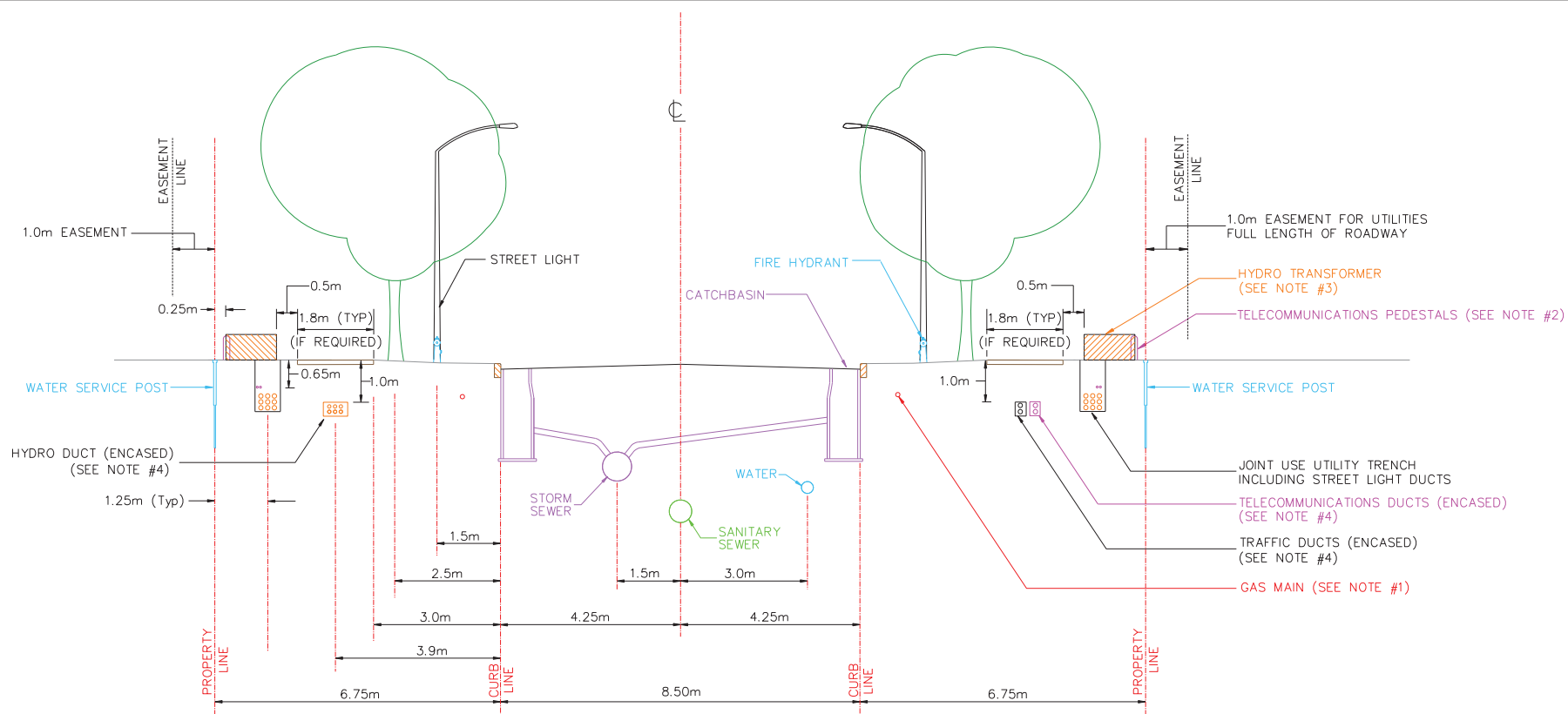
40mm SUPERPAVE 12.5 (PG58-34) SURFACE COURSE
 50mm SUPERPAVE 19.0 (PG58-34) BINDER COURSE
 150mm GRANULAR "A" OPSS
 400mm MIN. GRANULAR "B" TYPE II



NOTE:

1. AT CATCH BASIN AND HYDRANT LOCATIONS THE GAS MAIN SHALL HAVE A MINIMUM 0.6m CLEARANCE FROM STRUCTURES.

8.5 METRE PAVEMENT WIDTH ON 15.5m RIGHT OF WAY
ROGER NEILSON WAY
 (N.T.S.)



NOTES:

1. REFERENCE STANDARD NOTES ROAD ALLOWANCE (DGN:ROW-NOTES).
2. AT CATCH BASIN LOCATIONS THE GAS MAIN SHALL HAVE 0.6m CLEARANCE FROM STRUCTURES.
3. ALL PEDESTALS TO BE INSTALLED IN LINE WITH HYDRO TRANSFORMERS OR ON SIDE OF TRENCH AWAY FROM ROAD.
4. REQUIREMENT FOR PROTECTIVE BOLLARDS AT TRANSFORMERS SHALL BE DETERMINED BY HYDRO ON A CASE BY CASE BASIS.
5. HYDRO DUCTS & COMMUNICATION DUCTS (ENCASED) TYPICALLY REQUIRED ON ONE SIDE OF ROW ONLY. PROVIDE 1.0m COVER ON ALL CONCRETE ENCASED DUCTS.



RESIDENTIAL ROAD 22.0m ROAD ALLOWANCE SIDEWALK OPTIONS

W. R. Newell
W. R. Newell, P. Eng.

Director, Infrastructure Services Branch

Scale: 1:100 Revision: 4

Date: December 1, 2006

Dwg/Dgn: ROW-22 Sheet: 11 of 13

Appendix F

Delcan Traffic Impact Study Addendum

#1



1223 Michael Street, Suite 100, Ottawa, ON K1J 7T2
Tel: 613.738.4160 • Fax: 613.739.7105
www.delcan.com

May 18, 2007

OUR REF: TO1154TOC00

Transportation Planning, Transit and
the Environment Department
City of Ottawa
110 Laurier Avenue West, 4th Floor
Ottawa, Ontario K1P 1J1

Attention: Mr. Ed Blaszyński
Infrastructure Approvals Officer, Transportation

Dear Mr. Blaszyński:

**Re: Taggart/Loblaws Concept Plan, Kanata West TIS
Addendum #1, Response to City Comments**

Executive Summary

At our meeting on April 13, 2007, it was agreed that this Addendum should address the 2008 (Loblaws opening) road modification requirements on Campeau Drive, west of Terry Fox Drive, and Didsbury Road north to Terry Fox Drive. It was also agreed that longer-term conditions of the Terry Fox/Goulbourn Forced Road Intersection would also be addressed. The findings and recommendations on these topics follow.

To accommodate the planned Loblaws, RONA and Dymon site developments, the following network improvements (depicted in Figure 1) are recommended on **Campeau Drive**:

- A two-lane cross-section (i.e. one eastbound and one westbound lane) from the most westerly, shared Loblaws/RONA driveway (i.e. Access #1) easterly to the right-in/right-out driveways (i.e. Loblaws Access #2);
- A three-lane cross-section (i.e. two eastbound lanes and one westbound lane) from Loblaws Access #2 east to the Didsbury Road Intersection;
- A four-lane cross-section (i.e. two eastbound and two westbound lanes) from Didsbury Road easterly to Terry Fox Drive.
- A protected left-turn phase should be provided for eastbound and westbound Campeau Drive volumes at the Terry Fox/Campeau Intersection to ensure that these movements have sufficient greentime, given the heavy opposing through volumes;
- Signalization of the Campeau/Didsbury Intersection is recommended from Loblaws opening (i.e. 2008). On Campeau Drive, opposing left-turn lanes and single, shared through/right-turn lanes are recommended on the eastbound and westbound

approaches. A single all-movement lane on the northbound and southbound Didsbury Road approaches would be sufficient to accommodate projected 2008 volumes.

It is also recommended that **Didsbury Road** be extended to connect with the existing Terry Fox/Signature Centre Intersection, which is currently outfitted with appropriate signals and turn-lanes.

With respect to the **Terry Fox/Goulbourn Forced Road intersection**, the following Interim Improvements would be required (in lieu of the mini-interchange) for the intersection to operate at Level of Service D or better during peak hours, based on the projected 2008 volumes and prior to the completion of the Campeau Drive extension:

- A double northbound left-turn lane on Terry Fox Drive with approximately 120 m of storage, which would require that the existing median on the opposing (north) be widened and that a second westbound receiving lane be constructed on Goulbourn Forced Road. Given the impacts, this modification may not be desirable in the short-term.
- A southbound right-turn (SBR) lane on Terry Fox Drive;
- A double eastbound right-turn lane with approximately 105 m storage per lane, and a separate signal phase, on the west leg of the Terry Fox/Goulbourn intersection;
- Also on the eastbound leg, a dedicated left-turn lane (with a minimum 25 m storage length) and separate through-lane would be required on the Goulbourn Forced Road.

Introduction

Thank you for meeting with Delcan on Friday, April 13, 2007, to discuss the City-concerns relating to the planned Taggart/Loblaws located within the Kanata West Development Area (KWDA). This meeting was helpful in determining what additional information the City requires to approve the Taggart/Loblaws Site Plan Application, which will directly impact the approval process for the proposed RONA and Dymon developments. At this meeting, it was agreed by City staff (yourself, Steve Stoddard and Bill Jolliffe) that a letter would be an acceptable means of addressing the following three topics for which you require additional information:

- Campeau Drive extension from Terry Fox Drive westerly (timing, cross-section and traffic operations);
- Didsbury Drive extension from Campeau Drive northerly to Terry Fox Drive (timing, cross-section and traffic operations); and
- Terry Fox Drive/Goulbourn Forced Road intersection improvement options.

As discussed at the above-noted meeting, IBI Group (Kevin Harper) indicated that RONA would be submitting its subdivision application the week of April 20th, 2007. This has not yet occurred but is expected to shortly. The required Transportation Impact Assessment (TIA), entitled *RONA Home and Garden Store: Kanata West Traffic Impact Study* (February 14, 2007), was prepared by Delcan for IBI Group and will be included with this application. The RONA TIS addresses a number of the comments made by the City in their review of the

Taggart/Loblaws TIS, particularly with respect to 2013 projections and site access details. This letter, therefore, is intended to respond to the three remaining City comments and should accompany the RONA TIS to close the loop.

Background

The following retail developments are planned for the lands north of Highway 417, west and south of Terry Fox Drive and east of Huntmar Drive:

- an 11,700 m² (126,183 ft²) RONA with an additional 2,500 m² (26,880 ft²) outdoor garden sales area;
- a 16,500 m² (177,605 ft²) Loblaws grocery store with 5,400 m² (58,125 ft²) ancilliary retail and a 6-pump gas bar;
- a 12,588 m² (135,505 ft²) drive-in self-storage facility with a separate single-storey 4,056 m² (43,660 ft²) retail development and a 511 m² (5,500 ft²) stand-alone retail development; and,
- a 4,705 m² (previously proposed) retail development.

It is noteworthy that Richcraft Homes is currently revisiting its original proposal of a 4,705 m² retail development; however, an updated traffic impact study is not available at this time. In addition, Canadian Tire plans to expand its existing building, located within the development area in question, by 35,000 ft² (3,250 m²); however, the expansion is intended to increase the width of in-store aisles and will not impact the type and volume of products sold. Thus, an increase in the number of site-generated trips is not anticipated.

Delcan has prepared numerous transportation documents for these commercial and retail developments including the following:

- Taggart/Loblaws Concept Plan Kanata West Transportation Study (April 26, 2004)
- Taggart/Loblaws concept Plan Kanata West Transportation Study (February 28, 2005)
- Richcraft Goulbourn Forced Road Retail Site Traffic Impact Overview (March 2006)
- Richcraft Goulbourn Forced Road Retail Site Traffic Impact Overview Addendum #1 (May 2006)
- RONA Home and Garden Store: Kanata West (February 14, 2007)
- Dymon Mixed Use Development, Didsbury Road, Kanata TIS (March 7, 2007)

Each study considers on-site circulation, parking, site accesses and adjacent street impacts. Whenever possible, the analysis considers known site-traffic generation for other planned developments. Thus, with each successive transportation study, more is known about the adjacent land uses and anticipated traffic generation, thereby enabling the development of a more complete picture of the overall traffic impacts.

From the list of studies provided above, the *Dymon Mixed Use Development, Didsbury Road, Kanata TIS* (March 7, 2007) provides the most comprehensive analysis of impacts to the Terry Fox Drive/Goulbourn Forced Road intersection and the Private Road located adjacent to the south of the Canadian Tire store. The *RONA Home and Garden Store: Kanata West TIS*, which the City is anticipated to receive shortly, provides the most up-to-date analysis of Campeau Drive and Didsbury Road impacts and requirements.

Infrastructure Improvements

Campeau Drive

Currently, Campeau Drive begins at March Road in the east and terminates in the west at a T-Intersection with Didsbury Road. The City of Ottawa has recently initiated an Environmental Assessment for the 4-laning of Campeau Drive between March Road and Didsbury Road within the Kanata Urban Area. The EA does not include the section of roadway from Didsbury Road westerly to Huntmar Drive. It is our understanding that, while the City acknowledges the need for this roadway to service the planned retail developments, it is not identified in the capital budget and funding for its design and construction is not available at this time. It should be noted that the EA for a four-lane Campeau Drive from Didsbury Road west to Huntmar Drive has been completed by the Kanata West Owners Group, and it is before the MOE for approval.

Once extended west to Huntmar Drive, Campeau Drive would provide an alternate travel route to/from the existing and planned retail developments for Stittsville residents and others living within the KWDA. The connection to Huntmar Drive would also provide an alternative route to/from Highway 417 thereby relieving some of the pressure from the Terry Fox Drive Intersections. Until the Campeau Drive link is in place, the "catchment area" of the proposed developments would be smaller than envisioned in projecting site-traffic. Thus, it is unlikely that the projected 2008 and 2013 site-traffic will be realised without this connection. Consumers may choose less cumbersome routes to more accessible retail stores until such time as the Campeau Drive connection to Huntmar Drive is complete. Regardless, the transportation analysis contained herein assumes that the projected site-traffic generation volumes are achieved.

The proposed RONA store is planned for the northwest quadrant of the Didsbury/Campeau Intersection, adjacent to both Campeau Drive and Didsbury Road. The Taggart/Loblaws store development is planned for the southwest quadrant. For the RONA site, two driveway connections are proposed off of the Campeau Drive extension, which will be located opposite the approved accesses to the Taggart/Loblaws development, to the south. The more westerly access, which is located approximately 250 m west of Didsbury Road, will be signalized. The more easterly site connections will be right-in/right-out only to ensure that signals will not be warranted (as appropriate signal spacing could not be achieved). These connections are located approximately 150 m west of Didsbury Road. The Campeau/Didsbury Intersection is likely to require signalization in future.

Based on discussions with IBI Group, it is our understanding that Taggart/Loblaws and RONA intend to provide both the Campeau Drive and the Didsbury Road extensions, through agreements with the City of Ottawa, as they are essential for site access. West of these sites, however, the completion of Campeau Drive is not anticipated to occur before store opening. It is our understanding that Minto will be constructing the Carp River crossing in conjunction with its Arcadia subdivision. From discussion with Minto staff (Barry Gray), the bridge could be constructed as early as 2008, but the subdivision is likely to build-out from Huntmar Drive eastward due to less favourable soil conditions on the eastern portion of the site. Thus, in our estimate, completion of (a minimum two-lane) Campeau Drive west to Huntmar Drive is not likely to occur prior to 2010.

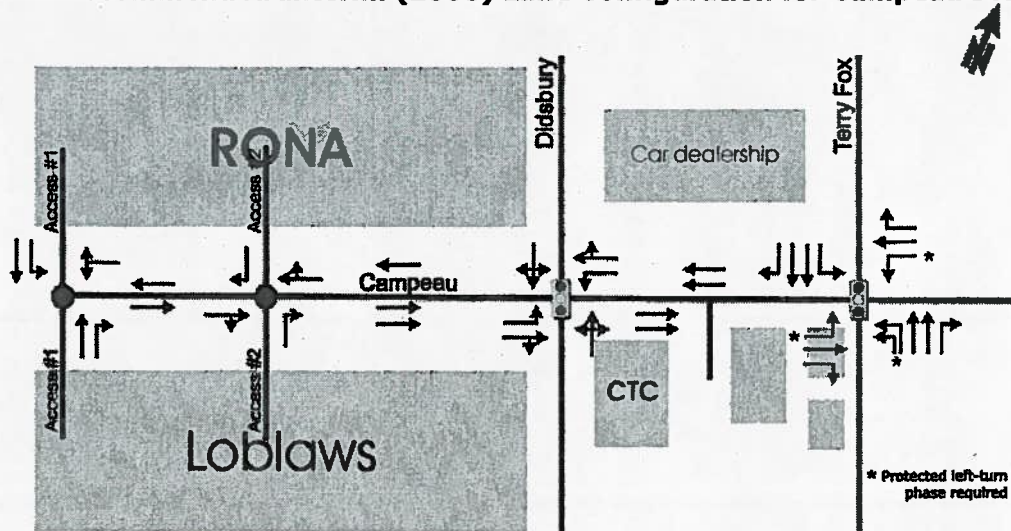
The absence of the Campeau Drive west connection to Huntmar Drive at site build-out (i.e. 2008) will impact traffic distribution and increase the short-term demands on the other network intersections, namely Campeau Drive/Terry Fox Drive and Terry Fox Drive/Goulbourn Forced Road. As the recently-submitted RONA and Dymon TIS's assumed that Campeau Drive between Terry Fox and Huntmar Drive would be in place at store opening (2008), we have revisited the analysis herein to determine 2008 traffic conditions assuming that Campeau Drive terminates immediately west of the westerly RONA/Loblaws site connections. With this in mind, based on the likely general distribution of households within the catchment area of the proposed store, the following is the assumed directional split of generated traffic by opening day, 2008.

- to/from the north (Terry Fox Dr/Didsbury Road) 15%
- to/from the south/east (Terry Fox Dr/Campeau Dr) 85%

Synchro (version 6) software was used to determine the roadway and intersection requirements prior to the completion of Campeau Drive, from the Carp River crossing westward. Traffic that was previously assigned to travel westbound in the Dymon TIS and RONA TIS was reassigned to the Campeau/Terry Fox intersection for the 2008 analysis year. Approximately 100 (50 inbound and 50 outbound) and 120 (60 inbound and 60 outbound) vehicle trips per hour were redistributed from westbound Campeau Drive to travel eastbound, through the Terry Fox/Campeau intersection. Of the two analysis periods, the Saturday afternoon peak hour was the critical one, which is not unexpected given that Campeau Drive will be primary road link for several retail uses and, until it is extended to Huntmar Drive, will not be used by commuter traffic.

From the conclusions and recommendations of the various transportation impact assessments and re-analysis of 2008 conditions (described previously), the following Interim (2008) Campeau Drive transportation network improvements, as depicted in Figure 1, are recommended:

Figure 1: Recommended Interim (2008) Lane Configuration for Campeau Drive



- Widen the existing Campeau Drive cross-section to provide a four-lane cross-section between Terry Fox Drive and Didsbury Road. With two eastbound lanes and two westbound lanes, vehicles travelling to/from the existing and planned retail sites will have an easier time entering/existing. A two-lane cross-section would result in significant delays to outbound vehicles (LoS F).
- Extend Campeau Drive west of Didsbury Road to the westerly Loblaws and RONA Intersection pair (i.e. Access #1). A two-lane cross-section (i.e. one eastbound and one westbound lane) from Access #1 easterly to the paired right-in/right-out accesses (i.e. Access #2) should be sufficient to accommodate projected 2008 site-traffic volumes. A double westbound left-turn (WBL) should be provided in the long-term.

From Access #2 (i.e. the easterly, right-in/right-out connections) to Campeau Drive, a three-lane cross-section would be sufficient to accommodate Loblaws and RONA site-generated traffic. A single westbound lane and two eastbound lanes would be required to ensure that the right-out connection from Loblaws (i.e. Access #2) operates at an acceptable Level of Service. The additional eastbound lane would be carried to the signalized Didsbury/Campeau Intersection, at which opposing left-turn lanes and shared through/right-turn lanes are proposed on the Campeau Drive approaches.

The results of the Synchro analysis suggests that the outbound southbound left-turn movement from the RONA site will fall without all-way STOP-control, prior to the installation of signals. However, it is our experience that Synchro typically overestimates delay to cross-traffic (by over-estimating the critical gap). Furthermore, if delays are significant, vehicles will opt for the site connection to Didsbury Road. According to Synchro, three-way STOP-control at the Campeau/Access #1 Intersection would improve traffic operations (though the WB movement is projected to operate at LoS F with this option) and a signalized intersection would operate best (LoS D or better, depending on cycle-length and lane configuration of the site accesses). We would recommend two-way STOP-control in the northbound and southbound direction at store opening (i.e. 2008), assuming the Campeau/Didsbury signals are in place. If, as Synchro suggests, delays are significant for outbound southbound left-turning RONA site-traffic, STOP-control could be implemented for westbound traffic.

- Traffic signal control should be provided at the Campeau/Didsbury Intersection from opening day (i.e. 2008). Using the MTO method, it was concluded in both the Dymon and RONA TIS's that signal warrants would not be satisfied in 2008, but would be circa 2013 (i.e. once Campeau Drive is extended to connect with Huntmar Drive, on account of the additional commuter through traffic that would result). However, in order to facilitate inbound and outbound movements at existing and planned retail sites (including Access #1), it is recommended that these signals be implemented from day one, as they would create gaps in the main traffic stream.

At the Campeau/Didsbury Intersection, a dedicated left-turn lane and shared through/right-turn lane should be provided on the Campeau Drive approaches (i.e.

eastbound and westbound). A single shared lane would be sufficient on Didsbury Road. With this geometry, and assuming a 120 second cycle-length, the intersection would operate a Level of Service B or better during the peak hours, with projected intersection delays of approximately 18 seconds or less. All movements would operate at LoS D or better during the critical peak hours.

The 50th-percentile and 95th-percentile queues on the east leg of the Campeau/Didsbury Intersection are projected at 90 and 100 m, respectively and are therefore unlikely to interfere with the existing retail connections east of the intersection. (The car dealership's access and the Canadian Tire/Staples driveway are located approximately 90 and 150 m east of the intersection.

- A double northbound left-turn lane would be required at the Terry Fox/Campeau intersection for the intersection to operate at an acceptable Level of Service (LoS D or better) by City standards. However, given the costs associated with this development and because a significant proportion of this NBL could divert to the Huntmar/Campeau intersection, once it is in place, the double NBL may be an impractical short-term consideration. Furthermore, it is unlikely that projected site-generated traffic volumes for these sites would be achieved prior to the connection of Campeau Drive to Huntmar Drive.
- A protected westbound left-turn phase will be required at the Terry Fox/Campeau intersection to facilitate left-turn movement, given the additional opposing (through) volume that is anticipated, as a result of the proposed development. A second eastbound receiving lane would be required on the east leg of the Campeau/Terry Fox intersection to accommodate the two eastbound through-lanes. Presumably this improvement will be identified in the Environmental Assessment for the 4-laning of Campeau Drive between March Road and Didsbury Road within the Kanata Urban Area.

With the above improvements in place (as illustrated in Figure 1), the Terry Fox/Campeau intersection is projected to operate at Level of Service (LoS D) during the PM and Saturday peak hours. During the PM peak hour, however, the northbound double left-turn is projected to operate at LoS F ($v/c=1.05$). In order for all movements to operate at an acceptable Level of Service during these critical peak hours, a double westbound left-turn lane would also be required. It is assumed that the geometric requirements of the east leg of this intersection will be addressed in greater detail by the ongoing Campeau Drive Extension EA.

Didsbury Road

Didsbury Road was not considered in the KWDA EA. The extension of Didsbury Road was, however, considered in the recent construction of Terry Fox Drive as a four-lane arterial north of Campeau Drive. The all-movement intersection that was constructed at Signature Centre includes signals and turn-lanes capable of accepting the eventual northerly extension of Didsbury Road to intersection Terry Fox Drive.

The proposed RONA store is to be located in the northwest quadrant of the Didsbury Road/Campeau Drive intersection adjacent to both Campeau Drive and Didsbury Road. Two

accesses to parking are proposed off the Campeau Drive extension, located opposite the approved accesses to the Taggart/Loblaws development, to the south. A truck access is proposed on Didsbury Road located north of the Campeau Drive/Didsbury Road Intersection. Further north, a fourth access to general parking is proposed off Didsbury Road.

Due to the access locations on both Campeau Drive and Didsbury Road, the TIS's assumed that Didsbury Road would be extended to the already-constructed intersection located on Terry Fox Drive at Signature Centre, which has the traffic signals heads and auxiliary lanes required to accommodate this new leg. The completion of this connection will provide valuable access to the RONA/Loblaws developments, catering to traffic from North Kanata and reducing traffic volumes at the more critical Terry Fox Drive/Campeau Drive Intersection further south.

In the absence of the Didsbury connection, the Campeau Drive/Terry Fox Intersection would have to process an additional 270 veh/h to 350 veh/h (two-way) in the southbound and eastbound left-turn lanes. While the intersection is capable of accommodating the additional traffic, it is desirable to have additional access points. Furthermore, because a significant portion of the two-lane roadway will be built to provide access to the site, it makes sense to extend the roadway to connect with the existing Terry Fox/Signature Centre Intersection, which already has the required signals and turn-lanes.

Terry Fox Drive/Goulbourn Forced Road Intersection

Based on the RONA TIS, approximately 25% of site traffic generated by Taggart/Loblaws is anticipated to use the Terry Fox Drive/Goulbourn Forced Road Intersection. In addition, the Richcraft development and the Dymon development will also be contributing site-generated trips to this intersection, which currently services Canadian Tire, a gas station and strip mall with convenience store.

Collectively, the aforementioned developments are anticipated to add a two-way total of approximately 730 vehicle trips/hour (veh/h) and 835 veh/h during the weekday and Saturday afternoon peak hours, respectively. These projected site-generated trips were superimposed on top of 2013 background traffic volumes. (It is our understanding that the timing of the mini-interchange solution is unknown at this time, but is a long-range solution.)

For reasons of accuracy, Delcan conducted a traffic count on Saturday, April 21, 2007, at the Terry Fox/Goulbourn Forced Road/Centrum Drive Intersection. As was done with the PM peak hour volumes in the RONA and Dymon Traffic Impact Studies, a 5% annual growth was applied to Terry Fox "through" traffic to increase volumes from 2007 to 2008 (build-out year). To grow traffic from 2008 to 2013 (the horizon year) an annual growth rate of +3% was applied. Overall, a growth rate of 21.7% was applied to the 2007 Saturday through volumes. The resulting 2013 background traffic volumes are shown on Figure 2.

The project site-generated traffic from the Loblaws, RONA and Dymon sites was superimposed onto background 2013 traffic volumes to determine the project total traffic for the weekday and Saturday afternoon peak hours. These volumes, which are presented in Figure 3, were analysed using Synchro.

Figure 2: Existing 2006(2007) Balanced Afternoon Peak Hour Traffic

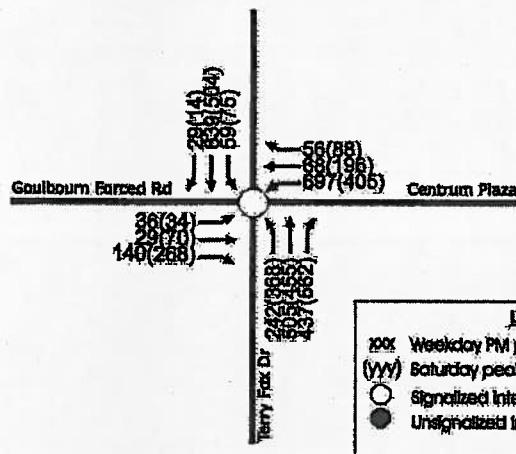
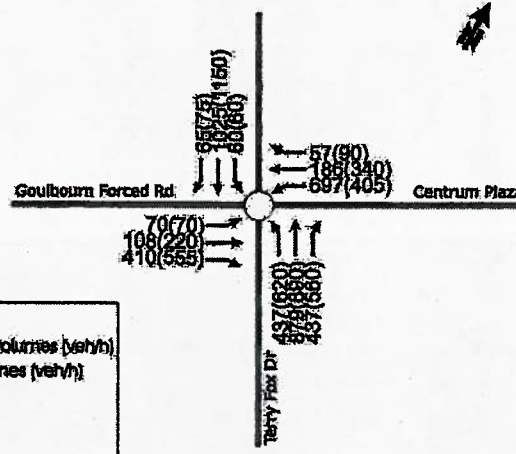


Figure 3: Projected 2013 Total Afternoon Peak Hour Traffic



Analysis of the Terry Fox/Goulbourn Intersection indicates that it could not function at an acceptable Level of Service (LoS) under projected 2013 conditions without geometric modifications. The various TIS's stated that the construction of the once-proposed "mini interchange" connecting Terry Fox Drive and Goulbourn Forced Road would address any deficiencies. However, in its 12 April 2007 e-mail response to the Dymon TIS, the City stated that "there are no identified plans for the City to construct the mini-interchange at Terry Fox Drive and Goulbourn Forced Road". As such, Delcan has endeavoured to determine an alternative short-term solution to the mini-interchange design that would accommodate projected 2013 conditions.

Using the analysis tool Synchro (version 6), it was determined that the following intersection improvements would be required for the Terry Fox Drive/Goulbourn Forced Road to support the proposed developments while operating at an acceptable LoS by City standards (i.e. LoS D or better):

Terry Fox Drive

- A double northbound left-turn lane (i.e. for movement from northbound Terry Fox Drive to Goulbourn Forced Road) is required to accommodate projected 2013 Saturday volumes (620 veh/h). Based on TAC and the City's guideline of 45%/55% lane distribution, approximately 120 m of storage would be required. To ensure that the southbound left-turn lane opposes the northbound turn-lanes, the existing median on the north leg of the intersection would have to be widened. As well, a second receiving lane would be required on Goulbourn Forced Road of sufficient length to allow the 620 vehicles per hour sufficient distance over which to merge to one lane. Provisions would also have to be made at the existing retail driveway, located on Goulbourn Forced Road approximately 45 m west of Terry Fox Drive to ensure, safe access/egress with minimal disruption to the flow of northbound left-turning vehicles. This, however, may be difficult to accomplish.

- A southbound right-turn (SBR) lane (i.e. for movement from Terry Fox Drive to Goulbourn Forced Road) would increase the overall Intersection capacity so as to achieve the desired Intersection Level of Service (i.e. LoS D or better). According to TAC, a minimum storage length of 21 m would be required to accommodate the projected SBR volume (60 veh/h).

Goulbourn Forced Road

- A double eastbound right-turn lane (from eastbound Goulbourn Forced Road to southbound Terry Fox Drive) would be required to accommodate the projected 2013 Saturday afternoon volumes (555 veh/h). From the TAC formula, a storage length of approximately 105 m (assuming a 45%/55% lane distribution per City TIA guidelines) would be required. With respect to the existing signal timing plan, the existing split phase could be maintained or a protected eastbound right-turn phase could be implemented. Signage would have to clearly indicate that double right-turns are not permitted on red lights.
- Also in the eastbound direction, a dedicated left-turn lane and separate through-lane are required on the Goulbourn Forced Road. From TAC, approximately 25 m of storage would be required for the left-turn lane. Thus, with the two westbound lanes and two eastbound double right-turn lanes, a total of six lanes would be required on the west leg of the Intersection to accommodate the projected 2013 Saturday volumes. Assuming minimum lane widths and the addition of a centre median, a minimum pavement width of approximately 23 m would be required on Goulbourn Forced Road.

Based on the Synchro analysis (attached), the geometric modifications and signal timing plan adjustments outlined above are required to ensure that the Goulbourn Forced/Terry Fox Intersection operates at LoS D overall with a 120 seconds cycle-length. Under the projected 2013 weekday and Saturday afternoon conditions, the overall Intersection delay is estimated to be approximately 46 and 55 seconds, respectively. During the PM peak, the northbound double-left turn lane (NBL) and southbound through-lane (SBT) and anticipated to operate at LoS E, with corresponding volume-to-capacity ratios (v/c) of 0.96 and 0.98, respectively. For the Saturday afternoon peak hour, the NBL, SBT and westbound through-lane (WBT) movements are projected to fail, with corresponding v/c ratios of 1.06, 1.02 and 1.05, respectively.

Completion of the Campeau Drive westerly to Intersect Huntmar Drive will provide some relief to the Terry Fox/Goulbourn Forced Road Intersection by providing an alternative, direct route to Highway 417 and community south thereof. The magnitude of the shift in traffic is unknown but it is unlikely that the shift would be sufficient to resolve all the anticipated capacity issues.

Conclusions and Recommendations

The foregoing analysis outlines the roadway and intersection modifications required to accommodate site-generated traffic for the planned developments south and west of Terry Fox Drive, north of Highway 417. The considerable retail development planned for these lands will generate significant traffic volumes, which the adjacent road network cannot absorb as is.

In the case of the Terry Fox/Goulbourn intersection, extensive work would be required to accommodate the planned development without the implementation of the "mini-interchange". Although this report identifies what is required from a transportation perspective, the solution provided may not be feasible in terms of cost and/or land requirements. It is noteworthy, however, that Delcan has not modelled the mini-interchange option and can only assume that it would be a more appropriate solution to the capacity issues at the Terry Fox/Goulbourn Intersection.

With regard to Campeau Drive, west of Terry Fox Drive prior to its westerly extension to Huntmar Drive, the following is recommended:

- A two-lane cross-section (i.e. one eastbound and one westbound lane) from the shared, westerly Loblaws/RONA driveway (i.e. Access #1) easterly to the right-in/right-out Loblaws driveway (i.e. Access #2);
- A three-lane cross-section (two eastbound lanes and one westbound lane) from Access #2 to the Didsbury Road Intersection;
- A four-lane cross-section (two eastbound and two westbound lanes) from Didsbury Road easterly to Terry Fox Drive;
- A protected phase should be provided for the eastbound and westbound left-turn lanes to ensure that these movements have sufficient greentime to process the demand, given the heavy opposing through volumes;
- Signalization of the Campeau/Didsbury Intersection is recommended from site opening (i.e. 2008). On Campeau Drive, opposing left-turn lanes and a single through/right-turn lane is recommended on the eastbound and westbound approaches. A single all-movement lane will be sufficient on the northbound and southbound Didsbury Road approaches.

It is also recommended that Didsbury Road be extended to connect with the existing Terry Fox/Signature Centre Intersection, which is currently outfitted with appropriate signals and turn-lanes.

With respect to the Terry Fox/Goulbourn Forced Road Intersection, the following Interim Improvements are required in order for the intersection to operate at Level of Service D or better during peak hours, based on the projected 2008 volumes and prior to the completion of the Campeau Drive extension:

- A double northbound left-turn lane on Terry Fox Drive with approximately 120 m or storage would be required, which will necessitate the widening the existing median on the north leg of the intersection, and construction of a second westbound receiving lane on Goulbourn Forced Road;
- A southbound right-turn (SBR) lane on Terry Fox Drive;

- A double eastbound right-turn lane with approximately 105 m storage per lane, and a separate signal phase, on the west leg of the Terry Fox/Goulbourn intersection;
- Also on the eastbound leg, provide a dedicated left-turn lane with 25 m storage length, and separate through-lane are required on the Goulbourn Forced Road;

Ed, we trust that the foregoing information is sufficient for the City to process Taggart/Loblaws application. Please do not hesitate to contact me should you require anything further.

Yours truly,



Ronald M. Jack, P.Eng.
Vice President
Manager, Transportation Systems Division



Elizabeth E. Newgard, P.Eng.
Project Engineer

Attachment

c.c. Jeff Parkes, Taggart Realty Management (jparkes@taggart.ca)
Bill Jolliffe, City of Ottawa (William.Jolliffe@ottawa.ca)
Steve Stoddard, City of Ottawa (Steven.Stoddard@ottawa.ca)
Miguel Tremblay, FoTenn (tremblay@fotenn.com)
Kevin Harper, IBI Group (kharper@ibigroup.com)

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Appendix G

Correspondence

Lynch, Amanda

From: Whittaker, Damien <Damien.Whittaker@ottawa.ca>
Sent: Friday, February 03, 2012 10:05 AM
To: Cody, Neal
Subject: RE: Water main boundary condition request Taggart-Loblaws

Neal,

Please find a boundary condition response below.

Max Day + FF = 150.5 m while assuming a fire flow of 250 L/s

Minimum HGL during Peak Hour = 155.8 m

Max Pressure Check HGL = 163.4 m

Regards,

Damien Whittaker, P.Eng.
Project Manager

Development Review Suburban Services Unit
West Branch
110 Laurier Avenue West
613-580-2424 x16968

From: Cody, Neal [<mailto:Neal.Cody@stantec.com>]
Sent: January 26, 2012 1:03 PM
To: Whittaker, Damien
Subject: RE: Water main boundary condition request Taggart-Loblaws

Hi Damien,

Marie-Gen gave me a call to confirm the requested fire flow rate in the email below. I can confirm that the required fire flow is 15,000 LPM as per the FUS calculation method. Please forward this email to Gen for confirmation.

Thanks,
Neal

From: Cody, Neal
Sent: Thursday, January 26, 2012 10:43 AM
To: 'Damien.Whittaker@ottawa.ca'
Cc: Wilkie, Tim
Subject: FW: Water main boundary condition request Taggart-Loblaws

Hi Damien,

Tim asked me to forward you the revised boundary conditions request for the Taggart Loblaws subdivision. These demands are taken from section 1.6 of our Potable Water Servicing Analysis (which formed part of our Servicing Report), dated November 11, 2011:

Table 1.1: Demand projections for proposed Taggart - Loblaws development

Service Area	Land Area or # of Seats	Average Day (L/s)	Maximum Day (L/s)	Peak Hour (L/s)
Commercial	13.17ha	3.81	5.72	10.29
Church	300 seats	0.13	0.19	0.34
Total		3.94	5.91	10.63

Please apply these demands at the end of the 406mm at the corner of Campeau and Didsbury.

The previously-cited fire flow from the City was 15,000 LPM.

Please let me know if you have any questions.

Thanks,
Neal

Neal Cody, P.Eng, LEED Green Assoc.

Water Resources Engineer

Stantec

1505 Laperriere Avenue

Ottawa ON K1Z 7T1

Ph: (613) 724-4380

Fx: (613) 722-2799

neal.cody@stantec.com

stantec.com

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From: Wilkie, Tim
Sent: Monday, January 23, 2012 1:30 PM
To: Cody, Neal
Subject: FW: Water main boundary condition request

Can you please respond?

From: Whittaker, Damien [<mailto:Damien.Whittaker@ottawa.ca>]
Sent: Monday, January 23, 2012 12:11 PM
To: Wilkie, Tim
Subject: Water main boundary condition request

Tim,

The City is unable to confirm the previously provided boundary conditions for the Taggart-Loblaws subdivision of June 18th, 2007. Please re-submit the previously sent request.

Please feel free to ask for clarification, or further information, on any of the items above.

Regards,

Lynch, Amanda

Subject: FW: RE: TAGGART - Kanata West

From: Wilkie, Tim
Sent: Tuesday, October 05, 2010 9:59 AM
To: 'Jeff Parkes'
Subject: RE: TAGGART - Kanata West

Hi Jeff

Larry had a great meeting with the City and Don Moss last week. In general it looks like we now have some direction:

1. The City is amenable to consider a land swap to avoid moving the proposed SWM pond back another 3.8 m. We will need to do volume calculations to demonstrate that the land being exchanged will account for the volume loss by leaving the pond where currently indicated.
2. The design criteria to calculate the stormwater drainage remains pre-development or 170 m³/ha, whichever proves to be the worst case scenario. The entire drainage area for the pond must be used in the calculation. Don Moss and City staff recommended to start with the volume of storage there would be between normal water level and full pond. Anything exceeding what the pond can accommodate could then be made up through on-site measures such as roof and parking lot storage at each of the development projects in the drainage area of the pond.
3. The matter of the outlet sewer invert to the pond being below the Carp River was not deemed to be an issue provided that the 100 year level of the Carp did not impact the sewer. The sewer as it is has an invert below the normal water level of the pond; therefore it is a matter to determine where the 100 level will reach. It was also acknowledged that most of the development within the drainage shed would be slab on grade buildings and that surcharged sewers would have no impact to them. The only exception could be the existing church. It is unknown whether it has a basement.
4. The City and Don Moss indicated that the SWM pond would be deemed an interim one until such time that the monitoring and modeling results for the Carp River had been finalized to the satisfaction of the approval agencies, and the Carp River Rehabilitation Plan had cleared the Part 2 Orders with the MOE. It was acknowledged that it would be highly unlikely that the outcome to the above two conditions would impact your pond.

I suggest that we do an overall review of the SWM design to see if we can make sure the Site can work based on the above criteria.

Jeff the only other item that still needs to be resolved is the Signature Ridge Pump Station issues with capacity.

I will call you to discuss.

Tim Wilkie
Senior Project Manager
Stantec
1505 Laperriere Avenue
Ottawa ON K1Z 7T1
Ph: (613) 724-4081
Fx: (613) 722-2799
tim.wilkie@stantec.com
stantec.com

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Date: June 4, 2012

Tim Wilkie, P.Eng.
Senior Project Manager, Associate
Stantec Consulting Limited
1505 Laperriere Avenue, Ottawa, ON K1Z 7T1

Dear Mr. Wilkie;

**Re: 2nd Engineering Review
Taggart-Loblaws Subdivision**

The following comments are provided in response to the engineering submission for your project number 1604-00402 dated May 2, 2011, with the City of Ottawa. Listed below are preliminary comments:

The following drawings and reports were reviewed for this engineering submission:

Reports:

- *Taggart-Loblaws Subdivision, Kanata West, Servicing Report, File No. 1604-00402, April 27, 2012, Stantec Consulting Limited*
- *Taggart-Loblaws Subdivision, Stormwater Management Facility, Design Report, File No. 1604-00402/83, April 27, 2012, Stantec Consulting Limited*

Drawings:

- *Overall Servicing Plan, Taggart-Loblaws Subdivision, Kanata West, Drawing OSP-1, Revision 5, April 27, 2012, Stantec Consulting Limited*
- *Didsbury Road Grading Plan, Taggart-Loblaws Subdivision, Kanata West, Drawing GP-1, Revision 5, April 27, 2012, Stantec Consulting Limited*
- *Didsbury Road Grading Plan, Taggart-Loblaws Subdivision, Kanata West, Drawing GP-2, Revision 5, April 27, 2012, Stantec Consulting Limited*
- *Street No. 2 and, Temporary Access Road, Grading Plan, Taggart-Loblaws Subdivision, Kanata West, Drawing GP-3, Revision 5, April 27, 2012, Stantec Consulting Limited*
- *Didsbury Road, Plan and Profile Sta. 0+000 to 0+280, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-1, Revision 5, April 27, 2012, Stantec Consulting Limited*
- *Didsbury Road, Plan and Profile Sta. 0+280 to 0+560, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-2, Revision 5, April 27, 2012, Stantec Consulting Limited*
- *Didsbury Road, Plan and Profile, Sta. 0+560 to 0+773.9, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-3, Revision 5, April 27, 2012, Stantec Consulting Limited*

- *Storm Sewer Outlet, Plan and Profile, Sta. -0+260 to Sta. 0+000*, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-4, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Roger Neilson Way, Plan and Profile, Sta. 0+000 to Sta. 0+284*, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-5, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Campeau Drive Extension, Plan and Profile, Sta. 0+000 to Sta. 0+260*, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-6, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Campeau Drive Extension, Plan and Profile, Sta. 0+260 to Sta. 0+458.6*, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-7, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Service Easement, Plan and Profile, Sta. 0+000 to Sta. 0+160*, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-8, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Service Easement, Plan and Profile, Sta. 0+140 to Sta. 0+309.27*, Taggart-Loblaws Subdivision, Kanata West, Drawing PP-9, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Erosion Control, Details and Notes*, Taggart-Loblaws Subdivision, Kanata West, Drawing DS-1, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Details and Notes*, Taggart-Loblaws Subdivision, Kanata West, Drawing DS-2, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Overall Storm Drainage Plan*, Taggart-Loblaws Subdivision, Kanata West, Drawing SWM-1, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Overall Sanitary Drainage Plan*, Taggart-Loblaws Subdivision, Kanata West, SAN-1, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Existing Pond Conditions*, Taggart-Loblaws Subdivision, Kanata West, Drawing EX-1, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Plan and Cross Sections*, Taggart-Loblaws Subdivision, Kanata West, Drawing POND-1, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Inlet and Draw-down, Chamber Details*, Taggart-Loblaws Subdivision, Kanata West, Drawing POND-2, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Outlet Structure Details*, Taggart-Loblaws Subdivision, Kanata West, Drawing POND-3, Revision 5, April 27, 2012, Stantec Consulting Limited
- *Floodplain Compensation Plan*, Taggart-Loblaws Subdivision, Kanata West, Drawing POND-2, April 26, 2012, Stantec Consulting Limited
- *Floodplain Compensation, Cross Sections*, Taggart-Loblaws Subdivision, Kanata West, Drawing POND-2, April 26, 2012, Stantec Consulting Limited

General

The floodplain compensation for the project is integral and requires documentation.

Servicing Report

Section 3 – Stormwater Management

1. Sub-section 3.6 – HGL above grade is not acceptable. Please raise the road grade. The consultant is reminded that overland flow paths, if any exist, must be maintained

Section 4 – Sanitary Sewers/Pumping Station

2. Sub-section 4.4 – HGL at or near grade is not acceptable. Please raise the road grade. As above, the consultant is reminded that overland flow paths must be maintained.

Section 5 - Water Distribution

3. Sub-section 5.2 – Figure 3, reviewed in collaboration with the section does not clearly show existing and proposed water pipes.

Appendix A - Sewer Design Sheets/Calculations

1. There are differences between the sanitary design sheets and plan SAN-1
2. There may be an accumulation error in the storm sewer design sheet; the second 0.203 ha area addition does not correlate with SWM-1.
3. The pipe data presented in the storm sewer design sheet “SWM Pond Drainage Area” between STM104 and STM 103 could not be verified on plan SWM-1.

Stormwater Management Facility Design Report

Please provide a calculation for the sediment storage area

Section 2 - Stormwater Management Design

1. Sub-section 2.4.3.1 refers to minimum and maximum infiltration rates. The rates should be referred to as initial and final.
2. The decay constant presented in sub-section 2.4.3.1 does not appear to agree with 5.4.5.5 of the *Sewer Design Guidelines*, November 2004, revised September 2008, amended January 2012, City of Ottawa (*Guidelines*). Comment is required for deviations from the *Guidelines*. The City is not opposed to deviation, rather deviation without discussion.
3. Section 2.4.6 states that sediment will be included in pipes with standing water, namely those pipes with inverts below the permanent water level, but section 2.5.3 states that the HGL for STM MH 107 is not of concern because its elevation is above the permanent pond elevation. Please clarify the conflicting statements.
4. It is not sufficient that modeling is “roughly equal” to the MVC 1983 flood level. Modelling is required to start at least at that elevation as stated in section 2.7 of the report.
5. Sub-section 2.5.1 details highly unstable model results. It is not sufficient to merely state such a result. The result should be better defined and more modelling work should be performed to remove instabilities.
6. Sub-section 2.6 should determine alternative infiltration techniques rather than closing by stating that is doubtful that infiltration targets will be achieved. The consultant is reminded that site plan developers will look to the consultant’s subdivision documents for direction.
7. Sub-section 2.6.2.1 on page 2.13 appears to be repeated on page 2.14.
8. Sub-section 2.7 includes two ‘Error! Not a valid bookmark self-reference.’ entries.

Section 3 – Detailed Facility Design Components

1. Please correct the spelling of manoeuvrability in sub-section 3.5.

Section 5 – Operations and Maintenance

1. Please state how pond water levels will be monitored, as stated in section 5.1.2.

Appendix E – Pond Design

2. There are no units assigned to the Total MOE Volume columns.
3. Please confirm and qualify the value of 134.4 m³/ha for the Water Quality Volumetric Requirements table in Appendix E.
4. The second reference for the Water Quality Volumetric Requirements directs to Table 3.1 of the Stormwater Management Planning and Design Manual, 2003, Ontario Ministry of the Environment. It is suggested that the reference may be intended for Table 3.2.

5. The frame at the bottom right of the Detailed Outlet Structure Discharge Calculations page, in Appendix E is not legible and the explanation of P_w appears to extend past the printing margins.
6. Appendix E, page 5 of 5 "Sediment Forebay Sizing Calculations" does not confirm that the settling length is satisfactory.
7. The velocity calculation, within Appendix E, page 5 of 5, shows a velocity of 0.14 m/s, compared with a target velocity of 0.15 m/s, but then states that the calculated velocity is higher than preferred.
8. The pond outflow hydrographs are not satisfactory. Please refer to previous instability comments.

Appendix F – Background Reports and References

1. The pond outflow hydrographs are not satisfactory. Please remove the instabilities.
2. The report requires that a comparison is made between the Third-Party hydrographs and the proposed hydrographs- with comment on any discrepancies.

Appendix G – Correspondence

3. The section does not have a divider.
4. Infiltration is a Conservation Authority requirement as per the Kanata West Master Servicing Study. The City can convene a meeting between the two parties in the absence of infiltration techniques.

Drawings

1. A 97.972 proposed grade appears on plan GP-3 right next to a 97.12, on Didsbury, just north of the intersection with Roger Nielson Way.
2. The major system drainage path proposed at the intersection of Didsbury with Roger Nielson Way shown on GP-3 does not agree with that shown on SWM-1. The grading on GP-3 is conducive to the major system flow shown on GP-3, not SWM-1
3. The grade of 96.15 just above the label of Roger Nielson Way, on Plan GP-3 appears to outlet overland flow before flowing to the pond.
4. Please include the sanitary HGL, defined in section 4 of the Servicing Report, on appropriate drawings, for example PP-1.
5. As per OPSS 407, sub-section 407.07.14, clause 6 of the "Storm and Sanitary Sewers" notes on drawing DS-1 should be amended to state that storm manholes shall be benched to obvert.
6. Please correct the spelling of "service" at the end of note 17 of the "Storm and Sanitary Sewers" notes on drawing DS-1.
7. The cross-section of Campeau Drive shown may not agree with the proposed design for works on the west side of Carp River that show a super-elevated road.
8. A number of labels are hidden underneath sub-catchment lines, on drawing SWM-1.
9. Drawing SWM-1 shows the MVCA Regulatory flood leader lines pointing to both a thin dashed line and a thick solid line.
10. Drawing SWM-1 and drawing POND-1 show the pond maintenance road crossing through the sediment storage area.
11. Drawing POND-1 shows the drawdown pipes at the pond bottom elevation. The pipes should be raised from the bottom of the pond.
12. The font size presented on SAN-1 and SWM-1 for the manhole and pipe labels needs to be larger.
13. Please correct spelling of "development" on plan SWM-1.

14. In accordance with the comments included in the Servicing Report, Appendix B, section 1.8.1 please add a note to all grading plans, the servicing plan and the notes on drawing DS-1 regarding the required use of PRVs. It is anticipated that pressures will not change with the future watermain works: it is left to the consultant to decide to include a note on the plans saying same and that PRVs may need adjusting in future.
15. The floodplain compensation plans should be part of the drawing set for the project.
16. A note of inclusion of pressure reducing valves shall be placed on the servicing plan.

Pond Drawings

General

1. Please show flow direction arrows for all structures.
2. A City standard headwall detail and road width is forthcoming

Drawing Pond-2

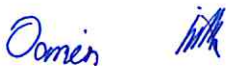
1. Please provide a 2.5 m (wide) x 2.5 m (long) x 0.5 m (deep) "sump" in front of the forebay exit pipe on Section D-D.
2. The cover in Section D-D should be flush with finished level.
3. A 1 m wide elevation is required at the same level as the cover is required in Section D-D. The contours in the main frame on the sheet may need amending.
4. An additional clay seal is required in each section of sewer in Section D-D
5. Rodent grills are required at the pond connections in Section D-D.
6. Please specify the stem for the sluice gate to be "non-rising."
7. The details of the drawdown chamber do not align.
8. Please provide a keyhole in the drawdown chamber. Keyholes should be flush and should be either cored or set.
9. Please specify the sluice gate material as stainless.
10. Please provide an invert and a N.W.L. elevation on the Inlet Detail.
11. Please provide rip rap, at an appropriate size, at the entry to the forebay.

Drawing Pond-3

1. Please remove the access ladder detail as it not used.
2. Please remove the stop log lifting beam and cross-section as they are not used.
3. Please include railings in Section A-A of the outlet structure sheet frame.
4. The orifice valve box should be cored, or set, and be mounted flush.

Please provide a response showing how each of the comments is addressed in the resubmission. Any revisions or addenda to any studies must be accompanied by a *.pdf copy of the report (either by CD or e-mail). Should you wish to contact me regarding any questions or a follow up meeting you may reach me at 580-2424 ext. 16968.

Sincerely,



Damien Whittaker, P.Eng.

Project Manager, Infrastructure Approvals

Planning and Growth Management Department

cc. *Sean Moore, Planner, Development Review*



Stantec Consulting Ltd.
1505 Laperriere Avenue
Ottawa ON K1Z 7T1
Tel: (613) 722-4420
Fax: (613) 722-2799

Stantec

August 14, 2012
File: 160400402-10

City of Ottawa
Infrastructure Approvals Division
110 Laurier Ave, 4th floor
Ottawa, Ontario K1P1J1

Attention: Damien Whittaker, P.Eng
Project Manager, Infrastructure Approvals, Development Review (Suburban Services) Planning and Growth Management Department

Dear: Mr. Whittaker

Reference: Taggart/Loblaws Subdivision - Response to City Comment: June 4, 2012

The following summarizes Stantec's response to e-mail comments received from the City of Ottawa (dated June 27th, 2012) on the current submission of the Taggart/Loblaws Subdivision drawings package dated June 2012 and submitted in support of Arcadia Phase 1 MOE Submission for Approval:

General

1. The floodplain compensation for the project is integral and requires documentation.

Response: A discussion of the floodplain compensation requirements and rationale is included in Section 5.0 of the SWM report and Section 3.7 of the Servicing Report. Calculations are included on Drawings DET-1 to DET-3.

Servicing Report

1. Sub-Section 3.6 – HGL above grade is not acceptable. Please raise the road grade. The consultant is reminded that overland flow paths, if any exist, must be maintained.

Response: Road grade along Didsbury Road has been raised to show HGL below proposed road grade. Refer to Drawing GP-1 .

2. Subsection 4.4 – HGL at or near grade is not acceptable. Please raise the road grade. As above, the consultant is reminded that overland flow paths must be maintained.

Response: The road grade has been raised but please note that the HGLs and road grades presented in Servicing Report Table 4-3 are from an excerpt from the "Signature Ridge Pump

August 14, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 2 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Station Hydraulic Grade Line Analysis” prepared by IBI Group, July 2011, the proposed road grades have been raised.

3. Sub-section 5.2 – Figure 3, reviewed in collaboration with the section does not clearly show existing and proposed water pipes.

Response: Figure 3 has been re-printed in colour to better show the distinction between existing and proposed water pipes.

Appendix A – Sewer Design Sheets/Calculations

1. There are differences between the sanitary design sheets and plan SAN-1.

Response: Sanitary design sheets have been revised to match plan SAN-1.

2. There may be an accumulation error in the storm sewer design sheet; the second 0.203 ha area addition does not correlate with SWM-1.

Response: The second 0.203 was a typographical error and has been removed.

3. The pipe data presented in the storm sewer design sheet “SWM Pond Drainage Area” between STM104 and STM 103 could not be verified on plan SWM-1.

Response: STM pipe labels have been moved so as to not overlap with Drainage Area boundaries on SWM-1. See Drawing SWM-1.

Stormwater Management Facility Design Report

1. Please provide a calculation for the sediment storage area.

Response: Discussion of sediment storage calculations is included in SWM Report Section 3.5. Sediment volume calculations are included in the pond design calculations in Appendix E.

Section 2 – Stormwater Management Design

1. Sub-section 2.4.3.1 refers to minimum and maximum infiltration rates. The rates should be referred to as initial and final.

Response: Section 2.4.3.1 has been revised.

2. The decay constant presented in sub-section 2.4.3.1 does not appear to agree with 5.4.5.5 of the Sewer Design Guidelines. Comment is required for deviations from the Guidelines. The City is not opposed to deviation, rather deviation without discussion.

August 14, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 3 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Response: The units have been added to the decay constant, it is noted that the different value was simply due to a unit conversion from seconds to hours.

3. Section 2.3.6 states that sediment will be included in pipes with standing water, namely those pipes with inverts below the permanent water level, but section 2.5.3 states that the HGL for STM MH 107 is not of concern because its elevation is above the permanent pond elevation. Please clarify the conflicting statements.

Response: The initial model was built such that the "sedimented" scenario showed sediment in all pipes rather than only those with inverts below the permanent pool. The present models for this submission have been revised such that only pipes below (or with sections of pipe below) the permanent pool have were modeled with sediment.

4. It is not sufficient that modeling is "roughly equal" to the MVC 1983 flood level. Modelling is required to start at least at that elevation as stated in section 2.7 of the report.

Response: Dynamic boundary conditions obtained from the Carp River modeling have peak elevations higher than the MVC 1983 flood level. Additionally, use of the dynamic boundary conditions allowed for the adjustment of the time of peak based on the duration of the storm event.

5. Sub-section 2.5.1 details highly unstable model results. It is not sufficient to merely state such a result. The result should be better defined and more modeling work should be performed to remove instabilities.

Response: Modeling has been revised to improve model results. See revised section 2.5.1 and Section 2.4.6.

6. Sub-section 2.6 should determine alternative infiltration techniques rather than closing by stating that it is doubtful that infiltration targets will be achieved. The consultant is reminded that site plan developers will look to the consultant's subdivision documents for direction.

Response: Sub-section 2.6 has been revised to include discussion of the infiltration requirements for the pond block area as well as provide general direction on infiltration approaches for future development areas. Example calculation spreadsheets for water balance calculations and infiltration basin design are included on the CD attached with the report.

7. Sub-section 2.6.2.1 on page 2.13 appears to be repeated on page 2.14.

Response: The repeated text has been removed.

8. Sub-section 2.7 includes two 'Error! Not a valid bookmark self-reference.' Entries

August 14, 2012

Damien Whittaker, P.Eng
Project Manager, Infrastructure Approvals, Development Review (Suburban Services)
Page 4 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Response: The error has been removed and replaced with the proper table reference.

Section 3 – Detailed Facility Design Components

1. Please correct the spelling of Manoeuvrability in sub-section 3.5.

Response: The spelling of Manoeuvrability has been revised.

Section 5 – Operations and Maintenance

1. Please state how pond water levels will be monitored, as stated in section 5.1.2.

Response: A discussion has been added noting that water levels will be monitored using a continuous recording level logger installed within the outlet structure.

Appendix E – Pond Design

2. There are no units assigned to the Total MOE Volume columns.

Response: Units have been added to the Total MOE volume.

3. Please confirm and qualify the value of 134.4m³/ha for the Water Quality Volumetric Requirements table in Appendix E.

Response: The value of 134.4m³/ha is was obtained from interpolation based on the average runoff coefficient for the site and the 70% removal requirement.

4. The second reference for the Water Quality Volumetric Requirements directs to Table 3.1 of the Stormwater Management Planning and Design Manual, 2003, Ontario Ministry of the Environment. It is suggested that the reference may be intended for Table 3.2.

Response: The reference has been revised to reference Table 3.2.

5. The frame at the bottom right of the Detailed Outlet Structure Discharge Calculations page, in Appendix E is not legible and the explanation of Pw appears to extend past the printing margins.

Response: The print margin has been extended so that the full explanation of Pw is legible.

6. Appendix E, page 5 of 5 “Sediment Forebay Sizing Calculations” does not confirm that the settling length is satisfactory.

Response: Both the required forebay length and actual length are indicated in the calculation sheet. A comment has been added stating that the actual length is satisfactory.

August 14, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 5 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

7. The velocity calculation, within Appendix E, page 5 of 5, shows a velocity of 0.14m/s, compared with a target velocity of 0.15m/s, but then states that the calculated velocity is higher than preferred.

Response: The statement has been revised to confirm that the velocity meets the target.

8. The pond outflow hydrographs are not satisfactory. Please refer to previous instability comments.

Response: Revised pond outflow hydrographs have been included.

Appendix F – Background Reports and References

1. The pond outflow hydrographs are not satisfactory. Please remove the instabilities.

Response: The hydrographs included in this appendix are from third party background reports and modeling and are not from our present models.

2. The report requires that a comparison is made between the Third-Party hydrographs and the proposed hydrographs – with comment on any discrepancies.

Response: Section 2.6.2 compares the third party target hydrographs for the SWMF and the total site area to the modeled post-development hydrographs.

Appendix G- Correspondence

3. The section does not have a divider.

Response: A divider has been added to this section.

4. Infiltration is a Conservation Authority requirement as per the Kanata West Master Servicing Study. The City can convene a meeting between the two parties in the absence of infiltration techniques.

Response: Stantec has conversed with the MVC regarding infiltration requirements and has provided additional infiltration calculations for the proposed SWMF as well as example water balance calculations and infiltration basin calculations for use in the detailed design of the future development areas. See report **Section 2.7**.

August 14, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 6 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Drawings

1. A 97.972 proposed grade appears on plan GP-3 right next to a 97.12, on Didsbury, just north of the intersection with Roger Nielson Way.

Response: Grading has been revised, see DWG GP-3.

2. The major system drainage path proposed at the intersection of Didsbury with Roger Nielson Way shown on GP-3 is conducive to the major system flow shown on GP-3 not SWM-1.

Response: Grading has been revised, see DWG GP-3 and PP-3.

3. The grade of 96.15 just above the label of Roger Nielson Way, on Plan GP-3 appears to outlet overland flow before flowing to the pond.

Response: The overland flow elevation will be the high point (gutter) at 96.12.

4. Please include the sanitary HGL defined in section 4 of the Servicing Report, on appropriate drawings, for example PP-1.

Response: Sanitary HGLs have been included on the appropriate plan and profile drawings.

5. As per OPSS 407, subsection 407.07.14, clause 6 of the "Storm and Sanitary Sewers" notes on drawing DS-1 should be amended to state that storm manholes shall be benched to overt.

Response: Notes have been revised, see DWG DS-1, however benching should be installed as per OPSD 701.021.

6. Please correct the spelling of "service" at the end of note 17 of the "Storm and Sanitary Sewers" notes on drawing DS-1.

Response: Notes have been revised, see DWG DS-1

7. The cross-section of Campeau Drive shown may not agree with the proposed design for works on the west side of Carp River that show a super-elevated road.

Response: Cross-section has been revised as per latest available details from Stantec Transportation.

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Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 7 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

8. A number of labels are hidden underneath sub-catchment lines, on drawing SWM-1.

Response: Labels have been moved to eliminate overlap, see DWG SWM-1.

9. Drawing SWM-1 shows the MVC regulatory flood leader pointing to both a thin dashed line and a thick solid line.

Response: Leaders have been revised to point to the correct lines, see DWG SWM-1

10. Drawing SWM-1 and drawing POND-1 show the pond maintenance road crossing through the sediment storage area.

Response: The pond maintenance road has been moved.

11. Drawing POND-1 shows the drawdown pipes at the pond bottom elevation. The pipes should be raised from the bottom of the pond.

Response: The drawdown pipes have been raised, see DWG POND-1.

12. The font size presented on SAN-1 and SWM-1 for the manhole and pipe labels needs to be larger.

Response: The font size has been increased, see DWGs SAN-1 and SWM-1.

13. Please correct the spelling of "development" on plan SWM-1.

Response: Spelling has been corrected on all plans.

14. In accordance with the comments included in the Servicing Report. Appendix B, section 1.8.1 please add a note to all grading plans, the servicing plan and the notes on drawing DS-1 regarding the required used of PRVs. It is anticipated that pressures will not change with the future watermain works: it is left to the consultant to decide to include a note on the plans saying same and that PRVs may need adjusting in the future.

Response: Notes have been added to all the required drawings.

15. The floodplain compensation plans should be part of the drawing set for the project.

Response: Floodplain compensation drawings have been included as part of the drawing set.

16. A note of inclusion of pressure reducing valves shall be placed on the servicing plan.

August 14, 2012

Damien Whittaker, P.Eng
Project Manager, Infrastructure Approvals, Development Review (Suburban Services)
Page 8 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Response: As per comment 14 above, a note has been added to the servicing plan.

Pond Drawings

General

1. Please show flow direction arrows for all structures.

Response: Flow direction arrows have been added to all structure details, see POND-2.

2. A City standard headwall detail and road width is forthcoming.

Response: No city standard detail is available yet and so MOE, OPSD standards have been used.

Drawing Pond-2

1. Please provide a 2.5 m (wide) x 2.5 m (long) x 0.5 m (deep) "sump" in front of the forebay exit pipe on Section D-D.

Response: A sump has been provided as requested.

2. The cover in Section D-D should be flush with finished level.

Response: Cover in Section D-D has been revised.

3. A 1.0 m wide elevation is required at the same level as the cover is required in Section D-D. The contours in the main frame on the sheet may need amending.

Response: A 1.0 m wide elevation has been included.

4. An additional clay seal is required in each section of sewer in Section D-D.

Response: Additional clay seals have been included in Section D-D.

5. Rodent grills are required at the pond connections in Section D-D.

Response: Rodent grills have been added to the pond connections in Section D-D.

6. Please specify the stem for the sluice gate to be "non-rising".

Response: Sluice gate stem has been specified as non-rising.

August 14, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 9 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

7. The details of the drawdown chamber do not align.

Response: Details have been revised.

8. Please provide a keyhole in the drawdown chamber. Keyholes should be flush and should be either cored or set.

Response: Keyholes have been provided.

9. Please specify the sluice gate material as stainless.

Response: Sluice gate material has been specified as stainless.

10. Please provide NWL elevation on the Inlet Detail.

Response: NWL has been provided on Inlet Detail.

11. Please provide rip rap, at an appropriate size, at the entry to the forebay.

Response: Rip-rap has been provided.

Drawing Pond-3

1. Please remove the access ladder detail as it is not used.

Response: Access ladder detail has been removed

2. Please remove the stop log lifting beam and cross-section as they are not used.

Response: Stop log lifting beam and cross-section have been removed.

3. Please include railings in Section A-A of the outlet structure sheet frame.

Response: Railings have been included in Section A-A.

4. The orifice valve box should be cored, or set, and be mounded flush.

Response: orifice valve box has been revised.

August 14, 2012

Damien Whittaker, P.Eng
Project Manager, Infrastructure Approvals, Development Review (Suburban Services)
Page 10 of 10

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Sincerely,

STANTEC CONSULTING LTD

Tim Wilkie
Senior Project Manager
Tel: (613) 722-4420
Fax: (613) 722-2799
Tim.wilkie@stantec.com

stantec.com

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Fiszman, Eytan

From: Cody, Neal
Sent: Tuesday, November 06, 2012 5:49 PM
To: Fiszman, Eytan
Subject: FW: Taggart - Kanata West

FYI

From: Whittaker, Damien [<mailto:Damien.Whittaker@ottawa.ca>]
Sent: Monday, November 05, 2012 11:09 AM
To: Wilkie, Tim
Cc: Lynch, Amanda; Cody, Neal
Subject: RE: Taggart - Kanata West

Tim,

Sorry for the late response- I have been off sick. Please make the minor revisions to the SWM report and then the re-submission can be forwarded to the third-party review.

Regards,

Damien Whittaker, P.Eng.
Project Manager

Development Review, Suburban Services - West
110 Laurier Avenue West, 4th Floor
613-580-2424 x16968
damien.whittaker@ottawa.ca

From: Wilkie, Tim [<mailto:tim.wilkie@stantec.com>]
Sent: October 29, 2012 3:00 PM
To: Whittaker, Damien
Cc: Lynch, Amanda; Cody, Neal
Subject: RE: Taggart - Kanata West

Hi Damien:

We have now coordinated with Aecom concerning the CRRP and Pond conflicts. (Please see attached Email). I hope we can now move forward with the 3 third partly review process. Please let me know ASAP and we will make the minor revisions to the SWM report as requested and resubmit.

Thanks

Tim Wilkie
Senior Project Manager
Stantec
1505 Laperriere Avenue
Ottawa ON K1Z 7T1
Ph: (613) 724-4081
tim.wilkie@stantec.com
stantec.com

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Fiszman, Eytan

From: Wolter, Wolfgang <Wolfgang.Wolter@aecom.com>
Sent: Friday, October 26, 2012 4:36 PM
To: Jeff Parkes; Wilkie, Tim
Cc: GCormier@JLRICHARDS.CA; k.willis@rogers.com; Gillis, Sheridan; Lynch, Amanda
Subject: RE: Taggart Kanata - West

Follow Up Flag: Follow up
Flag Status: Flagged

Hello All

With respect to the Taggart Pond, please note the following comments:

- The Taggart Pond has been identified as being in potential conflict with the Carp River proposed plan
- AECOM does not anticipate that adjusting the plan form of the Carp and the adjacent habitat pond would be an issue
- Our concerns are related to ensuring that the cross sectional end area in the location of the pond does not adversely affect conveyance or flood storage
- Email correspondence from both the City of Ottawa and staff at Stantec suggest an equivalent storage requirement is being provided in Taggart pond design drawings

It is our understanding that KWOG, the City of Ottawa and Taggart Realty Management are in agreement for AECOM to make the design adjustment to accommodate the conflict on the Carp River design drawings, and accordingly AECOM agrees to make the change. The change will affect the location of habitat pond 5 and a horizontal translation of the position of approximately three meander bends of the Carp. The appropriate cross sections would be updated.

If you have any questions, please call

AECOM

Wolfgang Wolter
Water Resources Market Sector Manager, Canada East
50 Sportsworld Crossing Road, Unit 290
Kitchener, ON Canada
N2P 0A4

From: Jeff Parkes [<mailto:jparkes@taggart.ca>]
Sent: Thursday, October 25, 2012 12:09 PM
To: 'Wilkie, Tim'; Wolter, Wolfgang
Cc: GCormier@JLRICHARDS.CA; k.willis@rogers.com; 'Gillis, Sheridan'; 'Lynch, Amanda'
Subject: RE: Taggart Kanata - West

Thanks, yes, this was the agreed approach. We are really in a bind here so I'm hoping this can be dealt with quickly.

Jeff
Jeff Parkes
Director of Leasing & Development



225 Metcalfe Street, Suite 708
Ottawa, ON K2P 1P9

T: (613) 234-7000 Ext. 235

F: (613) 526-7947

www.taggart.ca

From: Wilkie, Tim [<mailto:tim.wilkie@stantec.com>]

Sent: October-25-12 9:59 AM

To: wolfgang.wolter@aecom.com

Cc: Parkes Jeff; GCormier@JLRICHARDS.CA; k.willis@rogers.com; Gillis, Sheridan; Lynch, Amanda

Subject: RE: Taggart Kanata - West

Hi Wolfgang:

Further to my long voice message this week concerning the coordination of the Taggart Pond and the CRRP, the City is requesting something in righting from you before they will move forward with the 3 Party review process . At this point we are on hold! I am hoping you can help us move forward.

In meeting with the representatives from KWOG (Kathleen Willis and Guy Cormier) they have no issues with us coordinating this with you as along as Taggart pays for any revisions. I understand that Amanda from our office have sent you your drawings.

Let me if you need anything from us, KWOG or Taggart.

Thanks

Tim Wilkie

Senior Project Manager

Stantec

1505 Laperriere Avenue

Ottawa ON K1Z 7T1

Ph: (613) 724-4081

tim.wilkie@stantec.com

stantec.com

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Fiszman, Eytan

From: Cody, Neal
Sent: Tuesday, November 06, 2012 3:52 PM
To: Fiszman, Eytan
Subject: FW: Taggart-Loblaws subdivision review comments

FYI

From: Wilkie, Tim
Sent: Tuesday, September 18, 2012 2:23 PM
To: Cody, Neal; Lynch, Amanda
Subject: FW: Taggart-Loblaws subdivision review comments

Can you please review.

From: Whittaker, Damien [<mailto:Damien.Whittaker@ottawa.ca>]
Sent: Tuesday, September 18, 2012 4:00 PM
To: Wilkie, Tim
Subject: Taggart-Loblaws subdivision review comments

Tim,

Please find comments in preparation of the third-party review below.

Please define the trigger/s and the definition of Interim as it relates to the pond.

Please explain the trigger/s and the definition of Final as it relates to the pond.

Please provide detail and drawings showing how the pond will match into the CRRP for the interim and final conditions.

Please find comments related to the general review of the application below.

Servicing Report

Section 5 - Water Distribution

1. Sub-section 5.2 – Figure 3, Please provide different dash linetypes so that if the page is photocopied it will still be clear which are existing and which are proposed watermains.

Stormwater Management Facility Design Report

Please provide a calculation for the sediment storage area

Section 2 - Stormwater Management Design

1. The minimum and maximum infiltration rates presented in Sub-section 2.4.3.1 are the same as section 5.4.5.5 of the Sewer Design Guidelines, First Edition, Document SDG001, November 2004, City of Ottawa (Guidelines), including Section 8, revised September 19, 2008, Technical Bulletin ISD-2010-1, September 28, 2010, and Technical Bulletin ISDTB-2012-4, June 20, 2012. The numbers in the Guidelines are actually reversed.
2. Section 2.6.2 includes a hanging line.
3. The spelling of manoeuvrability in section 3.5 is correct in the loose comments provided, but not in the report.

Appendix E – Pond Design

1. The second reference for the Water Quality Volumetric Requirements directs to Table 3.1 of the Stormwater Management Planning and Design Manual, 2003, Ontario Ministry of the Environment. It is suggested that the reference may be intended for Table 3.2.
2. The frame at the bottom right of the Detailed Outlet Structure Discharge Calculations page, in Appendix E is not legible.
3. Please correct spelling of “ensure” in note 2 of section 3 of Appendix E.3 – Infiltration Calculations.

Drawings

1. There is a leader line without note at the interface between subcatchments TRANSITWAY 1 AND TRANSITWAY 2 on drawing SWM-1.
2. Please confirm that the road cross-section without super-elevation, shown on drawing DS-2 is correct.

Pond Drawings

Drawing Pond-3

1. Road width is not constant across drawing Pond-1 and Pond-2.
2. Please change reference of Holliday to Halliday on Pond-3.

Please feel free to ask for clarification, or further information, on any of the comments above.

Regards,

Damien Whittaker, P.Eng.
Project Manager

City of Ottawa

Development Review, Suburban Services - West
110 Laurier Avenue West, 4th Floor
613-580-2424 x16968
damien.whittaker@ottawa.ca

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Stantec

Stantec Consulting Ltd.
1505 Laperriere Avenue
Ottawa ON K1Z 7T1
Tel: (613) 722-4420
Fax: (613) 722-2799

November 06, 2012
File: 160400402-10

City of Ottawa
Infrastructure Approvals Division
110 Laurier Ave, 4th floor
Ottawa, Ontario K1P1J1

Attention: Damien Whittaker, P.Eng
Project Manager, Infrastructure Approvals, Development Review (Suburban Services) Planning and Growth Management Department

Dear: Mr. Whittaker

Reference: Taggart/Loblaws Subdivision - Response to City Comment: September 18, 2012

The following summarizes Stantec's response to e-mail comments received from the City of Ottawa (dated September 18, 2012) on the current submission of the Taggart/Loblaws Subdivision report packages dated November 06, 2012: "**Taggart-Loblaws Subdivision, Kanata West Servicing Report**" and the "**Taggart – Loblaws Subdivision Stormwater Management Facility Design Report**".

General

Please define the trigger/s and the definition of Interim as it relates to the pond.

Please explain the trigger/s and the definition of Final as it relates to the pond.

*R/ "Interim" refers to **after** the pond is built and **before** the CRRP is built.*

*"Final" refers to **after** the pond is built and **after** the CRRP is built.*

Please provide detail and drawings showing how the pond will match into the CRRP for the interim and final conditions.

R/ We have corresponded with AECOM, please see the correspondence file. AECOM has agreed to accommodate our current design in their plans, since this project will be constructed before the CRRP. See the servicing Appendix p267-270 and the stormwater management Appendix p284-287.

Servicing Report

Section 5 - Water Distribution

1. Sub-section 5.2 – Figure 3, Please provide different dash linetypes so that if the page is photocopied it will still be clear which are existing and which are proposed watermain.

November 06, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 2 of 4

Reference: Taggart/Loblaws Subdivision - Response to City Comment

R/ The existing and proposed watermain line colour was addressed by making each line a different line type. The existing watermain is now a dashed line, and proposed watermain is now a dashed-dotted line.

Stormwater Management Facility Design Report

Please provide a calculation for the sediment storage area.

R/ Please refer to page 3.7, section 3.5, point 2 in the Stormwater Management Facility Design Report for further details (volume/depth=772m³/1m=772m²).

Section 2 - Stormwater Management Design

1. The minimum and maximum infiltration rates presented in Sub-section 2.4.3.1 are the same as section 5.4.5.5 of the Sewer Design Guidelines, First Edition, Document SDG001, November 2004, City of Ottawa (Guidelines), including Section 8, revised September 19, 2008, Technical Bulletin ISD-2010-1, September 28, 2010, and Technical Bulletin ISDTB-2012-4, June 20, 2012. The numbers in the Guidelines are actually reversed.

R/ The typo in the Sewer Guidelines was copied into this report, however the models always used the correct values. The typo has been addressed as follows:

Final Infiltration Rate = 13.2 mm/hr

Initial Infiltration Rate = 76.2 mm/hr

2. Section 2.6.2 includes a hanging line.

R/ Typo in the report. This has been addressed; the text from the hanging line now joins to the paragraph.

3. The spelling of manoeuvrability in section 3.5 is correct in the loose comments provided, but not in the report.

R/ Typo in the report. Spelling of "Manoeuverability" has been corrected to "manoeuvrability"; the letter "e" has been removed.

Appendix E – Pond Design

1. The second reference for the Water Quality Volumetric Requirements directs to Table 3.1 of the Stormwater Management Planning and Design Manual, 2003, Ontario Ministry of the Environment. It is suggested that the reference may be intended for Table 3.2.

R/ Typo in the report. This has been addressed by correcting "Table 3.1" to "Table 3.2" and "between 70% and 80%" was changed to "between 70% and 85%". See the stormwater management Appendix p180.

2. The frame at the bottom right of the Detailed Outlet Structure Discharge Calculations page, in Appendix E is not legible.

November 06, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 3 of 4

Reference: Taggart/Loblaws Subdivision - Response to City Comment

R/ Graphical error. This has been addressed by reformatting the P_w frame to be legible. See the stormwater management Appendix p182.

3. Please correct spelling of “ensure” in note 2 of section 3 of Appendix E.3 – Infiltration Calculations.

R/ Typo in appendix. The spelling mistake has been addressed, “enusre” was changed to “ensure”. See the stormwater management Appendix p196.

Drawings

1. There is a leader line without note at the interface between subcatchments TRANSITWAY 1 AND TRANSITWAY 2 on drawing SWM-1.

R/ Unnecessary leader line mistakenly placed in SWM-1 (1:1250 scale) drawing). This has been address by removing the leader line.

2. Please confirm that the road cross-section without super-elevation, shown on drawing DS-2 is correct.

R/ The road cross-section on drawing DS-2 been confirmed with the transportation group. The road cross-section provided is specific to station 2+420.

Pond Drawings

Drawing Pond-3

1. Road width is not constant across drawing Pond-1 and Pond-2.

R/ The road width in the Pond-3 drawing - “Asphalt Access Road” detail was incorrectly labelled as 3.5m. The label has been corrected to be 3.0m; 3.0m is consistent with the asphalt width in the Pond-1 and Pond-2 drawings.

2. Please change reference of Holliday to Halliday on Pond-3.

R/ Typo in the Pond-3 drawing. The spelling mistake has been addressed; “Holliday” has been changed to “Halliday”.

November 06, 2012

Damien Whittaker, P.Eng

Project Manager, Infrastructure Approvals, Development Review (Suburban Services)

Page 4 of 4

Reference: Taggart/Loblaws Subdivision - Response to City Comment

Sincerely,

STANTEC CONSULTING LTD

Tim Wilkie

Senior Project Manager

Tel: (613) 722-4420

Fax: (613) 722-2799

Tim.wilkie@stantec.com

stantec.com

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Mississippi Valley Conservation

File: W12/62

December 10, 2012

Taggart Commercial Development Ltd.
225 Metcalfe Street, Suite 708
Ottawa, ON K2P 1P9

c/o Tim Wilkie
Stantec Consulting
1505 Laperriere Avenue
Ottawa, ON K1Z 7T1

**RE: Application Made Under Ontario Regulation 153/06 to Remove Fill within the Regulation
Limit of the Carp River, Lot 2 and 3, Concession 2, City of Ottawa (Kanata)**

Attached please find the Mississippi Valley Conservation permit for site works at the above noted location. Please sign and return a copy of Page 3 to indicate that you have read and accept the conditions of the permit, and acknowledge that the details of the proposal as described in the permit are accurate and complete. This permit comes into effect immediately and expires on December 10, 2014.

Pursuant to Section 28 of the *Conservation Authorities Act* (R.S.O. 1990, as amended), any or all the conditions of the Permit may be appealed to the Executive Committee of the Conservation Authority.

I trust this is clear. If you have any questions or concerns, please contact the undersigned.

Sincerely,

Myra Lavoie, EIT
Water Resources Engineering Intern
613-259-2421 ext. 259



Conservation
ONTARIO
Natural Champions

4175 Hwy. 511, Lanark, Ontario K0G 1K0 • Tel. (613) 259-2421 • Fax (613) 259-3468 • info@mvc.on.ca



MISSISSIPPI VALLEY CONSERVATION AUTHORITY

4175 Hwy 511, R.R. #2 Lanark, Ontario. K0G 1K0 Tel (613) 259-2421, Fax (613) 259-3468

PERMIT

DEVELOPMENT, INTERFERENCE WITH WETLANDS AND ALTERATIONS TO SHORELINES AND WATERCOURSES ONTARIO REGULATION 153/06

File: W12/62

December 10, 2012

Taggart Commercial Development Ltd.
225 Metcalfe Street, Suite 708
Ottawa, ON K2P 1P9

c/o Tim Wilkie
Stantec Consulting
1505 Laperriere Avenue
Ottawa, ON K1Z 7T1

**RE: Application Made Under Ontario Regulation 153/06 to Remove Fill within the Regulation Limit
 of the Carp River, Lot 2 and 3, Concession 2, City of Ottawa (Kanata)**

Mississippi Valley Conservation (MVC) is in receipt of an application to remove fill within the regulation limit of the Carp River and to remove fill from a small area within the 100 year floodplain of the Carp River. As an alternative to widening the Carp River corridor limit by 3.8 meters across the site, an equivalent amount of flood storage volume will be gained by taking cut at three locations on the site. According to Drawings 160400402-Det-1, 160400402-Det-2, and 160400402-Det-3, a total cut of approximately 317 cubic meters will be taken to account for the approximately 259.42 cubic meters of storage that would be gained by the 3.8 meter corridor widening.

For any application, MVC is required to assess an application based on the *Conservation Authorities Act*, the Regulations in force under that Act, and our Board-approved policies under those Regulations. These requirements apply for any project that involves:

- Straightening, changing, diverting, or interfering in any way with the existing channel of a watercourse with an upstream area greater than 125 ha.
- Interfering in any way with a Provincially Significant Wetland.
- Development within the regulated area that may affect the control of flooding, erosion, pollution, or the conservation of land.

This proposal is considered development within the regulated area that may affect the control of flooding, erosion, pollution, or the conservation of land. MVC has reviewed the cut volumes and a balance exists over a

PERMIT
DEVELOPMENT, INTERFERENCE WITH WETLANDS AND ALTERATIONS TO
SHORELINES AND WATERCOURSES
ONTARIO REGULATION 153/06

0.20 meter interval. MVC has found that the work proposed will not negatively impact the conveyance or flood storage within the Carp River. The sediment and erosion control plan for the proposal consists of silt fence installed along the bottom of the fill slope. MVC is satisfied that all issues regarding flooding, erosion, pollution and the conservation of land have been addressed. This has also been reviewed within the context of the Carp River Restoration Plan.

The Mississippi Valley Conservation Authority hereby grants you permission to undertake the proposed works as in application W12/62, as shown on Schedule "A" (2 sheets and Drawings 160400402-Det-1, 160400402-Det-2, and 160400402-Det-3) attached. Work according to this permit may commence immediately, to be completed by December 10, 2014, subject to the following conditions:

1. The permit holder shall maintain a copy of this permit on site and shall ensure that all individuals involved in carrying out the work are aware of the permit and its conditions.
2. All construction shall conform to Schedule "A" (2 sheets and Drawings 160400402-Det-1, 160400402-Det-2, and 160400402-Det-3), submitted with this application.
3. The contractor shall provide sediment and erosion control plans to MVC for approval prior to commencing work.
4. The sediment control measures shall be in place prior to commencing work and shall remain in place until all exposed areas are stabilized.
5. It will be the proponent's responsibility to ensure that the sedimentation control measures are inspected daily and that any maintenance, including the removal of accumulated sediment, is carried out as required.
6. All work shall be carried out in such a manner that will prevent the release of construction and excavation wastes, overburden soil, and any other substances deleterious to aquatic life into the Carp River.
7. Confirmation that proposed works have been completed as approved is required through submission of an as built survey from a qualified professional to MVC for review. A deposit of \$1,500.00 to MVC is required and will be returned once confirmation of final grades, volumes, and limits are provided demonstrating that there has been no loss of floodplain storage.
8. All disturbed fill areas shall be stabilized with a suitable ground cover. Until vegetation has matured sufficiently to stabilize the exposed soil surfaces, additional measures, such as the placement of anchored erosion blankets, will be required and are to be discussed with MVC.
9. All excavated material must be removed from site; no stockpiling within the regulation limit or the floodplain of the Carp River will be permitted.

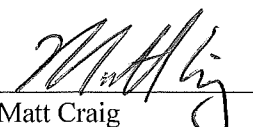
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DEVELOPMENT, INTERFERENCE WITH WETLANDS AND ALTERATIONS TO
SHORELINES AND WATERCOURSES
ONTARIO REGULATION 153/06

Any deviation from the approved criteria without prior written approval from the Mississippi Valley Conservation Authority will constitute a violation of the approved permit. This could result in the permit being revoked in accordance with Section 8 of Ontario Regulation 153/06, which states:

8 (1) The Authority may cancel a permission if it is of the opinion that the conditions of the permission have not been met.

By this letter the Mississippi Valley Conservation Authority assumes no responsibility or liability for any flood, erosion, or slope failure damage which may occur either to your property or the structures on it or if any activity undertaken by you adversely affects the property or interests of adjacent landowners. This letter does not relieve you of the necessity or responsibility for obtaining approval from the municipality or for acquiring any other federal or provincial permits (including, but not limited to, Ontario Planning Act zoning, site plan approvals and/or Building Code Act permits) or compliance with the Endangered Species Act.

NOTE: The information on this form is being collected for the purpose of administering a regulation made pursuant to Section 28, Conservation Authorities Act, R.S.O. 1990, Chapter 27. This application and supporting documents and any other documentation received relating to this application, may be released, in whole or in part, to other persons in accordance with the Municipal Freedom of Information and Protection of Privacy Act, R.S.O. 1990c. M.56, as amended.


Matt Craig
Manager of Planning and Regulatory Services

Dated at Lanark, Ontario, this 10 day of December, 2012.

By signing this permit and returning to Mississippi Valley Conservation Authority in the provided stamped, self-addressed envelope, I, as the owner or agent of the owner, agree to the conditions herein.

Signed on behalf of the Owner

Print Name

Date