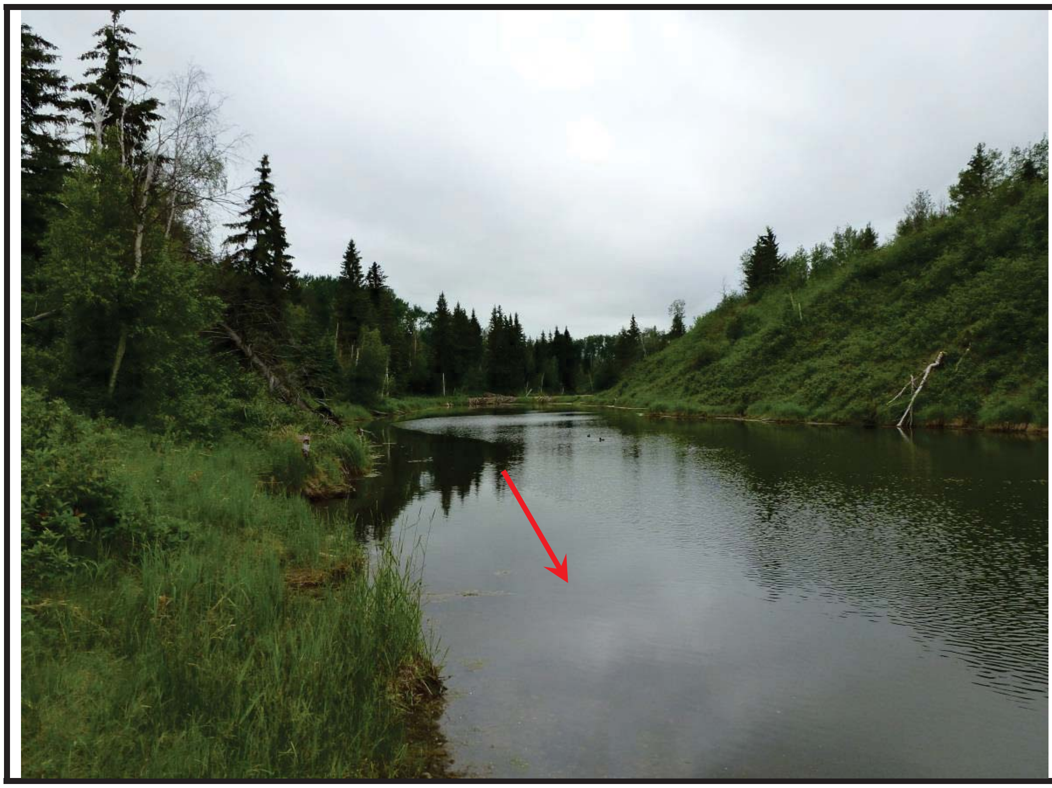


Photo 17: Looking upstream at channel and banks upstream of existing culvert.

Note: Ponding water due to beaver dam.

Photo 18: Looking upstream at channel and banks upstream of existing culvert.

Note: Ponding water due to beaver dam.

Photo 19: Looking from upstream to downstream at area U/S of existing culvert

Note: Ponding water due to beaver dam

Photo 20: : Looking from upstream to downstream at area U/S of existing culvert

Note: Ponding water due to beaver dam

Photo 21: Looking from upstream to downstream at area U/S of existing culvert

Note: Ponding water due to beaver dam and erosion control measures along facing upstream (west) sideslope.

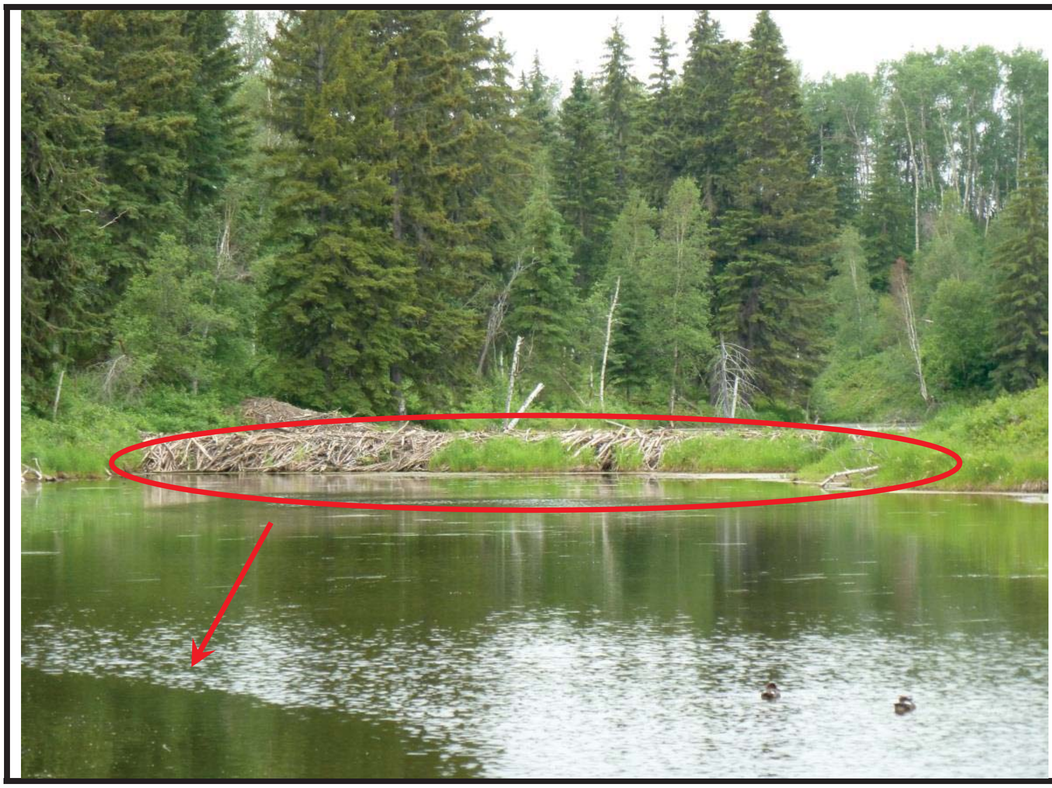


Photo 22: Looking upstream at beaver dam approximately 200 m upstream of existing culvert upstream end.

Photo 23: Looking north along upstream (west) existing sideslope.

Photo 24: Looking north along upstream (west) existing sideslope.

Photo 25: Looking downstream (east) from existing culvert downstream end.
Note: size and meander pattern of channel banks lateral erosion.

Photo 26: Looking north along downstream face of existing roadway embankment.

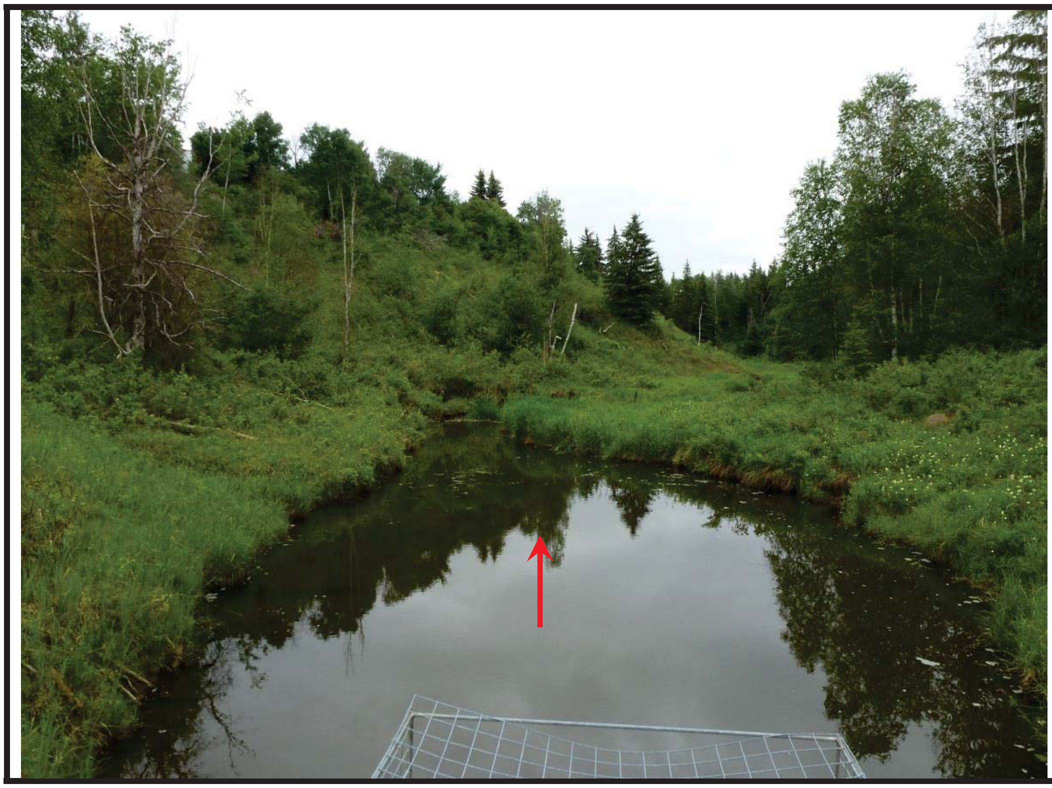


Photo 27: Looking downstream (east) from existing culvert downstream end.
Note: scour hole and water ponding (due to beaver dam)

Photo 28: Looking upstream at downstream end of existing culvert
Note: scour hole and water ponding (due to beaver dam)

Photo 29: Looking at beaver dam approximately 25 m downstream of downstream end of culvert

Photo 30: Looking downstream at left (north) bank material downstream of existing culvert.

Note: Lateral bank erosion

Photo 31: Looking downstream at left (north) bank material downstream of existing culvert.

Note: Lateral bank erosion

Photo 32: Looking downstream at left (north) bank material downstream of existing culvert.

Note: Lateral bank erosion

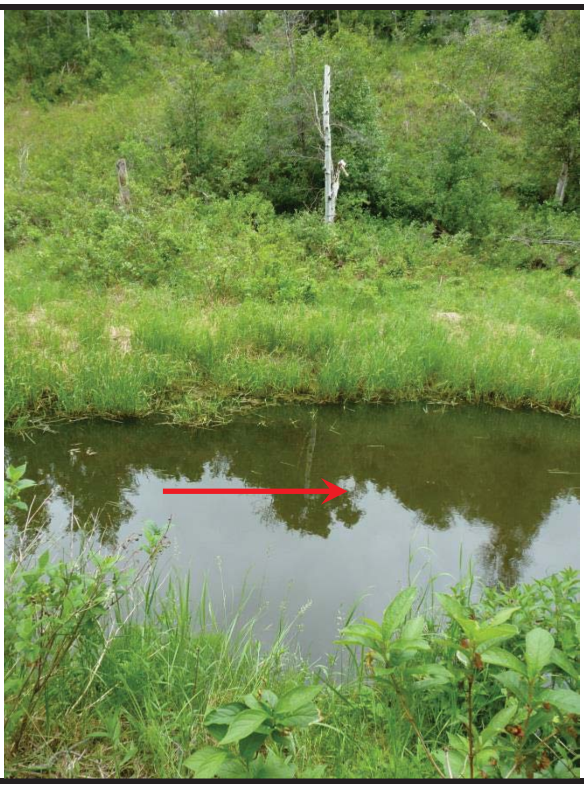


Photo 33: Looking from right (south) bank to left (north) bank at section downstream of existing culvert.

Photo 34: Looking downstream (east) from existing culvert downstream end.
Note: size and meander pattern of channel

Photo 35: Looking downstream at Wedgewood Creek from 215 Street crossing
Note: Beaver dam

Photo 36: Looking upstream at Wedgewood creek from 215 street crossing
Note: Removal of beaver dam

Photo 37: Looking upstream at Wedgewood creek from right (south) bank under Anthony Henday crossing.
Note: size of creek

Photo 38: Looking downstream at Wedgewood creek from right (south) bank under Anthony Henday crossing.
Note: size of creek

APPENDIX C

CHANNEL HYDRAULIC RESULTS

APPENDIX C1

AT'S CHANNEL CAPACITY SENSITIVITY ANALYSIS AT SECTION 1.4 KM DOWNSTREAM FROM 199 STREET CROSSING

199 Street Crossing Over Wedgewood Creek Project
AT's Channel Capacity Sensitivity Analysis at Section 1.4 km D/S from 199 Street Crossing

Field Visited

			h	Y_{cc}	Y_{spec}
S	0.01100	Y	0.7	1.2	0.45
B	2.0	A	2	3	1
h	0.7	d	0.5	1.0	0.3
T_h	3.0	V	1.8	2.9	1.4
Roughness	0.035	Q	3.2	9.5	1.5

HDG 'n' 0.055

			h	Y_{cc}	Y_{spec}
S	0.01100	Y	0.9	1.4	0.45
B	2.0	A	2	4	1
h	0.9	d	0.6	1.1	0.3
T_h	3.0	V	2.0	3.1	1.4
Roughness	0.035	Q	4.5	11.6	1.5

HDG 'n' 0.055

			h	Y_{cc}	Y_{spec}
S	0.01100	Y	1	1.5	0.45
B	2.0	A	3	4	1
h	1.0	d	0.6	1.1	0.3
T_h	3.0	V	2.1	3.2	1.4
Roughness	0.035	Q	5.3	12.7	1.4

HDG 'n' 0.055

HDG 'n' represents 'n' value as per Hydrotechnical Design Guidelines based on B and S.

APPENDIX C2

AT'S HYDRO CHAN RESULTS AT SURVEYED SECTION 50 M DOWNSTREAM FROM 199 STREET CROSSING

Project Wedgewood Creek at 199 Street_Surveyed Section E (50 m d/s from culvert)

XS Geometry

STA (m)	ELEV (m)
0.00	680.91
1.42	680.05
1.72	679.85
2.52	679.47
4.48	677.72
8.49	674.48
8.59	674.41
8.59	674.41
8.61	674.39
12.64	672.31
14.34	671.67
16.55	671.26
19.00	670.31
20.79	670.23
22.16	670.15
24.67	670.02
27.74	669.69
28.69	669.68
29.25	669.61
29.61	669.36
29.67	669.34
29.81	669.24
32.02	668.74
32.23	668.72
33.14	668.57
34.07	668.41
34.47	668.47
35.38	668.75
35.69	669.07
35.75	669.36
36.05	669.51
36.85	669.86
37.09	669.87
37.94	669.93
38.17	669.93
38.19	669.95
38.68	670.24
40.52	671.23
49.80	674.24
49.92	674.28
49.95	674.29
49.97	674.29
50.02	674.30
58.88	675.37
61.70	675.66

Channel Partition :

Left Overbank	29.2
Right Overbank	36.9

Rating Curve :

Max depth	3
Increment	0.01

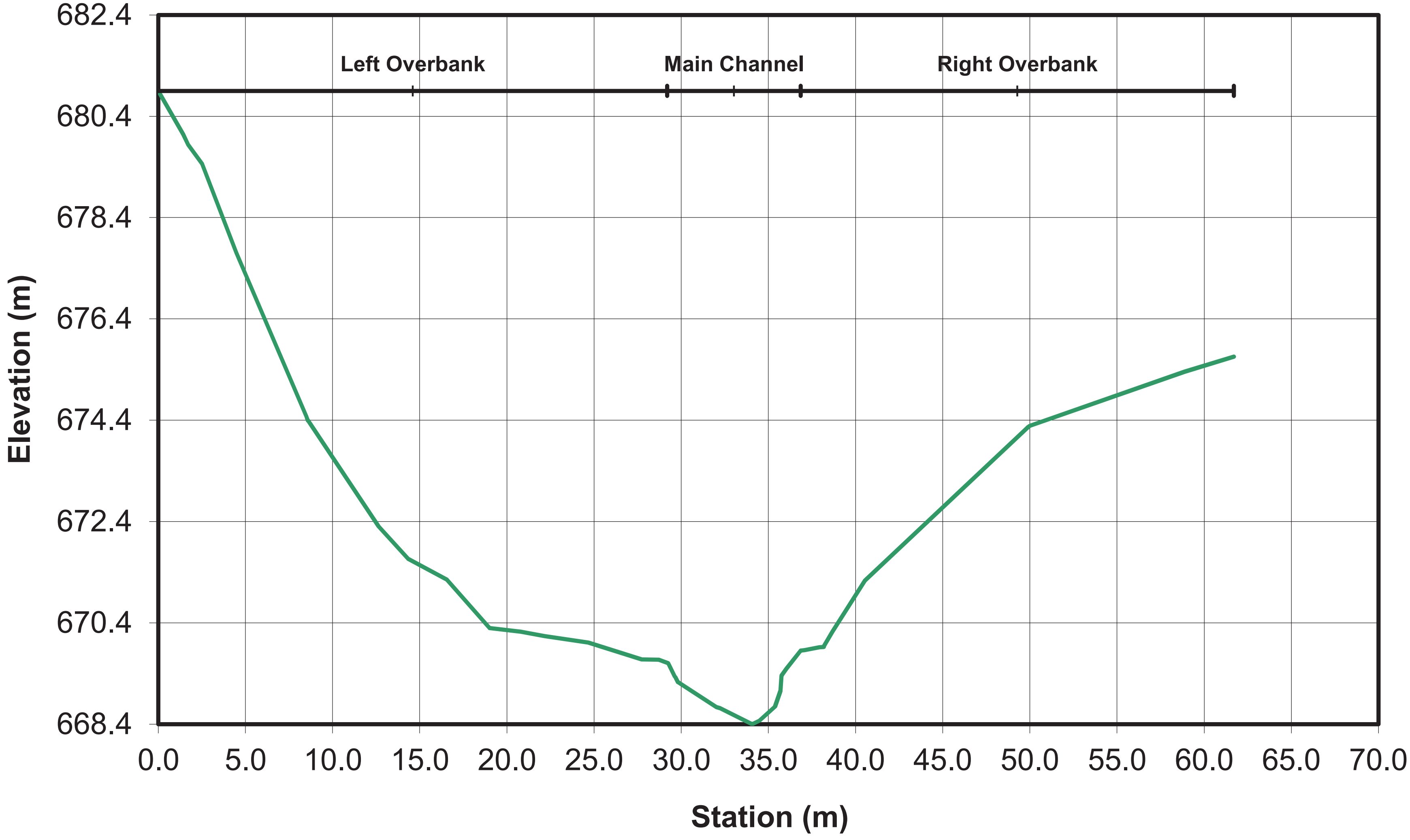
Hydraulic Parameters :

Roughness Type	n
Main Channel Roughness	0.035
Left Overbank Roughness	0.045
Right Overbank Roughness	0.05
Channel Slope	0.011

Boundary Conditions :

	Description	Q (m^3/s)	TW Elev (m)
1	Scenario 1	100	883
2			
3			
4			
5			
6			
7			
8			
9			
10			

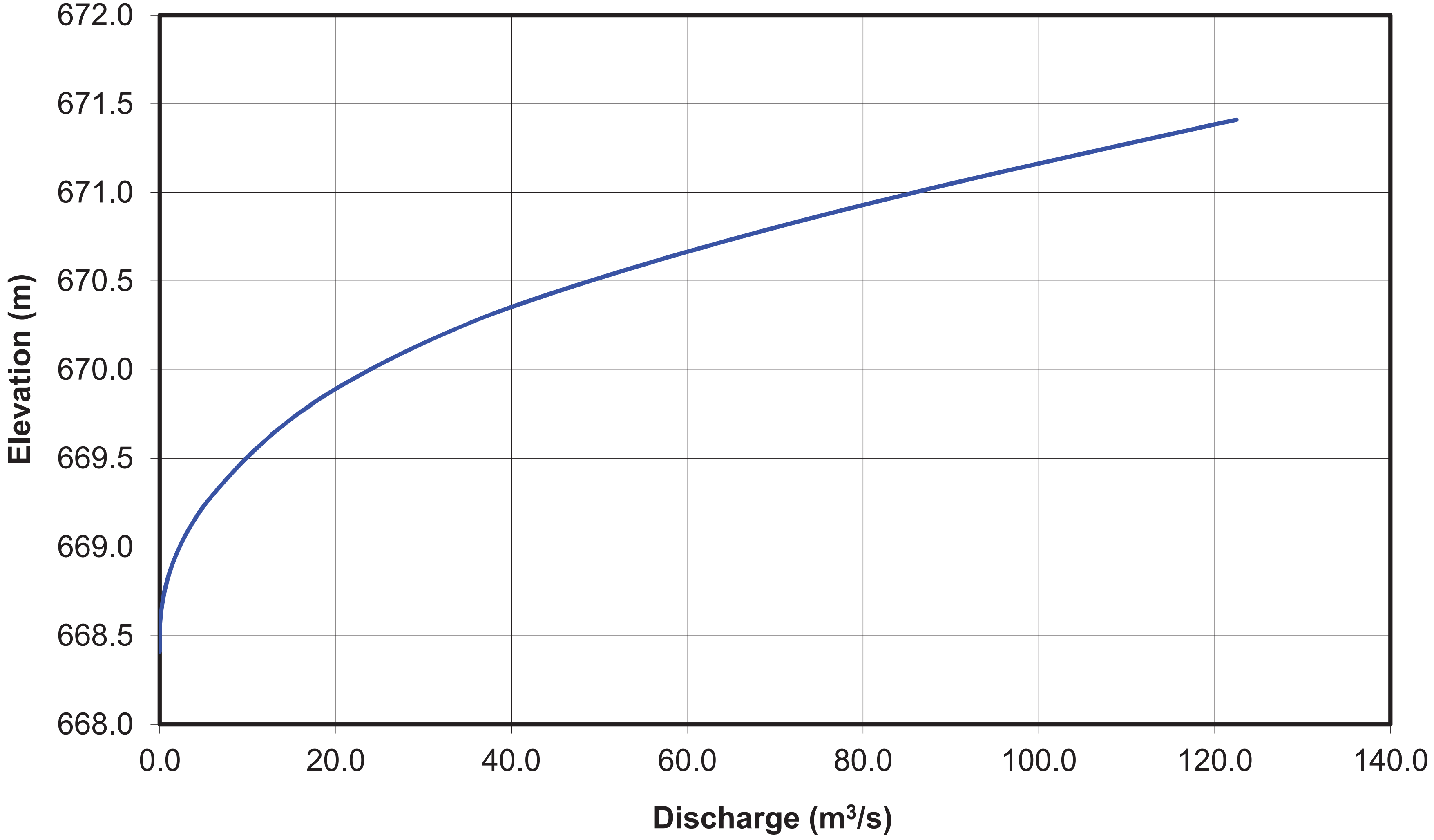
XS Plot - Wedgewood Creek at 199 Street_Surveyed Section E (50 m d/s from culvert)



	Left Overbank			Main Channel			Right Overbank			Total	Mean
Elevation	A	V	Q	A	V	Q	A	V	Q	Q	V
(m)	(m^2)	(m/s)	(m^3/s)	(m^2)	(m/s)	(m^3/s)	(m^2)	(m/s)	(m^3/s)	(m^3/s)	(m/s)
668.41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
668.44	0.00	0.00	0.00	0.01	0.18	0.00	0.00	0.00	0.00	0.00	0.18
668.47	0.00	0.00	0.00	0.02	0.29	0.01	0.00	0.00	0.00	0.01	0.29
668.50	0.00	0.00	0.00	0.05	0.39	0.02	0.00	0.00	0.00	0.02	0.39
668.53	0.00	0.00	0.00	0.08	0.48	0.04	0.00	0.00	0.00	0.04	0.48
668.56	0.00	0.00	0.00	0.13	0.55	0.07	0.00	0.00	0.00	0.07	0.55
668.59	0.00	0.00	0.00	0.18	0.62	0.11	0.00	0.00	0.00	0.11	0.62
668.62	0.00	0.00	0.00	0.24	0.69	0.16	0.00	0.00	0.00	0.16	0.69
668.65	0.00	0.00	0.00	0.31	0.75	0.23	0.00	0.00	0.00	0.23	0.75
668.68	0.00	0.00	0.00	0.38	0.81	0.31	0.00	0.00	0.00	0.31	0.81
668.71	0.00	0.00	0.00	0.47	0.86	0.40	0.00	0.00	0.00	0.40	0.86
668.74	0.00	0.00	0.00	0.56	0.91	0.51	0.00	0.00	0.00	0.51	0.91
668.77	0.00	0.00	0.00	0.66	0.97	0.64	0.00	0.00	0.00	0.64	0.97
668.80	0.00	0.00	0.00	0.77	1.04	0.80	0.00	0.00	0.00	0.80	1.04
668.83	0.00	0.00	0.00	0.88	1.10	0.97	0.00	0.00	0.00	0.97	1.10
668.86	0.00	0.00	0.00	1.00	1.16	1.16	0.00	0.00	0.00	1.16	1.16
668.89	0.00	0.00	0.00	1.12	1.22	1.37	0.00	0.00	0.00	1.37	1.22
668.92	0.00	0.00	0.00	1.25	1.28	1.59	0.00	0.00	0.00	1.59	1.28
668.95	0.00	0.00	0.00	1.38	1.33	1.83	0.00	0.00	0.00	1.83	1.33
668.98	0.00	0.00	0.00	1.51	1.38	2.09	0.00	0.00	0.00	2.09	1.38
669.01	0.00	0.00	0.00	1.66	1.43	2.37	0.00	0.00	0.00	2.37	1.43
669.04	0.00	0.00	0.00	1.80	1.48	2.67	0.00	0.00	0.00	2.67	1.48
669.07	0.00	0.00	0.00	1.95	1.53	2.98	0.00	0.00	0.00	2.98	1.53
669.10	0.00	0.00	0.00	2.11	1.57	3.32	0.00	0.00	0.00	3.32	1.57
669.13	0.00	0.00	0.00	2.27	1.62	3.68	0.00	0.00	0.00	3.68	1.62
669.16	0.00	0.00	0.00	2.43	1.67	4.06	0.00	0.00	0.00	4.06	1.67
669.19	0.00	0.00	0.00	2.60	1.71	4.45	0.00	0.00	0.00	4.45	1.71
669.22	0.00	0.00	0.00	2.78	1.75	4.86	0.00	0.00	0.00	4.86	1.75
669.25	0.00	0.00	0.00	2.95	1.80	5.31	0.00	0.00	0.00	5.31	1.80
669.28	0.00	0.00	0.00	3.13	1.86	5.81	0.00	0.00	0.00	5.81	1.86
669.31	0.00	0.00	0.00	3.31	1.91	6.32	0.00	0.00	0.00	6.32	1.91
669.34	0.00	0.00	0.00	3.49	1.96	6.85	0.00	0.00	0.00	6.85	1.96
669.37	0.00	0.00	0.00	3.68	2.01	7.38	0.00	0.00	0.00	7.38	2.01
669.40	0.00	0.00	0.00	3.86	2.05	7.92	0.00	0.00	0.00	7.92	2.05
669.43	0.00	0.00	0.00	4.05	2.09	8.48	0.00	0.00	0.00	8.48	2.09
669.46	0.00	0.00	0.00	4.25	2.13	9.05	0.00	0.00	0.00	9.05	2.13
669.49	0.00	0.00	0.00	4.44	2.17	9.65	0.00	0.00	0.00	9.65	2.17
669.52	0.00	0.00	0.00	4.64	2.21	10.27	0.00	0.00	0.00	10.27	2.21
669.55	0.00	0.00	0.00	4.85	2.25	10.90	0.00	0.00	0.00	10.90	2.25
669.58	0.00	0.00	0.00	5.05	2.29	11.55	0.00	0.00	0.00	11.55	2.29
669.61	0.00	0.00	0.00	5.26	2.32	12.23	0.00	0.00	0.00	12.23	2.32
669.64	0.00	0.11	0.00	5.47	2.36	12.90	0.00	0.00	0.00	12.90	2.36
669.67	0.01	0.19	0.00	5.69	2.40	13.64	0.00	0.00	0.00	13.64	2.39
669.70	0.04	0.19	0.01	5.91	2.44	14.39	0.00	0.00	0.00	14.40	2.42
669.73	0.09	0.30	0.03	6.13	2.47	15.16	0.00	0.00	0.00	15.19	2.44
669.76	0.15	0.38	0.06	6.35	2.51	15.95	0.00	0.00	0.00	16.01	2.46
669.79	0.22	0.45	0.10	6.57	2.55	16.76	0.00	0.00	0.00	16.86	2.48
669.82	0.30	0.51	0.15	6.80	2.59	17.58	0.00	0.00	0.00	17.74	2.50
669.85	0.38	0.56	0.21	7.03	2.62	18.43	0.00	0.00	0.00	18.64	2.52

669.88	0.47	0.61	0.29	7.26	2.66	19.33	0.00	0.10	0.00	19.62	2.54
669.91	0.58	0.66	0.38	7.48	2.71	20.26	0.02	0.18	0.00	20.65	2.55
669.94	0.69	0.70	0.48	7.71	2.75	21.21	0.05	0.23	0.01	21.71	2.57
669.97	0.81	0.74	0.60	7.94	2.79	22.17	0.09	0.33	0.03	22.80	2.58
670.00	0.93	0.78	0.73	8.17	2.83	23.15	0.13	0.41	0.05	23.93	2.59
670.03	1.07	0.81	0.87	8.40	2.87	24.13	0.18	0.47	0.08	25.08	2.60
670.06	1.22	0.82	1.00	8.63	2.91	25.13	0.22	0.53	0.12	26.25	2.61
670.09	1.39	0.84	1.17	8.86	2.95	26.14	0.27	0.58	0.16	27.46	2.61
670.12	1.58	0.86	1.35	9.09	2.99	27.15	0.32	0.63	0.20	28.71	2.61
670.15	1.78	0.88	1.57	9.32	3.02	28.18	0.37	0.67	0.25	30.00	2.62
670.18	2.00	0.91	1.83	9.55	3.06	29.22	0.42	0.71	0.30	31.35	2.62
670.21	2.24	0.94	2.11	9.78	3.10	30.27	0.47	0.75	0.35	32.74	2.62
670.24	2.49	0.97	2.41	10.01	3.13	31.33	0.53	0.78	0.41	34.15	2.62
670.27	2.76	0.99	2.72	10.24	3.16	32.40	0.58	0.82	0.47	35.59	2.62
670.30	3.05	1.00	3.06	10.47	3.20	33.48	0.64	0.85	0.54	37.07	2.62
670.33	3.35	1.06	3.54	10.70	3.23	34.56	0.70	0.88	0.61	38.71	2.62
670.36	3.66	1.11	4.07	10.93	3.26	35.66	0.76	0.91	0.69	40.41	2.63
670.39	3.97	1.17	4.63	11.16	3.30	36.76	0.82	0.93	0.77	42.16	2.64
670.42	4.29	1.22	5.22	11.39	3.33	37.87	0.89	0.96	0.85	43.95	2.65
670.45	4.60	1.27	5.84	11.62	3.36	39.00	0.95	0.99	0.94	45.77	2.67
670.48	4.92	1.32	6.49	11.85	3.39	40.12	1.02	1.01	1.03	47.64	2.68
670.51	5.24	1.37	7.16	12.07	3.42	41.26	1.09	1.03	1.13	49.54	2.69
670.54	5.56	1.41	7.86	12.30	3.45	42.41	1.16	1.06	1.23	51.49	2.71
670.57	5.89	1.46	8.58	12.53	3.48	43.56	1.23	1.08	1.33	53.47	2.72
670.60	6.22	1.50	9.33	12.76	3.50	44.72	1.31	1.10	1.44	55.49	2.74
670.63	6.55	1.54	10.11	12.99	3.53	45.88	1.38	1.13	1.56	57.55	2.75
670.66	6.88	1.59	10.91	13.22	3.56	47.06	1.46	1.15	1.67	59.64	2.77
670.69	7.21	1.63	11.74	13.45	3.59	48.24	1.54	1.17	1.80	61.77	2.78
670.72	7.55	1.67	12.59	13.68	3.61	49.42	1.62	1.19	1.93	63.94	2.80
670.75	7.89	1.71	13.47	13.91	3.64	50.62	1.70	1.21	2.06	66.14	2.81
670.78	8.23	1.75	14.36	14.14	3.66	51.82	1.79	1.23	2.20	68.38	2.83
670.81	8.57	1.78	15.29	14.37	3.69	53.03	1.87	1.25	2.34	70.65	2.85
670.84	8.92	1.82	16.23	14.60	3.72	54.24	1.96	1.27	2.48	72.96	2.86
670.87	9.27	1.86	17.20	14.83	3.74	55.46	2.05	1.29	2.64	75.30	2.88
670.90	9.62	1.89	18.20	15.06	3.76	56.68	2.14	1.31	2.79	77.67	2.90
670.93	9.97	1.93	19.21	15.29	3.79	57.91	2.23	1.33	2.96	80.08	2.91
670.96	10.32	1.96	20.25	15.52	3.81	59.15	2.32	1.34	3.12	82.53	2.93
670.99	10.68	2.00	21.32	15.75	3.84	60.39	2.42	1.36	3.30	85.00	2.95
671.02	11.04	2.03	22.40	15.98	3.86	61.64	2.52	1.38	3.47	87.51	2.96
671.05	11.40	2.06	23.51	16.21	3.88	62.89	2.62	1.40	3.66	90.06	2.98
671.08	11.77	2.09	24.64	16.44	3.90	64.15	2.72	1.41	3.84	92.63	3.00
671.11	12.13	2.13	25.79	16.66	3.93	65.41	2.82	1.43	4.04	95.24	3.01
671.14	12.50	2.16	26.97	16.89	3.95	66.68	2.92	1.45	4.24	97.89	3.03
671.17	12.87	2.19	28.17	17.12	3.97	67.96	3.03	1.47	4.44	100.56	3.04
671.20	13.25	2.22	29.38	17.35	3.99	69.23	3.14	1.48	4.65	103.27	3.06
671.23	13.62	2.25	30.63	17.58	4.01	70.52	3.25	1.50	4.87	106.01	3.08
671.26	14.00	2.28	31.89	17.81	4.03	71.81	3.36	1.51	5.07	108.77	3.09
671.29	14.38	2.30	33.06	18.04	4.05	73.10	3.47	1.52	5.28	111.44	3.10
671.32	14.77	2.32	34.26	18.27	4.07	74.40	3.59	1.53	5.50	114.15	3.12
671.35	15.16	2.34	35.47	18.50	4.09	75.70	3.71	1.54	5.72	116.89	3.13
671.38	15.56	2.36	36.72	18.73	4.11	77.00	3.83	1.55	5.95	119.67	3.14
671.41	15.96	2.38	37.99	18.96	4.13	78.31	3.95	1.57	6.19	122.49	3.15

Rating Curve Plot - Wedgewood Creek at 199 Street_Surveyed Section E (50 m d/s from culvert)



APPENDIX D

STRUCTURE HYDRAULIC RESULTS

APPENDIX D1

AT'S HYDRO CULV RESULTS FOR EXISTING 1.8 M DIAMETER SPCSP CULVERT

Project Wedgewood Creek at 199 street - Existing culvert

Culvert Data

Pipe No.		1	2	3	4
Include (Y/N)		Y			
Station (m)		380.086			
U/S Invert El (m)		671.430			
D/S Invert El (m)		669.140			
Length (m)		68.50			
Roughness	n	0.032			
Ent. Loss Coeff.		0.7			
Exit Loss Coeff.		1			
Shape		R			
Rise (m)		1.80			
Span (m)					

slope= 0.03343066

Boundary Conditions :

Description	Q (m^3/s)	TW Elev (m)	D/S Vel (m/s)
Q design	14	670.64	2.42
Q2	1.4	669.82	1.22
Qcheck1	30.8	671.14	2.62
Qcheck2	61.8	671.64	2.78

Output Summary - Wedgewood Creek at 199 street - Existing culvert

BC No.	1	2	3	4	
Q (cms)	14.0	1.4	30.8	61.8	
TW (m)	670.64	669.82	671.14	671.64	
Vds (m/s)	2.42	1.22	2.62	2.78	
HW (m)	679.72	672.35	713.99	843.13	
Headloss (m)	8.78	2.46	42.50	171.10	

BC No. 1 - Q design

	Pipe 1				
Q (cms)	14.00				
Freeboard (m)	-6.49				
Ynorm (m)	1.80				
Ycrit (m)	1.71				
Vout (m/s)	5.50				
Vin (m/s)	5.50				
Flow Desc.	Full Flow				

BC No. 2 - Q2

	Pipe 1				
Q (cms)	1.40				
Freeboard (m)	0.88				
Ynorm (m)	0.49				
Ycrit (m)	0.57				
Vout (m/s)	2.28				
Vin (m/s)	2.02				
Flow Desc.	S1 - Jump - S2				

BC No. 3 - Qcheck1

	Pipe 1				
Q (cms)	30.80				
Freeboard (m)	-40.76				
Ynorm (m)	1.80				
Ycrit (m)	1.80				
Vout (m/s)	12.10				
Vin (m/s)	12.10				
Flow Desc.	Full Flow				

BC No. 4 - Qcheck2

	Pipe 1				
Q (cms)	61.80				
Freeboard (m)	-169.90				
Ynorm (m)	1.80				
Ycrit (m)	1.80				
Vout (m/s)	24.29				
Vin (m/s)	24.29				
Flow Desc.	Full Flow				

Note: Pipe 1 is existing 1.8 m diameter SPCSP culvert.

APPENDIX D2

AT'S HYDRO CULV RESULTS FOR PROPOSED 3.0 M DIAMETER CSP CULVERT AND 4.5 M RISE X 14.0 M SPAN WILDLIFE PASSAGE BRIDGE

Project Wedgewood Creek at 199 street - Proposed 3.0 m CSP

Culvert Data

Pipe No.		1	2	3	4
Include (Y/N)		Y	Y		
Station (m)		380.086	380.086		
U/S Invert El (m)		670.000	675.930		
D/S Invert El (m)		668.400	675.770		
Length (m)		117.50	32.00		
Roughness	n	0.042	0.035		
Ent. Loss Coeff.		0.7	0.7		
Exit Loss Coeff.		1	1		
Shape		R	B		
Rise (m)		3.00	4.50		
Span (m)			14.00		

slope=	0.01361702	0.005

Boundary Conditions :

Description	Q (m^3/s)	TW Elev (m)	D/S Vel (m/s)
Q design	14	670.45	2.42
Q2	1.4	669.63	1.22
Qcheck1	30.8	670.95	2.62
Qcheck2	61.8	671.45	2.78

Output Summary - Wedgewood Creek at 199 street - Proposed 3.0 m CSP

BC No.	1	2	3	4		
Q (cms)	14.0	1.4	30.8	61.8		
TW (m)	670.45	669.63	670.95	671.45		
Vds (m/s)	2.42	1.22	2.62	2.78		
HW (m)	672.73	670.77	676.40	677.49		
Headloss (m)	1.99	1.06	5.10	5.64		

BC No. 1 - Q design

	Pipe 1	Pipe 2			
Q (cms)	13.99	0.00			
Freeboard (m)	0.27	0.00			
Ynorm (m)	2.15	0.00			
Ycrit (m)	1.62	0.00			
Vout (m/s)	2.36	0.00			
Vin (m/s)	2.56	0.00			
Flow Desc.	M1	No Flow			

BC No. 2 - Q2

	Pipe 1	Pipe 2			
Q (cms)	1.40	0.00			
Freeboard (m)	2.23	0.00			
Ynorm (m)	0.60	0.00			
Ycrit (m)	0.50	0.00			
Vout (m/s)	0.47	0.00			
Vin (m/s)	1.40	0.00			
Flow Desc.	M1	No Flow			

BC No. 3 - Qcheck1

	Pipe 1	Pipe 2			
Q (cms)	25.29	5.53			
Freeboard (m)	-3.40	4.03			
Ynorm (m)	3.00	0.38			
Ycrit (m)	2.21	0.25			
Vout (m/s)	3.61	1.57			
Vin (m/s)	3.58	1.05			
Flow Desc.	M2 - Full	M2			

BC No. 4 - Qcheck2

	Pipe 1	Pipe 2			
Q (cms)	26.86	35.06			
Freeboard (m)	-4.49	2.94			
Ynorm (m)	3.00	1.21			
Ycrit (m)	2.27	0.86			
Vout (m/s)	3.80	2.90			
Vin (m/s)	3.80	2.22			
Flow Desc.	Full Flow	M2			

Note: Pipe 1 is proposed 3.0 m diameter CSP culvert and Pipe 2 is 4.5 m rise x 14.0 m span wildlife passage bridge.

APPENDIX D3

AT'S HYDRO CULV RESULTS FOR PROPOSED 3.05 M DIAMETER SPCSP CULVERT AND 4.5 M RISE X 14.0 M SPAN WILDLIFE PASSAGE BRIDGE

Project	Wedgewood Creek at 199 street - Proposed 3.05 m SPCSP

Culvert Data

Pipe No.		1	2	3	4
Include (Y/N)		Y	Y		
Station (m)		380.086	380.086		
U/S Invert El (m)		670.000	675.930		
D/S Invert El (m)		668.400	675.770		
Length (m)		117.50	32.00		
Roughness	n	0.042	0.035		
Ent. Loss Coeff.		0.7	0.7		
Exit Loss Coeff.		1	1		
Shape		R	B		
Rise (m)		3.05	4.50		
Span (m)			14.00		

slope= 0.01361702 0.005

Boundary Conditions :

Description	**Q (m^3/s)**	**TW Elev (m)**	**D/S Vel (m/s)**
Q design	14	670.45	2.42
Q2	1.4	669.63	1.22
Qcheck1	30.8	670.95	2.62
Qcheck2	61.8	671.45	2.78

Output Summary - Wedgewood Creek at 199 street - Proposed 3.05 m SPCSP

BC No.	1	2	3	4		
Q (cms)	14.0	1.4	30.8	61.8		
TW (m)	670.45	669.63	670.95	671.45		
Vds (m/s)	2.42	1.22	2.62	2.78		
HW (m)	672.70	670.76	676.34	677.44		
Headloss (m)	1.95	1.06	5.04	5.60		

BC No. 1 - Q design

	Pipe 1	Pipe 2			
Q (cms)	13.99	0.00			
Freeboard (m)	0.35	0.00			
Ynorm (m)	2.11	0.00			
Ycrit (m)	1.62	0.00			
Vout (m/s)	2.32	0.00			
Vin (m/s)	2.57	0.00			
Flow Desc.	M1	No Flow			

BC No. 2 - Q2

	Pipe 1	Pipe 2			
Q (cms)	1.40	0.00			
Freeboard (m)	2.29	0.00			
Ynorm (m)	0.59	0.00			
Ycrit (m)	0.49	0.00			
Vout (m/s)	0.47	0.00			
Vin (m/s)	1.40	0.00			
Flow Desc.	M1	No Flow			

BC No. 3 - Qcheck1

	Pipe 1	Pipe 2			
Q (cms)	26.38	4.42			
Freeboard (m)	-3.29	4.09			
Ynorm (m)	3.05	0.33			
Ycrit (m)	2.24	0.22			
Vout (m/s)	3.68	1.45			
Vin (m/s)	3.61	0.96			
Flow Desc.	M2 - Full	M2			

BC No. 4 - Qcheck2

	Pipe 1	Pipe 2			
Q (cms)	27.89	33.56			
Freeboard (m)	-4.39	2.99			
Ynorm (m)	3.05	1.18			
Ycrit (m)	2.30	0.84			
Vout (m/s)	3.82	2.86			
Vin (m/s)	3.82	2.18			
Flow Desc.	Full Flow	M2			

Note: Pipe 1 is proposed 3.05 m diameter SPCSP culvert and Pipe 2 is 4.5 m rise x 14.0 m span wildlife passage bridge.

APPENDIX D4

AT'S HYDRO CULV RESULTS FOR PROPOSED 2.4 M RISE X 3.0 M SPAN CONCRETE BOX CULVERT AND 4.5 M RISE X 14.0 M SPAN WILDLIFE PASSAGE BRIDGE

Project Wedgewood Creek at 199 street - Proposed Concrete Box Culvert

Culvert Data

Pipe No.		1	2	3	4
Include (Y/N)		Y	Y		
Station (m)		380.086	380.086		
U/S Invert El (m)		670.150	675.930		
D/S Invert El (m)		668.550	675.770		
Length (m)		117.50	32.00		
Roughness	n	0.042	0.035		
Ent. Loss Coeff.		0.7	0.7		
Exit Loss Coeff.		1	1		
Shape		B	B		
Rise (m)		2.40	4.50		
Span (m)		3.00	14.00		

slope=	0.01361702	0.005

Boundary Conditions :

Description	Q (m^3/s)	TW Elev (m)	D/S Vel (m/s)
Q design	14	670.45	2.42
Q2	1.4	669.65	1.22
Qcheck1	30.8	670.95	2.62
Qcheck2	61.8	671.45	2.78

Output Summary - Wedgewood Creek at 199 street - Proposed Concrete Box Culve

BC No.	1	2	3	4		
Q (cms)	14.0	1.4	30.8	61.8		
TW (m)	670.45	669.65	670.95	671.45		
Vds (m/s)	2.42	1.22	2.62	2.78		
HW (m)	672.58	670.66	676.44	677.51		
Headloss (m)	1.83	0.93	5.14	5.67		

BC No. 1 - Q design

	Pipe 1	Pipe 2			
Q (cms)	13.99	0.00			
Freeboard (m)	-0.03	0.00			
Ynorm (m)	1.89	0.00			
Ycrit (m)	1.30	0.00			
Vout (m/s)	2.12	0.00			
Vin (m/s)	2.44	0.00			
Flow Desc.	M1	No Flow			

BC No. 2 - Q2

	Pipe 1	Pipe 2			
Q (cms)	1.39	0.00			
Freeboard (m)	1.89	0.00			
Ynorm (m)	0.37	0.00			
Ycrit (m)	0.28	0.00			
Vout (m/s)	0.40	0.00			
Vin (m/s)	1.24	0.00			
Flow Desc.	M1	No Flow			

BC No. 3 - Qcheck1

	Pipe 1	Pipe 2			
Q (cms)	24.54	6.29			
Freeboard (m)	-3.89	3.99			
Ynorm (m)	2.40	0.42			
Ycrit (m)	1.90	0.27			
Vout (m/s)	3.41	1.64			
Vin (m/s)	3.41	1.10			
Flow Desc.	Full Flow	M2			

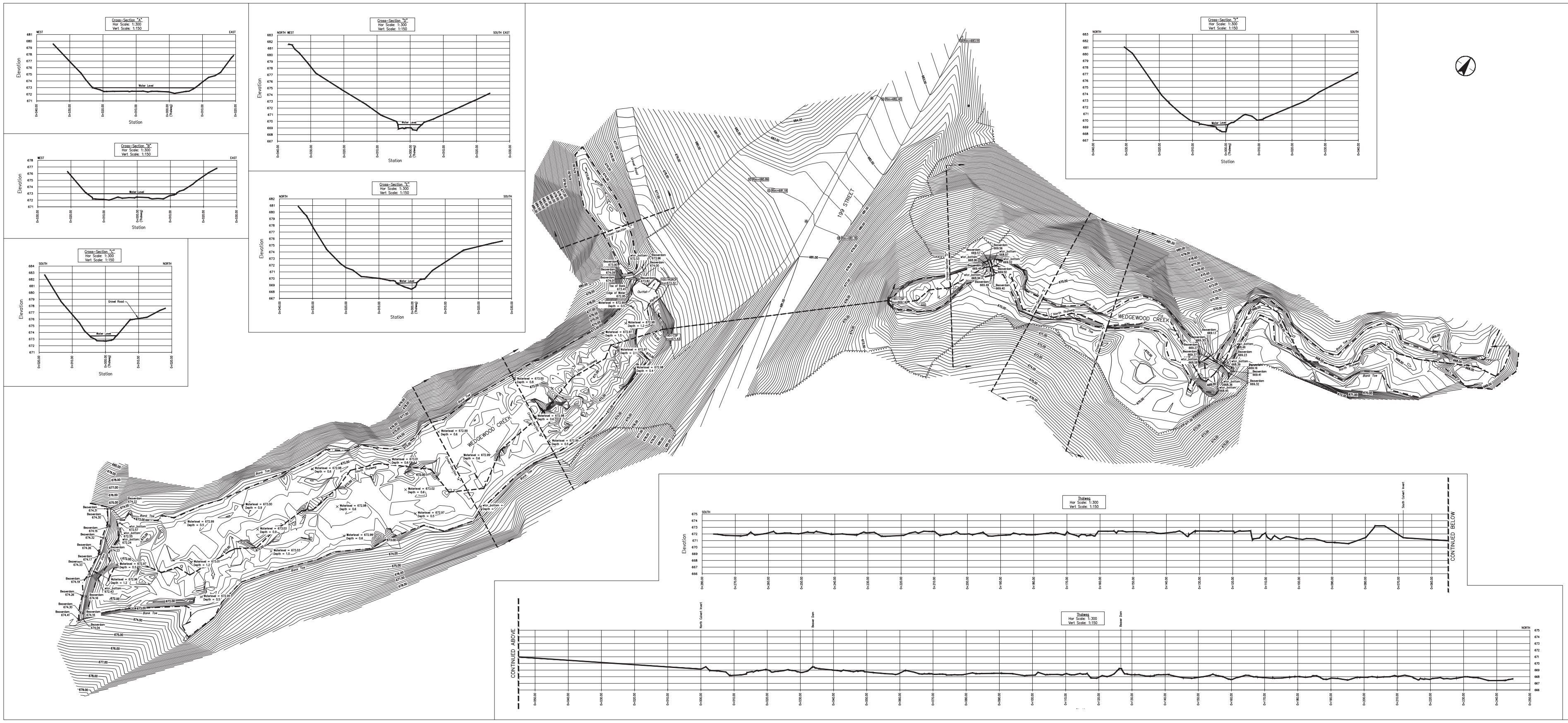
BC No. 4 - Qcheck2

	Pipe 1	Pipe 2			
Q (cms)	25.77	36.03			
Freeboard (m)	-4.96	2.92			
Ynorm (m)	2.40	1.23			
Ycrit (m)	1.96	0.88			
Vout (m/s)	3.58	2.93			
Vin (m/s)	3.58	2.24			
Flow Desc.	Full Flow	M2			

Note: Pipe 1 is proposed 2.4 m rise x 3.0 m span concrete box culvert and Pipe 2 is 4.5 m rise x 14.0 m span wildlife passage bridge.

APPENDIX E

SITE SURVEY INFORMATION



Stantec Geomatics Ltd.
10160-112th Street NW
Edmonton, Alberta, Canada
T5K 2L6
Tel. 780-917-7000
Fax. 780-917-7289
www.stantec.com

Client
RIVERVIEW HEIGHTS ESTATES LTD.

Title
WEDGEWOOD CREEK

EDMONTON **ALBERTA**

KEVIN MACLEOD, A.L.S. SCALE: 1:300
SURVEY PERFORMED BETWEEN JUNE 18th TO JUNE 22nd, 2015

Legend

MANHOLE	⊗
LIGHT STANDARDS	☼
POWERPOLE AND GUYWIRE	◐—•
CULVERT	≺
FENCE LINES	—x—x—x—x—
EDGE OF WATER	▬ · ▬ · ▬ · ▬
PROFILE/CROSS-SECTION LINES	▬ ▬ ▬ ▬ ▬
BUSHLINES	⌒⌒⌒⌒⌒⌒

Notes

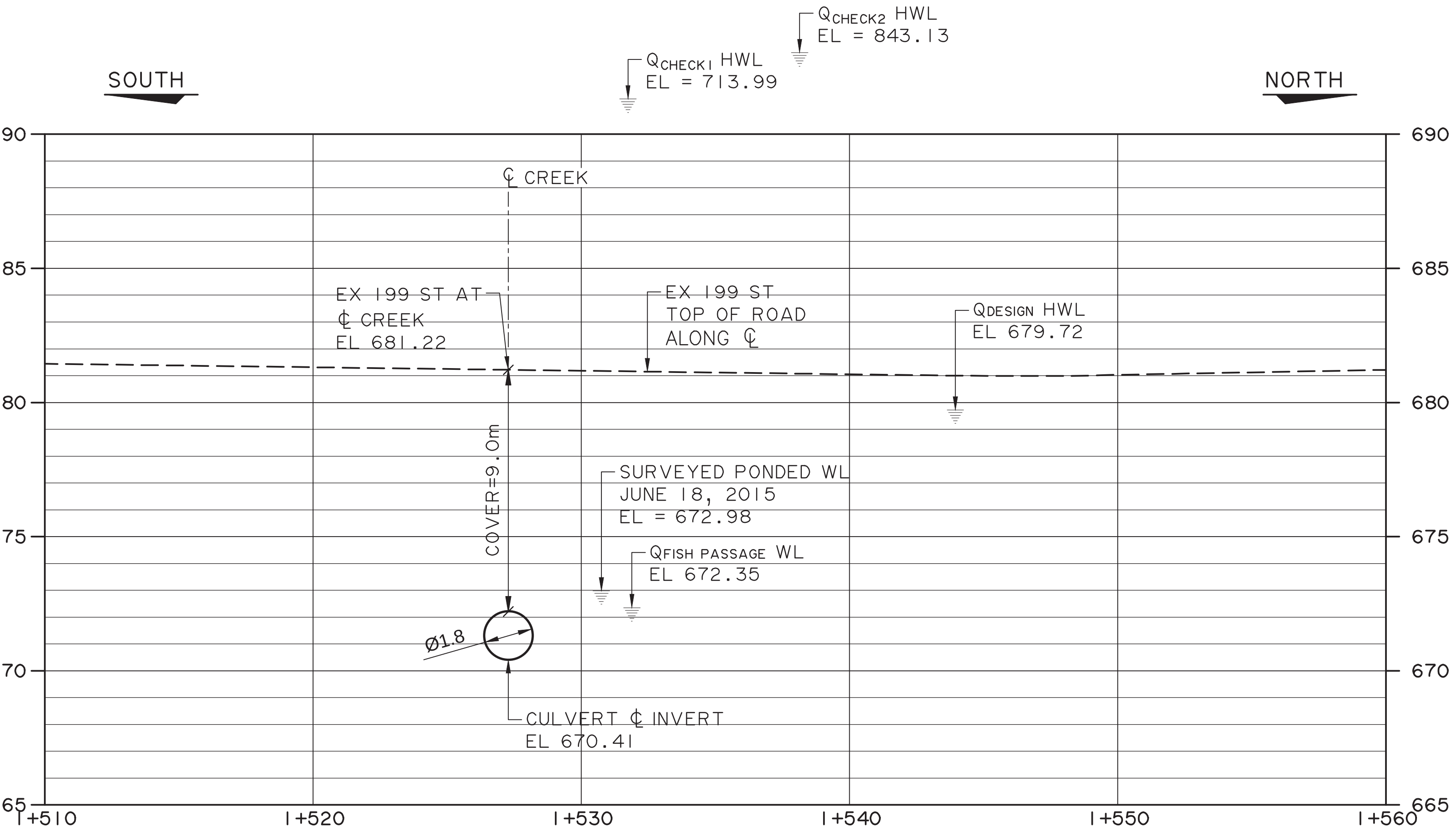
- ALL DISTANCES AND ELEVATIONS ARE IN METRES AND DECIMALS THEREOF.
- ELEVATIONS ARE GEODETIC AND ARE DERIVED FROM ALBERTA SURVEY CONTROL MARKER 169664 (ELEVATION =689.23)

0			
Revision	By	Appd.	Date

DRFT: JB FILE: 1161103725 Wedgewood Creek
EXM: BOOK No(s): 1388 SITE SURVEY
V:\1562\active\1161103725\drawing\103725ST2_C3D.dwg
7/9/2015 8:21 AM By: Bly, John

APPENDIX F

SKECHES



EXISTING 1-1.8 m DIA x 68.6 m LONG SPCSP CULVERT

1:250

ORIGINAL SHEET - ANSI A

JULY, 2015
1161103725

W:\temporary\1161103725\drawing\model_files\c3d\103725_culv.dwg
2015/08/14 12:44 PM By: Dressler, Trevor

Stantec

10160 - 112th Street
Edmonton AB
www.stantec.com

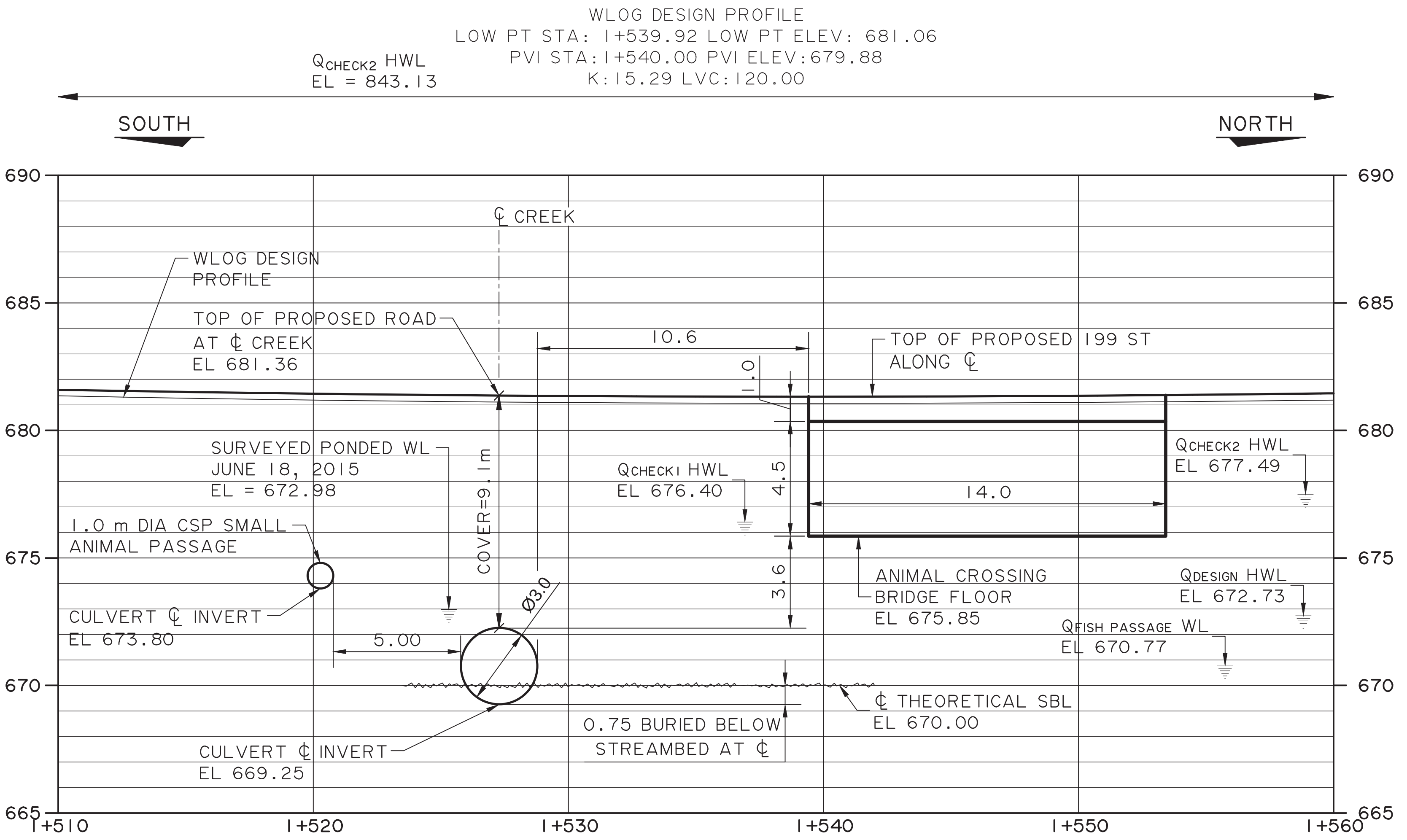
NOTES:

- ALL DIMENSIONS AND ELEVATIONS ARE SHOWN IN METERS
- FLOW ELEVATIONS ARE AT CULVERT INLET
- WATER IN WEDGEWOOD CREEK WAS PONDED DUE TO WOOD DEBRIS ACCUMULATION AROUND CAGE AT CULVERT INLET (SEE SITE VISIT PHOTOS GIVEN IN APPENDIX B)

Client/Project
QUALICO
199 STREET
CROSSING AT WEDGEWOOD CREEK

Figure No.
SK-1

Title
EXISTING CULVERT HYDRAULICS



1-3.0 m DIA x 117.5 m LONG CSP CULVERT

MANNINGS ROUGHNESS n = 0.042

1:250

ORIGINAL SHEET - ANSI A

JULY, 2015
1161103725

W:\temporary\1161103725\drawing\model_files\c3d\103725_culv.dwg
2015/08/14 12:44 PM By: Dressler, Trevor

Stantec

10160 - 112th Street
Edmonton AB
www.stantec.com

NOTES:

- ALL DIMENSIONS AND ELEVATIONS ARE SHOWN IN METERS
- FLOW ELEVATIONS ARE AT CULVERT INLET
- WATER IN WEDGEWOOD CREEK WAS PONDED DUE TO WOOD DEBRIS ACCUMULATION AROUND CAGE AT CULVERT INLET (SEE SITE VISIT PHOTOS GIVEN IN APPENDIX B)

Client/Project

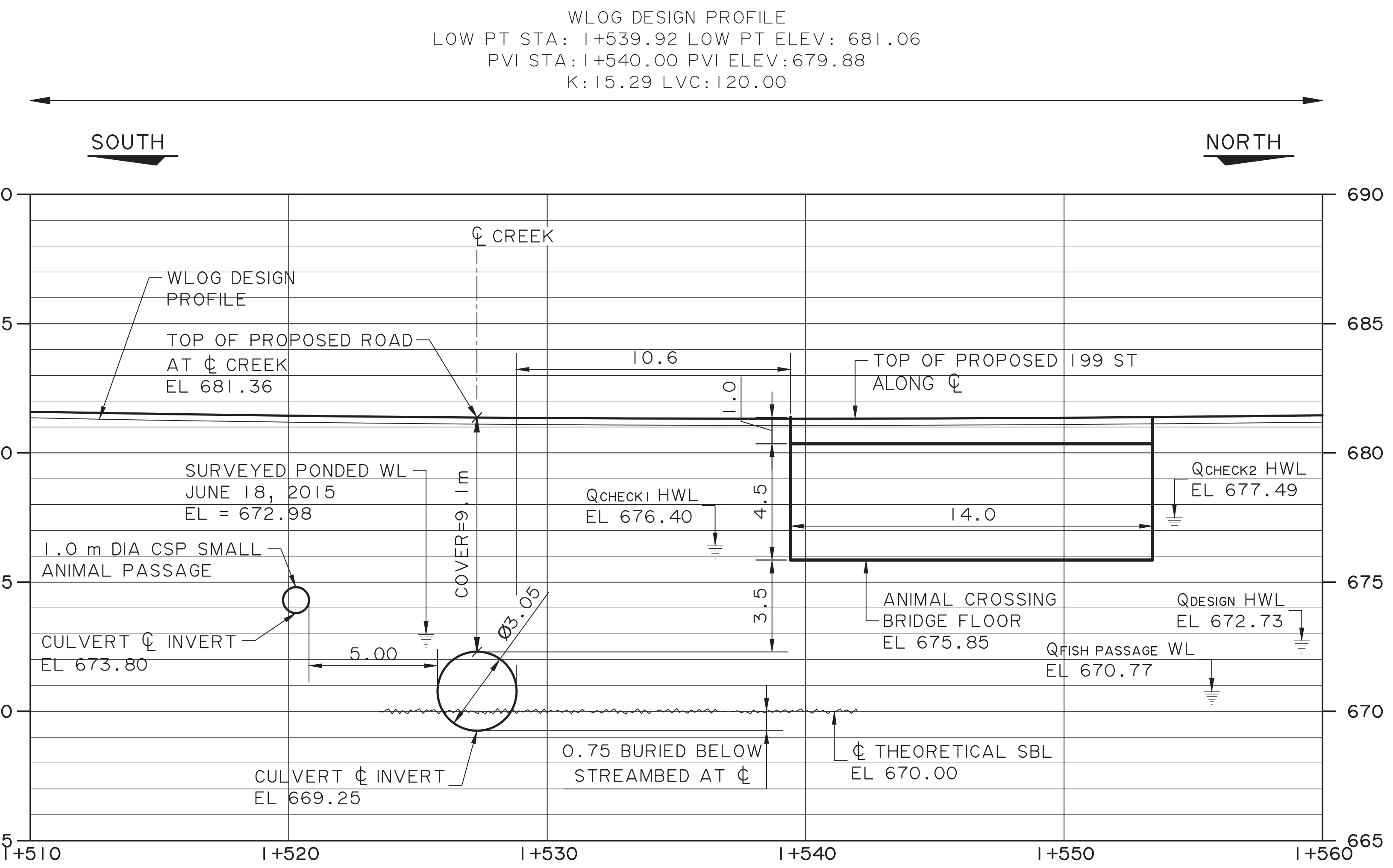
QUALICO
199 STREET
CROSSING AT WEDGEWOOD CREEK

Figure No.

SK-2

Title

OPTION 1 OF 3
CULVERT HYDRAULICS



1-3.05 m DIA x 117.7 m LONG SPCSP CULVERT

MANNINGS ROUGHNESS n = 0.042

1:250

ORIGINAL SHEET - ANSI A

JULY, 2015
1161103725

W:\temporary\1161103725\drawing\model_files\c3d\103725_culv.dwg
2015/08/14 12:44 PM By: Dressler, Trevor

Stantec

10160 - 112th Street
Edmonton AB
www.stantec.com

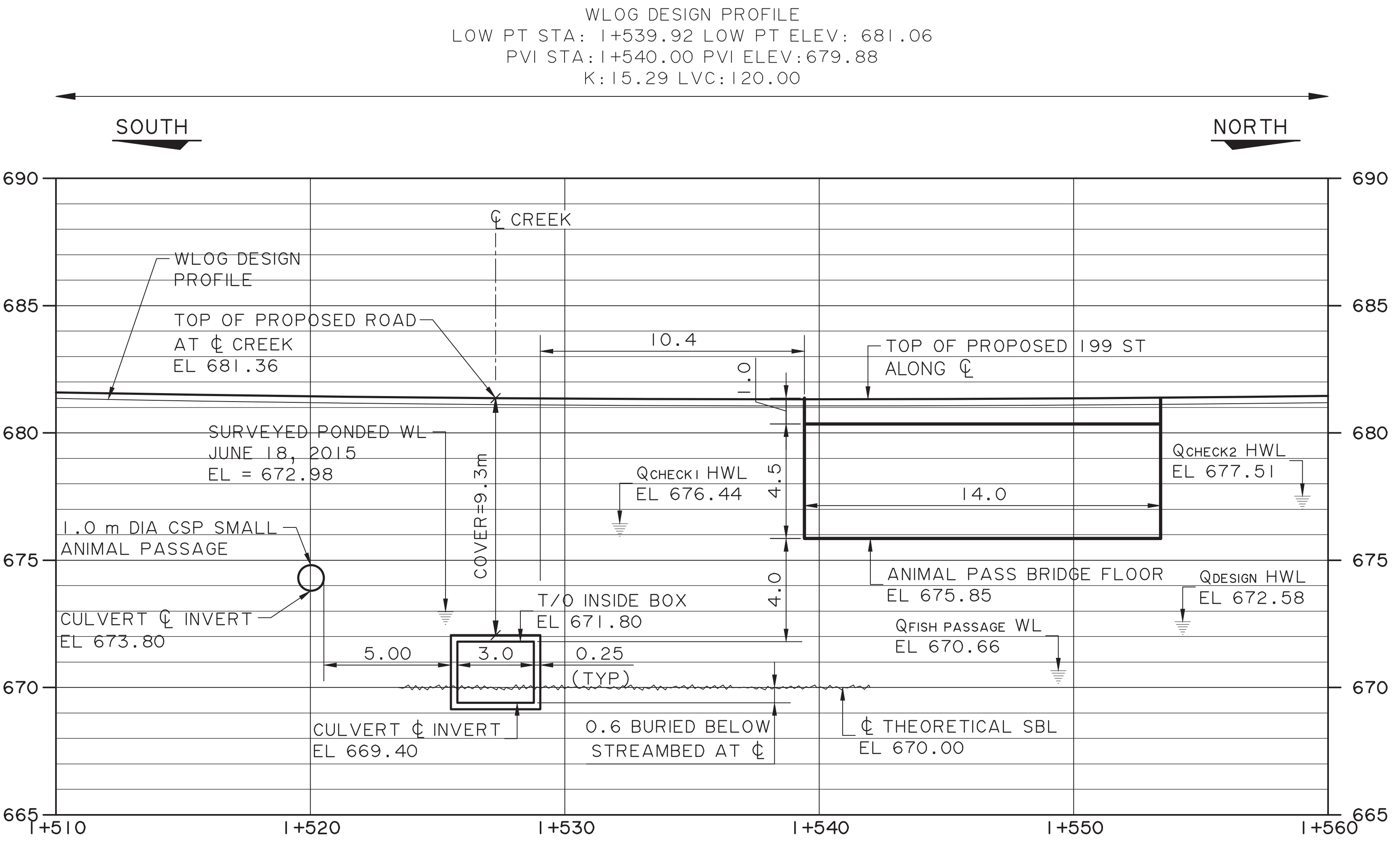
NOTES:

- ALL DIMENSIONS AND ELEVATIONS ARE SHOWN IN METERS
- FLOW ELEVATIONS ARE AT CULVERT INLET
- WATER IN WEDGEWOOD CREEK WAS PONDED DUE TO WOOD DEBRIS ACCUMULATION AROUND CAGE AT CULVERT INLET (SEE SITE VISIT PHOTOS GIVEN IN APPENDIX B)

Client/Project
QUALICO
199 STREET
CROSSING AT WEDGEWOOD CREEK

Figure No.
SK-3

Title
OPTION 2 OF 3
CULVERT HYDRAULICS



1-3.0 m SPAN x 2.4 m RISE x 117.5 m LONG CONCRETE BOX CULVERT OPTION

MANNINGS ROUGHNESS n = 0.042

1:250

ORIGINAL SHEET - ANSI A

JULY, 2015
1161103725

W:\temporary\1161103725\drawing\model_files\c3d\103725_culv.dwg
2015/08/14 12:44 PM By: Dressler, Trevor

Stantec

10160 - 112th Street
Edmonton AB
www.stantec.com

NOTES:

- ALL DIMENSIONS AND ELEVATIONS ARE SHOWN IN METERS
- FLOW ELEVATIONS ARE AT CULVERT INLET
- WATER IN WEDGEWOOD CREEK WAS PONDED DUE TO WOOD DEBRIS ACCUMULATION AROUND CAGE AT CULVERT INLET (SEE SITE VISIT PHOTOS GIVEN IN APPENDIX B)

Client/Project

QUALICO
199 STREET
CROSSING AT WEDGEWOOD CREEK

Figure No.

SK-4

Title

OPTION 3 OF 3
CONCRETE BOX CULVERT HYDRAULICS

APPENDIX G

GEOTECHNICAL REPORT

HOGGAN ENGINEERING & TESTING (1980) LTD.

REPORT NO: 6004-38

GEOTECHNICAL INVESTIGATION
PROPOSED 199TH STREET UPGRADES – STAGE 1
UNDERGROUND UTILITIES, DEEP FILL CULVERT AND WILDLIFE CROSSING
35TH AVENUE TO WOODBEND WYND NW
EDMONTON, ALBERTA

May 2015

HOGGAN ENGINEERING & TESTING (1980) LTD.
17505 – 106th Avenue
Edmonton, Alberta
T5S 1E7

PHONE: 780-489-0880
FAX: 780-489-0800

GEOTECHNICAL INVESTIGATION

PROJECT: Proposed 199th Street Upgrades – Stage 1
Underground Utilities, Deep Fill Culvert and Wildlife Crossing

LOCATION: 35th Avenue to Woodbend Wynd NW
Edmonton, Alberta

CLIENT: Qualico Communities
c/o Stantec Consulting Ltd.
10160 – 112th Street
Edmonton, Alberta
T5K 2L6

ATTENTION: Tony Chiarello, E.I.T.

1.0 INTRODUCTION

This report presents the results of the subsurface investigations made on the site of the proposed road upgrading in Edmonton, Alberta. The objective of the investigation is to determine the existing subsoil conditions along the proposed road alignment and to provide geotechnical recommendations for the roadway development, underground utility installation and wildlife crossing construction based on the soil data retrieved. Authorization to proceed with the investigation was received from Petrea Chamney of Stantec in February 2015. Field work for the project was completed in April 2015. Environmental and previous land use issues are beyond the scope of this report.

2.0 SITE DESCRIPTION

It is understood that the project consists of upgrading the existing rural 199th Street roadway to a four-lane urban arterial roadway, from 35th Avenue to 23rd Avenue. This project concentrates on Stage 1 of the upgrades between 35th Avenue to Woodbend Wynd NW as part of the overall Riverview Neighbourhood development. The new lanes will be constructed west of the existing 199th Street with minor widening to the east to accommodate walks and light standards. Water and storm services will be installed below the roadway as part of this project. The proposed depth of the utilities 6 to13 meters below existing ground surface. The deeper utilities are anticipated to be below the Wedgewood Creek (WWC).

In addition the proposed road upgrade will include replacement of the existing culvert from a 1900 millimeter diameter to a 2400 millimeter and constructing a separate wildlife passage at the WWC. It is understood that the wildlife passage will be a single-span bridge.

The existing 199th Street is a rural profile road which runs north south within the project limits. Stage 1 project limits are typically within the WWC crossing section. Power lines were noted on the east side of the road. Generally the road had a rolling terrain with a low area at the WWC location.

At the time of inspection, 199th Street was surfaced with hot mix asphalt. The road appeared in fair condition with no major rutting, cracking or failure noted.

Site reconnaissance was completed on the side slopes of the existing 199th Street at WWC on April 2, 2015. During the site inspection it was noted that the west slope was approximately 2.5H: 1V while the east slope was approximately 2H:1V. Both side slopes were covered with grass, light bush and small trees. The east slope featured areas where soil disturbance had occurred, likely due to the installation of underground utilities the previous year. Toe erosion was not noted on either side of the slope. A culvert, approximately 1900 millimeters in diameter was noted in the creek to allow for water flow under the road. The culvert appeared to be straight with no curvature. A protective metal cage was observed on the upstream portion of the culvert, on the west side of the road. The cage and culvert inlet was surrounded by a beaver dam. Further west of the culvert, a concrete storm outfall exists. The outfall was constructed in 2014. Beaver dam activity is quite evident upstream and downstream of the 199th Street. Beaver dams up to 1.5 to 2 meters high are noted. Evidence of side slope instability was not noted during our site visit. Site photos are provided in Appendix II.

Geotechnical Report Review

A search for geotechnical information was requested from the City of Edmonton Engineering Services Library. The following reports were reviewed:

- Slide Investigation, 199 Street and Wedgewood Creek, Edmonton, Alberta, Prepared by: Thurber Engineering, File No. 14-31-70, May 30, 1990.

- Slope Stability Assessment, Proposed Edgemont Neighborhood, North and West Bank of Wedgewood Creek, 215 Street and 35 Avenue, Edmonton, Alberta, prepared by Hoggan Engineering & Testing (1980) Ltd., File No. 6004-22, August 4, 2011.

Report 1 was completed on a failure of the west embankment fill of the existing 199th Street at WWC. The failure was noted to be shallow and not deep seated. It was determined that the failure occurred due to the buildup of water at the inlet of the culvert. The buildup of water occurred due to the beaver dam limiting the flow of water downstream. The observations indicated that the toe of the side slope became saturated leading to its failure. The report provided observations and recommendations for the repair of the side slope failure. No evidence of this past failure was noted during our site reconnaissance or the air photo review.

Report 2 completed by Hoggan was a slope stability analysis of the north and west banks of the Wedgewood Creek as part of the Edgemont Neighborhood Development. The slope assessment did not include the assessment of 199th Street side slopes at the WWC.

Aerial Photograph Review

Several sets of aerial photography taken between 1924 and 2014, covering the subject site and surrounding areas, were obtained from the City of Edmonton Mapping Department, the Alberta Sustainable Resource Development Library and Google Earth. The photos were reviewed to identify any signs of disturbances within the site.

The photo coverage obtained is summarized as follows:

Year	**Catalogue No.**	**Photo No.**	**Scale**
2004 – 2014	Google Earth	---------	Approximately 1:5000
2001	ED 2001-01	138 and 139	Approximately 1:20000
1993	AS 4383	208 and 209	Approximately 1:20000
1974	AS 1313	220	Approximately 1:12000
1962	AS 818	15	Approximately 1:31680
1949	AS 136	58 and 59	Approximately 1:40000
1924	C.ARS	35	Oblique

In1924, 199th Street did not cross the WWC at its existing location. It crossed the WWC to the west of its current crossing location. The road ends at 35th Avenue and then heads south roughly 160 meters west of the existing location and winds through the WWC. The road then follows its current alignment approximately 300 meters south of WWC. Little to no development with the exception of two farm houses was noted along the 199th Street to the south and north of the WWC. In 1949, road followed the same pattern and no observable changes to the road were noted. In the 1962 Air Photo, 199th Street appears to follow its current alignment and crosses WWC at its current crossing location. In the 1974 Air Photo, the road appears to be wider and appears to have been paved. Woodbend Wynd along with the subdivision development appears to the southeast of the WWC and 199th Street intersection. Several farm residences are noted to the north of WWC on the east and west sides of 199th Street. In the Air photos from 1993 Photos to the summer of 2014, no changes to the current road alignment from that of the 1974 Photo was noted. In the summer of 2014, 199th Street appears to have been removed due to the construction of underground utilities from 35th Avenue to the north edge of the WWC. Development of the Edgemont Subdivision is noted in the 2012 photos on Google Earth.

It should be noted that the failure noted in the 1990 Thurber Report could not be seen in any of the observed Air Photos. No slope stability concerns with the side slopes of 199th Street at the WWC were noted on the observed photos.

Geology

The geology of the site starts with the deposition of the bedrock soils in shallow seas present during the Cretaceous period. Clayey sandstone, shale, and bentonitic mudstone were formed at the bottom of these seas and are termed the Horseshoe Canyon Formation of the Edmonton Group. Long after the bedrock formation, a river flowed through the Edmonton area which also had several significant tributaries. Deep granular deposits termed Saskatchewan sands and gravels were formed in this river. This river was not the North Saskatchewan River as this flowed after the ice age came and went. However, it is noted that none of the deep testholes in this study encountered the bedrock or Saskatchewan sands and gravel formations.

The next major geologic event was the several advances of large ice sheets across most of North America. These large ice sheets plowed along the bedrock, then deposited a mixture of clay, silt and sand during their retreat, termed glacial clay till. A large lake formed over much of Edmonton near the end of the ice retreat. This lake deposited clay and silt soils, termed Lake Edmonton deposits.

On the west edge of the Lake Edmonton lacustrine deposits, aeolian (wind) deposits consisting of sand and silt were formed.

3.0 FIELD INVESTIGATION

The soils investigation for this project was undertaken on March 18 & 19, 2015 and April 8, 2015 utilizing a truck mounted drill rig owned and operated by SPT Drilling Ltd. of St. Albert, Alberta. Five testholes were drilled at locations shown on the attached site plan. The testholes were advanced to depths in the range of 14.9 and 26.7 meters below ground surface (BGS). The testhole layout was selected by Hoggan Engineering and Testing (1980) Ltd. (Hoggan) prior to drilling and the testholes were surveyed for location and elevation by Hoggan following drilling. The locations of the testholes were situated to avoid intersections and existing utilities. The testholes were drilled within the WWC crossing area. Drilling within the ditches was not possible due to the soft conditions, steep side slopes and power lines.

The testholes were advanced with 150 millimeter diameter solid stem augers in 1.5 meter increments in all of the testholes and probeholes. A continuous visual description, which included the soil types, depths, moisture, transitions, and other pertinent observations, was recorded on site. Disturbed samples were removed from the auger cuttings at 750 millimeter intervals for laboratory testing. Standard Penetration Tests c/w split spoon sampling was also taken at regular 1.5 meter intervals.

Following the drilling operation, slotted piezometric standpipes were inserted into all testholes for watertable level determination. The testholes were backfilled with cuttings, with bentonitic seals placed at the surface. Watertable readings were obtained between 12 to 13 days, 21 to 22 days and 27 to 28 after completion of drilling.

An additional probehole and standpipe was installed near Testhole 2015-02 in order to confirm the watertable readings in that testhole.

4.0 LABORATORY TESTING

All disturbed bag samples returned to the laboratory were tested for moisture content. In addition, the plastic and liquid Atterberg Limits and soluble soil sulphate concentrations were determined on selected samples. A grain size analysis was conducted on selected coarse grained samples. The Shelby Tube samples were tested for unconfined compressive strength and dry density. Lab results are included on the attached testhole logs located in Appendix I.

5.0 SOIL CONDITIONS

A detailed description of the soils encountered is found on the attached testhole logs in Appendix I. In general, the soil conditions at this site consisted of surficial asphalt and gravel underlain by clay fill, overlaying sand and/or lacustrine high plastic clay underlain by silt. The final soil encountered in the testholes was clay till.

Hot mix asphalt was noted at the surface of all testholes drilled at road surface. The asphalt was measured between 80 to 150 millimeters thick. Below the asphalt, moist, brown, well graded, dense, gravel was encountered to depths in the range of 450 and 700 millimeters BGS. The asphalt and gravel thicknesses are known at testhole locations only and may vary in between.

Fill was encountered below the asphalt in Testholes 2015-01 to 2015-04. The clay fill was typically, moist, very stiff, and medium to high plastic in nature and featured trace organics. The clay fill featured traces of coal, oxides, and pebbles throughout. In addition, the clay fill featured sandier areas within the deeper fill at the WWC crossing. The clay fill was encountered to depths in the range of 2.0 to 11.4 meters BGS in the testholes. In Testholes 2015-03 and 2015-04, an organic layer, approximately 0.1 to 0.8 meters thick was noted at the transition of the clay fill to the native clays. As mentioned previously, testhole drilling was not possible in the ditches; hence organic depths may vary away from the road.

Below the clay fill in Testhole 2015-02, silty sand was encountered. The sand was typically brown in colour and very moist to wet and compact in nature. The sand was encountered to a depth of approximately 8.8 meters BGS. Also, below the clay till in Testhole 2015-02 at a depth of approximately 19.0 meters BGS, a wet sand layer was encountered. This sand layer was generally wet, gravelly and featured traces of shale chips. The sand layer was encountered to testhole termination depth of approximately 21.0 meters BGS in Testhole 2015-02.

Below the clay fill in Testholes 2015-01, 2015-04 and 2015-05 and below the sand in Testhole 2015-02, lacustrine clay was encountered. The clay was typically moist and very stiff near the surface and became very moist to wet, medium plastic and firm to soft roughly 2 to 3 meters into the layer. The lacustrine clay transitioned into a very moist to wet clayey, sandy silt with increased depth. The silt was grey in colour, low to medium plastic in nature and was typically very soft, saturated and sensitive in nature. The clay and silt was encountered to depths in the range of 10.2 to 14.0 meters BGS.

Below the lacustrine clay and silts in Testholes 2015-01, 2015-02, 2015-04 and 2015-05 and the clay fill and organics in Testhole 2015-03, silty, sandy, glacial clay till was encountered. The clay till was typically moist with very moist areas and featured traces of coal, oxides, pebbles and the occasional sand lens or seam. The clay till was generally medium plastic in nature with a stiff to very stiff consistency. The clay till was encountered to testhole termination depths of 14.5 meters BGS in Testholes 2015-01, 2015-04 and 2015-05 and termination depth of 26.7 meters BGS in Testhole 2015-03 and to a depth of approximately 19.0 meters BGS in Testhole 2015-02.

During drilling, free water and slough were encountered in most of the testholes. See table in the next section for summary of free water and slough levels in each testhole at completion of drilling.

6.0 GROUNDWATER CONDITIONS

The groundwater table within the study area was generally moderate to low throughout the project area. The water table varied between 3.8 and 8.2 meters BGS. Three sets of watertable readings were taken, with the results shown in the table below.

Groundwater Table Readings
Proposed 199 Street Upgrades - Stage 1
(Metres Below Ground Surface)

Testhole	Elevation	Conditions At Testhole Completion	12 to 13 Day 30-Mar-15	21 to 22 Day 8-Apr-15	27 to 28 Day 14-Apr-15	Watertable Elevation
2015-01	683.04	5.2m water, 5.2m slough	5.73	5.60	5.65	677.39
2015-02	681.05	8.5m water, 4.3m slough	3.75	3.75	3.80	677.25
2015-03	680.88	16.3m water, 2.6m slough	9.21	7.82	7.79	673.09
2015-04	682.37	No water, No slough	5.85	5.75	5.85	676.52
2015-05	687.17	4.3m water, No slough	8.09	8.16	8.09	679.08

It should be noted that water table levels may fluctuate on a seasonal or yearly basis with the highest readings obtained in the spring or after periods of heavy rainfall. The above readings would be near the average seasonal levels.

The water level in Testhole 2015-02 indicated that the groundwater level is in the clay fill zone. This seemed peculiar. The standpipe was pumped from the water and the water level readings were further observed to be at the same level. Therefore, in order to confirm this reading, a second testhole was drilled next to Testhole 2015-02 in order to isolate the watertable within the sand. The watertable reading in the second testhole indicated a ground water level reading of approximately 5.6 meters BGS, within the native sand layer. Given that the higher groundwater level reading of 3.8 meters BGS has more of an adverse effect on the development, the higher reading was used in all of our analysis.

7.0 RECOMMENDATIONS

7.1 Underground Utilities

7.1.1 Open Excavation

1. The clay fill, upper clay, upper sand and clay till materials encountered in the testholes are considered fair to satisfactory while the lower clay, silt and sand material would be considered poor for the installation of underground utilities incorporating the City of Edmonton backfilling and compaction requirements. The clay fill, upper sands and clays, and clay till were near to slightly above optimum moisture content, while the lower silty clays, sands and silts were well in excess of optimum moisture content. Topsoil and other organic materials are not considered suitable for backfill material. The design sewer depths should minimize the cuts as much as possible due to the soft sensitive soils with depth.
2. Although the watertable was moderate to low between 3.8 and 8.2 meters BGS in the testholes, it would be considered high to moderate considering the proposed utility depths of approximately 6 to 12 meters BGS for this project. Saturated soil conditions, sloughing and ingressing groundwater will likely be encountered in most of the trenches at this site. The amount of ingressing water and sloughing conditions is dependent on the depth of utility design elevation compared to the water table. The amount of groundwater infiltration is expected to be slight to significant in the clay, clay till and silt areas and increased in the sand areas and will depend on the watertable versus trench depth at any given location. Temporary dewatering measures will likely be required during utility installation. Pumping from the trenches during installation should be sufficient to maintain trench working conditions in most areas. However, well points are a slight possibility in deeper trench locations. Delays in construction will likely occur in some locations. Weather conditions

will also have a significant bearing on site operations, with rain potentially causing significant problems in areas of open trenches due to the sand soils. Opening relatively long portions of utility trench is not recommended for this site.

3. Standard trenching cutback angles of approximately 45 degrees from the vertical are anticipated for the clay fill soil and some of the upper clay soils, and the lower clay till although some portions of the moister clays, silts and saturated sand seams will likely require increased cutback angles of more than 45 degrees or more in order to remain stable, due to their low strength and elevated moisture contents. Shoring of deeper trenches may be required (only for major sloughing). Actual cutback angles should be determined in the field during construction. Exact stable slope values cannot be pinpointed without detailed and extensive analysis. For this reason, this information should be used as a guideline only and that the optimum cutback angles for utility trenches be determined in the field during construction. The Occupational Health and Safety Code, Part 32 Excavating and Tunnelling should be strictly followed, except were superseded by this report.
4. Trench widths should be compatible with safe construction operations. The trench width must be wide enough to accommodate pipe bedding and compaction equipment.
5. Temporary surcharge loads, such as spill piles, should not be allowed to within 3.0 meters of an unsupported excavation face, while mobile vehicles should be kept back at least 1.0 meter. All excavations should be checked regularly for signs of sloughing or failures, especially after rainfall periods.
6. Pipe bedding and trench backfill procedures should adhere to the City of Edmonton specifications as outlined in The Servicing Standards manual. The backfill material beneath and above the pipe should be an approved bedding sand material where conditions allow. This material should be hand placed and hand tamped, with care taken to fill the underside of the pipe. The City of Edmonton trench bedding types are available in their specifications and are considered suitable. However, ingressing groundwater was encountered in many of the testholes around the site. To overcome the installation difficulties which may be encountered where ingressing groundwater and/or poor bearing conditions may be a problem, it is recommended that a washed rock and geotextile separator be utilized for pipe bedding in these areas. The washed rock and geotextile configuration should be determined

in the field during construction. The need for this configuration may be considerable at this site.

7. The moisture content of the clay fill was typically moist and near optimum moisture content. Minor moisture conditioning is anticipated for the existing clay fills encountered in the testholes. The moisture content of the silty clays in the testholes was variable, but was generally moist to very moist and wet with increased depth. The sand was typically dry to damp above the ground water table and very moist to wet below the ground water level. The clayey silts were typically wet and saturated with increased depth. The variable condition of the soils will cause a corresponding variability in the utility trench pipe bedding and backfill conditions. Some occasional wetting or drying will likely be required at this site to meet the moisture content criteria and adequately construct a platform for surface utility construction. The higher plastic clay materials should be moisture conditioned to a minimum of 1 percent over optimum moisture content (equal to approximately 3 percent above plastic limit) to help reduce swelling. Trenching operations may be slowed down due to the required moisture conditioning. Failure to adequately moisture condition the trench backfill may result in swelling or subgrade softening of the trench backfill. In occasional moister areas, drying or mixing of the backfill prior to placement in the trench will be required when adequate compaction cannot be achieved at the natural moisture content.

8. The majority of native inorganic soils and clay fill encountered in the testholes within the noted project area geotechnical investigation will meet the minimum 72 kPa allowable bearing capacity required by EPCOR for thrust block standard design. However, a portion of the native soils encountered will have an allowable soil bearing capacity that falls below the minimum 72 kPa. In the area of these testholes, thrust block designs should be modified to accommodate a design allowable bearing capacity of 50 kPa. The chart below depicts the testholes and their respective recommended bearing capacity at each individual testhole location. Engineered fill should have an allowable bearing capacity above 72 kPa for thrust block design.

   It is emphasized that soil conditions may vary away from the testhole locations. All thrust block excavation should be inspected to confirm the bearing capacity during construction prior to placement of concrete.

Watermain Thrust Blocks - Recommended Soil Bearing Values

Proposed 199 Street Upgrades - Stage 1

Testhole	Depth (m)	Allowable Bearing Capacity (kPa)	Testhole	Depth (m)	Allowable Bearing Capacity (kPa)
2015-01	0-13.1	50	2015-04	0-9.0	50
	13.1-14.9	Minimum 72		9.0-14.9	Minimum 72
2015-02	0-21.0	Minimum 72	2015-05	0-7.0	50
2015-03	0-26.7	Minimum 72		7.0-14.9	Minimum 72

9. Trench compaction requirements of the City of Edmonton are 100 percent of the One-Point Proctor Density above a depth of 1.5 meters, and 97 percent of the One-Point Proctor Density below this level. The maximum lift thickness is 300 millimeters. This degree of compaction should be achievable with occasional mixing or moisture conditioning of the trench backfill in portions of the trench as mentioned.

10. It should be noted that the ultimate performance of the trench backfill is directly related to the consistency and uniformity of the backfill compaction, as well as the underground contractors construction procedures. In order to achieve this uniformity, the lift thickness and compaction criteria should be strictly enforced.

Trench Backfill Maximum Moisture Content Criteria
Proposed 199 Street Upgrades - Stage 1

Testhole Number	Sample Depth	Liquid Limit	Plastic Limit	Field Moisture Content	Plasticity Index (PI)	Maximum Moisture Content Criteria							
						Uniform Backfill			Conventional Backfill			PL+10 Criteria	
						PI/2	PL+PI/2	+/- Criteria	PI/3	PL+PI/3	+/- Criteria	PL+10	+/- Criteria
2015-01	0.6 m	42.1	20.8	16.3	21.3	10.7	31.5	-15.2	7.1	27.9	-11.6	30.8	-14.5
2015-01	9.1 m	26.5	23.3	30.4	3.2	1.6	24.9	5.5	1.1	24.4	6.0	33.3	-2.9
2015-01	9.4 m	26.0	21.6	30.2	4.4	2.2	23.8	6.4	1.5	23.1	7.1	31.6	-1.4
2015-02	1.5 m	46.5	12.3	19.2	34.2	17.1	29.4	-10.2	11.4	23.7	-4.5	**22.3**	-3.1
2015-02	5.3 m	39.4	11.3	19.8	28.1	14.1	25.4	-5.6	9.4	20.7	-0.9	21.3	-1.5
2015-02	16.0 m	31.9	12.1	15.9	19.8	9.9	22.0	-6.1	6.6	18.7	-2.8	22.1	-6.2
2015-03	6.9 m	41.0	12.4	23.6	28.6	14.3	26.7	-3.1	9.5	21.9	1.7	22.4	1.2
2015-03	5.4 m	50.8	15.9	21.6	34.9	17.5	33.4	-11.8	11.6	27.5	-5.9	**25.9**	-4.3
2015-03	11.0 m	21.0	14.2	12.3	6.8	3.4	17.6	-5.3	2.3	16.5	-4.2	24.2	-11.9
2015-03	23.5 m	29.2	15.2	20.0	14.0	7.0	22.2	-2.2	4.7	19.9	0.1	25.2	-5.2
2015-04	3.7 m	58.9	16.1	24.8	42.8	21.4	37.5	-12.7	14.3	30.4	-5.6	**26.1**	-1.3
2015-04	8.4 m	28.5	21.0	30.4	7.5	3.8	24.8	5.7	2.5	23.5	6.9	31.0	-0.6
2015-05	1.5 m	49.9	15.1	30.0	34.8	17.4	32.5	-2.5	11.6	26.7	3.3	**25.1**	4.9

Notes:
- City specifications state that when the plasticity index criteria for maximum moisture content exceeds 10 percent over the plastic limit, the plastic limit plus 10 percent shall govern.
- All values are percentages.
- Bold values of PL+10 are governing criteria.
- Chart shows only the samples which were tested for Atterberg Limits. See testhole logs for all moisture content data.

7.1.2 Trenchless Installation

1. It is understood that trenchless installation may be utilized as the method of construction of the deep underground utilities, especially under WWC. The trenchless method to be used should be determined by the underground contractor.
2. Trenchless installation through the site clay fill, clay, and clay till soils will be considered fair to satisfactory while installation through the sand encountered in Testhole 2015-02 and the lower silt is considered fair. The sand and silt are susceptible to sloughing and squeezing, especially under the water table, as these soils are sensitive to disturbance. The mud composition may need alteration during installation to account for the variable soil conditions. Installation delays may occur due to the variable nature of the site soils.
3. Trenchless installation in the clay till soils encountered in the testholes may encounter some difficulties due to wet sand and gravel lenses and potential cobble and boulders, as the soil is a glacial deposit.
4. Exact potential for “frac-out” is difficult to determine, but it is generally considered low in the clay, silt and clay tills and moderate to high in the clay fill and sands soils. As a minimum, the contractor should review soil conditions on a continuous basis and take proper measures to prevent “frac-out” from occurring. An emergency “frac-out” response plan and contingency crossing plan that outline the protocol to monitor, contain and clean-up a potential “frac-out” should be in place prior to construction.
5. It is recommended that the drilling contractor follow standard horizontal directional drilling (HDD) practices. Such HDD practices can be found in “Horizontal Directional Drilling Good Practice Guidelines, Third Edition” as recommended by North American Society of Trenchless Technologies.

7.2 Surface Utilities

7.2.1 General Road Construction

1. The subsurface inorganic soil conditions encountered are considered generally fair to poor for the construction of roads, curbs, and sidewalks. Topsoil and all other deleterious materials along the road alignment should be removed prior to construction of the embankment across the ravine.
2. A main concern for surface utility construction at this site is the elevated moisture content of the lower silty clay, silt and sand materials. The near surface clay and clay fill is medium to

high plastic and was slightly above its optimum moisture content, but mixing and disturbance during underground utility installation will degrade the soil conditions. Extra subgrade work beyond standard scarification and re-compaction and cement stabilization may be required in order to construct an adequate working platform for the pavement structure placement and long term support. It is noted that the degree of trench backfill drying during underground utility installation affects the soil conditions for road and sidewalk construction, with increased drying improving the soil conditions.

3. The near surface site clays and clay fill are of low to moderate frost susceptibility, with the susceptibility becoming higher in the sands, silts and silty clay soils encountered at depth. A high watertable within approximately 3.0 meters of the road surface is required for significant frost heaving to occur. The closer the watertable is to the surface, the higher is the frost heave potential. The standpipes for this project have stabilized below this level, between 3.8 and 8.2 meters BGS, and as such, no frost heave concerns are foreseen, provided significant cuts are not made. For frost protection measure, the sand, silt and very silty low plastic clay backfill should be kept 1.5 meters or more below the subgrade.

4. Cement stabilization is the recommended minimum subgrade treatment for this site. For stiff clay subgrade, minimum 10 kilograms of cement per square meter of subgrade should be mixed to a depth of 150 millimeters, and re-compacted to 100 percent of Standard Proctor Density (SPD) near optimum moisture content. For soft to firm clay subgrade, 20 to 30 kilograms of cement per square meter of subgrade mixed to a depth of 300 millimeters would be required. The exact cement content and depths should be decided in the field based on a proof roll. Weather and time of year will also be factors.

   The subgrade should be inspected and proof rolled by qualified personnel after final compaction and any areas showing visible deflections should be repaired prior to paving.

5. If drying is not possible and cement stabilization fails to produce an adequate subgrade, replacing the subgrade with a gravel sub-base would be applicable. A pit-run gravel sub-base, 600 to 900 millimeters thick placed over a woven geotextile (Nilex 2006 or equivalent) is estimated for this purpose. The need for this sub-base should be low, but should be budgeted as a contingency for poor weather. The extent of subgrade

replacement should be decided on site during construction. The need for this measure is anticipated to be low at this site.

6. Surface water will often collect within the granular base, causing subgrade softening and pavement damage. Therefore, it is recommended that wic drains to be installed in the gravel road base at the curb bottom locations. The wic drains must be properly attached to the catch basins. Good drainage within the gravel base is imperative for lasting structural performance. The overall cross slope of the road subgrade should be as least 2 percent towards the wic drain connected to catch basins. Care must be taken not to allow any excess moisture into these soils.
7. It is recommended that all areas beyond the back of curb/sidewalk be landscaped as soon as possible to avoid water permeating into the subgrade from free standing puddles. The near surface clay soils encountered in some of the testholes throughout this area exhibit a moderate to high swelling potential. It is important that subgrade soils not be allowed to dry excessively when exposed, and moisture contents are kept slightly over optimum.
8. It is understood 199 Street will be a four lane divided arterial road. An estimated traffic volume of 35,750 vehicles per day in 2047 was found in the following report.

    - Riverview Neighbourhoods 1, 2 & 3, Neighbourhood Structure Plan, Transportation Impact Assessment, dated November 17, 2014, prepared by Bunt & Associates, file # 3366.03

It was assumed that trucks account for 7 percent of the traffic, with an aggregate truck factor of 1.2, a growth rate of 3 percent per year, as well as a design life of 20 years. Based on the above assumptions, the total traffic loading was estimated to be approximately 2.9 x 10^6 ESALs. Based on an estimated California Bearing Ratio (CBR) of 3.0 percent, the following staged pavement design is recommended for this site.

Recommended Staged Roadway Structures
Proposed 199 Street Upgrades - Stage 1

	Traffic Loading	Arterial ($2.9x10^6$ ESALs)
Stage 1	Asphaltic Concrete (10mm-HT) Asphaltic Concrete (20mm-B) Crushed Gravel (3-20 or 3-63)	45 mm 100 mm 350 mm
Stage 2	Asphaltic Concrete (10mm-HT)	50 mm
Note:	10mm-HT = City of Edmonton Designation Asphaltic Concrete 10mm-Heavy Traffic 10mm-B = City of Edmonton Designation Asphaltic Concrete 20mm-Base 3-20 = City of Edmonton Designation 3 Class 20 aggregate All granular base material should be compacted to 100 percent of the Standard Proctor Density in maximum 150 mm lifts.	

Our firm should be advised if updated traffic loading information becomes available and the pavement design should be modified accordingly.

9. At the connections between the old and new pavements, the new subgrade should be tapered to match the existing subgrade to ensure even drainage within the gravel bases.

7.2.2 Embankment Construction

1. Grading plans were forwarded to our firm and they indicate that no significant cuts are planned for this area. The new road grades will match the existing grades.
2. It is understood the existing embankment across the WWC ravine will be widened. The recommended construction method for embankment widening is to remove the existing embankment side slopes in a step fashion. The side slopes should be benched in order to obtain bonding between the existing grade and the new embankment. Proper organic stripping is a must as well.
3. In order to widen the embankments slopes, the creek will require dewatering. This can be achieved by construction of a clay dam and pump system and/or diversion of the creek. Any organic soils encountered at creek bottom will have to be removed. Our firm should inspect the fill areas in order to ensure that all unsuitable materials are removed.
4. The excavation of the existing side slopes in preparation for the proposed widening may expose the soft lower clay near the bottom of the creek. Construction traffic may encounter difficulties travelling on this surface. A clay pad 600 millimeters thick may be required in soft soil areas to allow for grading construction equipment to operate.

Judgment should be used in the field at the time of construction to determine an initial lift thickness.

5. The embankment fill material must be cohesive and non-organic to ensure a positive bonding to the existing grade surface and provide erosion resistance. The source of the embankment fill is not known at this time. It is recommended that the clay fill consist of a medium to high plastic clay material as these soils will have a low susceptibility to erosion. The import clay should be approved by JRP prior to use. All grading fill within the embankment should be compacted to a minimum 98 percent of Standard Proctor Density (SPD). All fill should be placed and compacted in maximum thickness lifts of 150 millimeters.
6. The stability analysis included assessment of end of construction (short term) condition based on effective stress analyses with construction generated excess pore pressures as well as long term stability after pore pressure dissipation. Pore pressures generated in the embankment fill and in the underlying native clay till layer during fill placement and compaction have been estimated based on B-bar value of 0.3.

   The desired minimum side slope for the embankment at the proposed creek to minimize the environmental impact on the WWC is 2H:1V. Global stability analysis on the side slopes based on the proposed 2H:1V indicated a non-stable slope. Therefore, the slope will require slope stabilization measures. Global stability analysis on the reinforced side slopes was completed. Based on the results of the analysis, the stabilization measures should consist of placing a bi-axial geo-grid (Tensar BX1200 or similar) at the interface of the native in-situ clay tills and the first engineered fill layer, and then utilizing a uni-axial geo-grid (Tensar UX1100 or similar) every one meter of fill placement after that. The analysis indicated stable side slopes once the reinforcement is applied within the compacted clays. The reinforcement should extend transversely with the roadway underneath the entire approach ramp footprint at the top three meters below top of subgrade and a minimum 20 meters from the edge of the side slopes below three meters below the subgrade. This reinforcement configuration is shown on the slope stability graphics in Appendix II.
7. End of construction and long term settlement analysis was carried out using the computer program FoSSA to estimate ground settlement under the new approach fill loading over

the project. The approach fill geometry was based on the existing slope profiles at WWC provided to Hoggan by Stantec and the proposed 2H:1V side slopes. The underlying native lower clay, encountered mainly along the north facing slope of WWC, will consolidate as a result of the weight of the new fill. However, it is assumed that the ground was level with the surroundings before the ravine was created. The approximately 12 meter tall embankment would bring the grade back to the original level before the ravine was present. Therefore, the loading pressure from the embankment should be below the pre-consolidation pressure. The clay till is very stiff and moist in nature and is considered over consolidated and should not settle significantly with the additional fills. Settlement of the existing fill below the new fill is considered negligible.

No consolidation test was performed for this site. Based on our knowledge and experience of the lacustrine clay material, consolidation parameters (recompression index, C_r = 0.02, compression index C_c = 0.2, initial void ratio = 0.8) were assumed in the analysis. The analysis showed the maximum total consolidation of the underlying native clay soil would be approximately 0.2 meters, at the point of highest new fill.

It was also estimated the self-weight settlement of future embankment fill will be approximately 1 percent of the fill height (0.12 meters) and should take two to five years to occur.

It should also be noted that the settlement will not be even and will vary with fill height. This unevenness should be accounted for in the design, construction and future maintenance of the project, including the proposed underground utilities.

8. Runoff near the ravine crossing should be intercepted and directed to erosion protected channels or storm sewer. The finished embankment side slope should be covered with vegetation as soon as possible for erosion protection.

7.2.3 Culvert Installation

1. The soils encountered at the culvert elevation consisted of clay till and is considered suitable for a culvert installation. The design and installation of the culvert should be done in accordance with the City of Edmonton Specifications, except where superseded by this report.
2. Topsoil, clay fill, and organic soils should be completely removed from the culvert base area, including below the side backfill. The depth of the culvert subgrade will be below

the groundwater table; therefore a temporary dewatering system will be required. This system would likely consist of a perimeter ditch draining to a sump area away from the culvert base. Dewatering measures are best determined onsite during construction. The proper compaction of the culvert granular base may not be achievable without the dewatering.

3. The culvert excavations should be performed by a backhoe operating remote from the bearing surface, due to the watertable. The depth of the excavation should be sufficient for the pipe to lie in the native clay till material. The standard minimum subcut of 0.6 meters below the culvert inlet will be adequate for this site. The width of the excavation should be the greater of 2 pipe spans or 1 pipe span plus 3.0 meters. The excavation should extend longitudinally from the inlet to the outlet.
4. Backfill will be defined as either structural backfill, which is material placed in the critical zone around the pipe in accordance with the City of Edmonton specifications, or embankment fill, which is material placed beyond the structural backfill envelope.
5. Standard trenching cutback angles of approximately 45 degrees from the vertical are anticipated for the site, although some portions of the moister clays, lower very moist to wet clays may require increased cutback angles of more than 45 degrees in order to remain stable, due to their low strength and elevated moisture contents. Actual cutback angles should be determined in the field during construction. Exact stable slope values cannot be pinpointed without detailed and extensive analysis. For this reason, this information should be used as a guideline only and that the optimum cutback angles for utility trenches should be determined in the field during construction. The Occupational Health and Safety Act, Part 32 Excavations and Tunnelling should be strictly followed, except were superseded by this report.

6. All structural backfill material should be comprised of granular material. The placement of a non-woven geotextile separator between the subcut floor and the first lift of structural fill is recommended for this project. Placement of the fabric should be done in accordance with the supplier's instructions. An initial lift of 450 millimeters of lightly compacted structural fill may need to be placed in order to achieve an adequate bridge above the clay subsoil. All subsequent lifts should be compacted to a minimum of 98 percent of Standard Proctor Density in maximum 150 millimeter lifts, at optimum

moisture content. Compaction in the haunch areas should be done manually in 100 millimeter lifts, and should still meet the above compaction requirement.

7. Exceptions to this compaction requirement are recommended for the 150 millimeter lift immediately below the bottom of the pipe (the bedding material), and the 300 millimeters of material immediately above the top of the pipe. These areas should have minimal compaction. The bedding material should be pre-shaped to the bottom of the pipe. The bedding shall be omitted in the clay seal areas. The 300 millimeters of material over the top of the pipe should be placed and compacted without vibration. Material above this level should meet the above compaction requirements.
8. In regards to settlement, the proposed culvert should be founded on native clay till soils. No significant consolidation settlement of the native clay till soil is expected. No significant heave should occur. If the gravel pad below the culvert is placed in thick lifts some settlement may occur once the base is reloaded, rough estimates of settlement are up to 25 millimeters.
9. Clay seepage cut-offs are recommended at both the inlet and outlet of the culvert structure. These may be eliminated from the culvert extensions where cutoffs already exist. The length of seal should be equal to 2 times the diameter of the culvert (as measured at the invert of the pipe). The length of the seal may be reduced when using large diameter culverts. The clay should be compacted to a minimum of 98 percent of Standard Proctor Density in maximum 150 millimeter lifts. This includes the area below the invert. The shape of the cutoffs will be as defined in the City of Edmonton specifications.
10. Compaction of both the structural fill and clay seal fill shall be by equipment moving parallel to the longitudinal axis of the pipe. Above 300 millimeters above the top of the pipe, the equipment should operate perpendicular to the longitudinal axis of the pipe. Backfill should progress simultaneously on both sides of the pipe. Backfill on one side of the pipe should not exceed the other by more than 300 millimeters. Care must be taken to ensure that no deflections in the pipe are caused by the backfill procedures. It is recommended that the rise and span of the pipe be measured at the center and 1/4 distances from each end during construction.

11. As requested, samples of soil and water were retained for resistivity and pH testing, as well as for the presence of sulphates, chlorides, and other salts. All samples were submitted to ALS Laboratories for testing. The results are as follows:

Soil Corrosion Testing Results

	Soil Samples		
Property	**Clay Fill 2015-02 @ 8.2 m**	**Clay Till 2015-02 @ 12.9 m**	**Water Sample WWC**
pH	7.94	8.21	8.09
Conductivity (paste)	0.356 dS/m	0.631 dS/m	586 uS/cm

7.3 Bridge Foundation

Part of the 199th Street Upgrades is the construction of a wildlife passage. It is understood from Stantec that the wildlife crossing will consist of a roughly 15 meter long single span bridge. The bridge construction will allow for roughly 4.5 meters of head space for the animals. In addition, MSE wing walls will be constructed on the side slopes. Testholes 2015-02 and 2015-03 were drilled at the proposed bridge location. A Sketch of the preliminary bridge design is available in Appendix III.

The following recommendations are provided to aid in the design and construction of the bridge.

7.3.1 Cast-in-Place Piles

1. The soils encountered at this site are suitable for a cast-in-place pile foundation. The structure may be founded on an adequately reinforced grade beam or pile cap supported by bored, cast-in-place, concrete piles. The design capacity can be calculated on the basis of factored skin friction or end bearing values. A combination of the two bearing modes may be utilized for individual piles.
2. The factored skin friction values that may be used are as follows:

Testhole 2015-02:

Soil Stratum	Ultimate Skin Friction Resistance	Geotechnical Resistance Factor	Factored Skin Friction Resistance
New Clay Fill (Top 1.5m)	0 kPa	0.4	0 kPa
New Clay Fill (Below 1.5m)	60 kPa	0.4	24 kPa
Existing Clay Fill	55 kPa	0.4	22 kPa
Sand	42 kPa	0.4	17 kPa
Clay Till*	90 kPa	0.4	36 kPa
Clay Till**	100 kPa	0.4	40 kPa

* (from Elevation 670.5 to 660.0 m)
** (below Elevation 660.0 m)

Testhole 2015-03:

Soil Stratum	Ultimate Skin Friction Resistance	Geotechnical Resistance Factor	Factored Skin Friction Resistance
New Clay Fill (Top 1.5m)	0 kPa	0.4	0 kPa
New Clay Fill (Below 1.5m)	60 kPa	0.4	24 kPa
Existing Clay Fill	55 kPa	0.4	22 kPa
Clay Till*	75 kPa	0.4	30 kPa
Clay Till**	100 kPa	0.4	40 kPa

* (from Elevation 669.0 to 662.0 m)
** (below Elevation 662.0 m)

Due to the close proximity of Testholes 2015-02 and 2015-03, the factored skin friction resistance below Elevation 660.0 for Testhole 2015-02 can be assumed to be the same as the provided factored skin friction resistance provided for Testhole 2015-03.

The above values include the total of all live and dead loads. Considering the effects of frost and seasonal moisture changes, the friction value for the first 1.5 meters of pile should not be considered in design.

3. To account for lateral load resistance, it is understood that batter cast-in-place piles may be required. Batter of cast-in-place skin friction piles are considered suitable at this site. A maximum batter angle of 1H:4V is recommended.
4. It should be noted that Serviceability Limit States (SLS) addresses the functional performance of a structure as opposed to Ultimate Limit States (ULS) which addresses failure. Therefore, the geotechnical issue for SLS loading on piles is settlement rather than

bearing capacity. While the predicted settlement of a pile is not readily calculated, the typical expectation of a structure placed on a pile foundation is essentially no settlement at all. In this case, the expected settlement for a skin friction pile loaded to the above factored bearing values would be less than 10 millimeters. Therefore, the design values provided in Item 7.3.2 are considered by the writer to be ULS and SLS values, if 10 millimeters of settlement is acceptable. It should be noted that piles in the new deep fill will have more involved settlement consideration due to the large negative skin friction/downdrag caused by the new fill. The existing fill is greater than 50 years old and is considered completely consolidated.

5. The preliminary bridge design drawing indicate that the piles along the south abutment will go through a maximum of 5.0 meters of new clay fill as part of the widening of the side slopes and the replacement of the existing culvert. Piles located in the new deep fill soils will be subjected to downdrag forces (negative skin friction) due to potential long term settlement of the new clay fill. Negative skin friction and downdrag forces generally do not affect the geotechnical/Ultimate Limit State capacity of the piles. Downdrag forces increase the pile settlement and should therefore be accounted for in the Serviceability Limit State assessment of the piles.

Downdrag forces do increase the axial load on the pile and the pile structural strength must account for this extra load.

The amount of settlement from the new fill is estimated at 1 percent of the fill height. The magnitude of the force is independent of the amount of settlement. Although, if a large amount of settlement occurs the fill could become stronger; this may increase the downdrag.

The downdrag load can be expressed as negative skin friction associated with the settling soil, and is given below.

Soil Stratum	**Negative Skin Friction Value**
New Clay Fill	-60 kPa

It should be noted that the negative skin friction is un-factored, as it essentially represents a load and not a skin friction resistance. Two loading scenarios should be considered when negative skin friction is involved. The first scenario is the normal design where the ULS factored live (transient) load and the factored dead (permanent) load are added and resisted by the geotechnical resistance of the pile. No drag load is considered in this scenario because drag load and transient load never combine. The second scenario is the factored dead (permanent) load combined with the drag load which must be resisted by the structural capacity of the pile.

Another significant design factor when addressing settling soils which cause negative skin friction is the settlement of the pile. The pile settlement will never be more than the soil settlement at the surface, and it is typically significantly less. It is a complex analysis to estimate the pile settlement when you can accurately predict the soil settlement. It is impossible when you cannot accurately estimate the soil settlement. The expected future settlement of the new side slope fill is estimated at 1 percent of the fill height. The maximum fill height for the side slopes is 12 meters, while the maximum fill height below the pile head is anticipated to be 5.0 meters.

The sensitivity of the structure to settlement is a large factor. If the structure is sensitive to movement, then the portion of the pile below the fill should be designed to withstand the drag load plus the permanent load utilizing factored resistances.

6. The recommended minimum pile depths at this site for frost uplift prevention in straight shaft piles is 6.0 meters in a non-continuously heated structure. The minimum pile diameter for all piles should be 400 millimeters, with a minimum skin friction pile spacing of 2.5 pile diameters on center. In addition, the minimum spacing between the edges of the bells at the bottom of the piles is 0.3 meters.

7. The clay till encountered in Testholes 2015-02 and 2015-03 were typically moist and very stiff in nature. The clay till encountered in the testholes is considered suitable for end-bearing below the proposed elevations as noted. The factored end-bearing values that may be used are as follows:

Testhole 2015-02

Soil Stratum	**Geotechnical Resistance Factor**	**Factored End-Bearing Resistance**
Clay Till (below Elevation 666.0m)	0.4	400 kPa

Testhole 2015-03

Soil Stratum	**Geotechnical Resistance Factor**	**Factored End-Bearing Resistance**
Clay Till (below Elevation 660.0 m)	0.4	750 kPa

The above values include the total of all live and dead loads. A combination of both skin friction and end-bearing resistance can be included in the design of end bearing piles. Shaft resistance should be neglected for the top 1.5 meters of the pile length, sides of the bell, and within one shaft diameter above the top of the bell.

End bearing piles should extend to a minimum of three bell diameters below the ground or excavation surface, and should have a minimum bell to shaft diameter ratio of 2:1 and maximum bell to shaft diameter ratio of 3:1. The bell should be fully formed in the clay till layer, with the bottom of the bell penetrating the stiff to very stiff areas below the specified elevations. The clay till encountered in the testholes may feature very sandy and gravelly zones and sand lenses, as it is in its nature. Forming a bell in the very sandy areas and in the sand lenses will be difficult. If very sandy layers or sand seams are encountered, it is recommended that the bell bottom be drilled deeper to a less sandy zone where the bell can be adequately formed.

8. All pile holes should be carefully inspected to ensure that no water or slough material is present prior to concrete placement. The ground water level stabilized at levels between 3.8 and 8.2 meters BGS. Also, significant free water and slough was encountered in the testholes. Casing of the piles will likely be required. The depth of casing is anticipated to below the depth of the creek, enough to form a seal. The pile concrete should be placed as soon as possible after the pile has been bored to minimize the volume of ingressing groundwater.
9. Some provision should be made for the possible swelling of the subsoil beneath the pile caps and the effects of frost action. This can be done by providing a void form or other provision for soil expansion beneath the grade beams and pile caps.
10. It is recommended that all piles be adequately reinforced. Concrete for all piles should be adequately vibrated.
11. All structural fill against foundation walls should be an inorganic material compacted in 150

millimeter lifts to at least 98 percent of the corresponding Standard Proctor Density at optimum moisture content.

7.3.2 Driven Piles

1. Driven piles are considered a suitable pile foundation at this site. The driven piles may be timber, pre-cast concrete, or steel H or pipe piles. All piles supporting the structure should be driven to refusal or to resistance as computed by a dynamic pile driving formula, such as the Hiley formula. The recommended maximum blow count in order to prevent pile damage for steel piles is 12 to 15 blows per 25 millimetres, although this should be confirmed after a review of the pile type, loads, and hammer data. It is recommended that all pile driving be conducted under the full-time supervision of geotechnical personnel.
2. With respect to driven piles, the preliminary design length can be calculated based on combined total/effective stress analysis. The theoretical capacity of driven steel H or pipe pile is as follows:

$$Q = r_s A_s D + r_t A_t \qquad \text{where:}$$

Q = Load on the piles (kN)
r_s = Average factored skin friction between piles and soil over applicable length (kPa)
A_s = Minimum perimeter of the pile section (m) [H piles: $A_s = 2(L+W)$; Pipe Pile $A_s=2\pi r$]
D = Effective depth of the pile embedment (m)
r_t = Factored end-bearing (kPa)
A_t = Cross-sectional area of the pile tip (m²) [plug may be assumed to form for steel piles at this site provided pile depth is a minimum 20 pile diameters]

3. The factored skin friction and end-bearing values (ULS) are given as follows. For driven piles, the end bearing and skin friction bearing modes may be combined.

Testhole 2015-02:

Soil Stratum	**Geotechnical Resistance Factor**	**Factored Skin Friction Resistance**	**Factored End-Bearing Resistance**
New Clay Fill (Top 1.5m)	0.4	0 kPa	N/A
New Clay Fill (Below 1.5m)	0.4	24 kPa	N/A
Existing Clay Fill	0.4	22 kPa	N/A
Sand	0.4	17 kPa	N/A
Clay Till*	0.4	36 kPa	400 kPa
Clay Till**	0.4	40 kPa	750 kPa

* (from Elevation 661.0 to 660.0 m)

** (below Elevation 660.0 m)

Testhole 2015-03:

Soil Stratum	**Geotechnical Resistance Factor**	**Factored Skin Friction Resistance**	**Factored End-Bearing Resistance**
New Clay Fill (Top 1.5m)	0.4	0 kPa	N/A
New Clay Fill (Below 1.5 m)	0.4	24 kPa	N/A
Existing Clay Fill	0.4	22 kPa	N/A
Clay Till*	0.4	30 kPa	N/A
Clay Till**	0.4	40 kPa	750 kPa

* (from Elevation 669.0 to 662.0 m)

** (below Elevation 662.0 m)

Due to the close proximity of Testholes 2015-02 and 2015-03, the factored skin friction resistance and end-bearing resistance capacities below Elevation 660.0 for Testhole 2015-02 can be assumed to be the same as the provided factored skin friction and end-bearing resistance capacities provided for Testhole 2015-03.

4. The driven piles will be subjected to downdrag forces (negative skin friction) due to the placement of the new fill as well as the settlement of the native clay soils. Item 7.3.1.2 should be reviewed for downdrag considerations of driven piles.
5. The actual capacity of a driven pile can only be determined accurately by a pile load test. Hoggan recommends that a wave equation formulae with a factor of safety of 2.5 be utilized for determining pile capacity at the subject site during installation. Alternatively, a pile driving analyser (PDA) may be utilized. Our firm does not have such equipment and would need to sub-consult this work. With PDA analysis, a higher resistance factor of 0.5 (FOS = 2), may be utilized.
6. The recommended minimum pile depths at this site to prevent frost uplift is 6.0 meters in a non-continuously heated structure. In the event that hard driving is encountered, guidelines for refusal criteria can be provided once the pile design and driving equipment have been finalized. Refusal criteria are directly dependent on such factors as pile size, length and wall thickness as well as the specified design load and driving energy.
7. To account for lateral load resistance, it is understood that batter piles will likely be required. A maximum batter angle of 1H:4V is recommended for driven piles.
8. Driven piles at this site may encounter low driving resistance due to strength loss as a result

of quickening of the saturated silt and sand materials. If such low resistance is encountered, the pile should be driven to within 1 meter of its anticipated design elevation and left undisturbed for a minimum of 96 hours. The pile should then be re-driven and the blow counts obtained utilized for load capacity calculation. A longer waiting period may be required for the soils to re-stabilize. This pile set-up should be accounted for in the pile installation plan.

9. The piles must be designed to withstand the bending moments caused by handling, and the design structural loads.
10. The top 1.5 meters of the pile should be neglected due to frost and seasonal moisture changes.
11. It is recommended that driving records be maintained for each pile and all adjacent pile elevations should be monitored during driving. Piles that have heaved due to the driving of adjacent piles should be re-driven. To avoid heaving problems, the spacing and driving pattern used during construction must be planned carefully.
12. The recommended minimum hammer weight for drop and single acting machines is twice the weight of the pile. The driving energy utilized for this project should be maximum $6x10^6$ Newton meters times the cross sectional area (in m^2) of the steel piles. It is recommended that our firm perform a WEAP analysis on the proposed driven steel piles to recommend pile hammer sizes and assess drivability.
13. The head of the pile should be protected by an adequate helmet. The pile head protection should be checked regularly during pile installation to ensure adequate protection is maintained.
14. The pile driving contractor should have adequate experience in driven pile installation.

7.3.3 Shallow Foundations – Wing Walls

1. Four mechanically stabilized earth (MSE) wing walls and abutment retaining walls are planned as part of the construction of the bridge. A footing foundation system is considered geotechnically satisfactory for the MSE as well as abutment retaining walls. Given the nature of the site conditions, the MSE and abutment retaining wall foundations will likely be founded on either undisturbed, native non-organic soil or the side slope clay fill. The factored bearing capacities (Ultimate Limit States) that may be used are as follows:

Soil Stratum	**Geotechnical Resistance Factor**	**Factored Bearing Resistance (Strip Footing)**	**Factored Bearing Resistance (Spread Footing)**
TOPSOIL	0.4	0 kPa	0 kPa
CLAY FILL*	0.4	100 kPa	120 kPa
SAND	0.4	150 kPa	180 kPa

*Engineered fill of the 199th Street side slopes.

These figures include the total of all live and dead loads. All footings within a continuously heated structure should have a minimum 1.5 meters frost cover, with a minimum cover of 2.5 meters for a non-continuously heated structure or exterior isolated footings. Alternatively, the MSE walls may be designed to allow for frost movement or rigid insulation.

2. It is not recommended that footings be constructed below the watertable, as this will require dewatering efforts. It is anticipated that the MSE walls will be constructed above the watertable. Therefore, it does not appear that the watertable will affect footing foundation construction, and no construction difficulties or delays are foreseen.
3. Settlement will be the main concern for the MSE and abutment retaining walls. The south walls will likely experience differential settlement due to the consolidation of the clay fill. It is estimated the self-weight settlement of future embankment fill will be approximately 1 percent of the fill height below wall and should take two to five years to occur. The depth of fill across the wall is difficult to determine but may be in the range of 5 to 7 meters or greater. The north walls should be founded in the native sands and should not experience any long term settlement, as the settlement is considered immediate. The MSE and abutment walls should be designed to account for differential settlement.
4. Care should be taken during construction and the life of the structure to prevent excessive changes in moisture content of the material. Footing excavations should be protected from drying, rain, snow, freezing, and the ingress of groundwater.
5. No loose, disturbed, remoulded or slough material should be allowed to remain in the open footing excavations. Hand cleaning is advised if an acceptable surface cannot be prepared by mechanical equipment. Excavations should be dug with equipment operating remote from the bearing surface.

7.4 Lateral Loads

1. Due to the nature of this project, lateral load information may be required. A coefficient of horizontal subgrade reaction may be applied to the analysis of soil resistance for laterally loaded piles according to the following:

Soil Stratum	**Coefficient of Lateral Subgrade Reaction (kN/m^3)**
Clay Fill (Top 1.5m)	0
Clay Fill (Below 1.5m)	7,000/d
Sand	7,350/d
Clay Till	11,000/d

(where d = diameter of the pile in metres)

2. For design purposes, the top 1.5 meters of pile length should be disregarded. Additional lateral load information can be provided once pile dimensions have been chosen and the pile stiffness becomes known.
3. The horizontal modulus of subgrade reaction applies to an individual pile or a pile in a group where the pile spacing is greater than about 7 diameters (or flange widths) center to center spacing. For closely spaced piles in groups, there will be interaction between piles and the lateral support to each pile will be reduced accordingly. Pile group interaction may be modelled by applying group reduction factors to the modulus of horizontal subgrade reaction. The group reduction factor will depend on the location of the pile within the group, the least reduction being applied to lead (front) row piles. Group reduction factors are presented in the table below as a function of pile row and the pile spacing to diameter ratio.

Group Reduction Factors for Modulus of Horizontal Subgrade Reaction

(Rollins et al, 2006)

Ratio of Pile Spacing to Pile Diameter (or Width)	**Group Reduction Factors for Modulus of Horizontal Subgrade Reaction**		
	Leading Row Piles	**Second Row Piles**	**Third and Higher Row Piles**
2.5	0.74	0.48	0.30
3.0	0.79	0.57	0.41
4.0	0.86	0.72	0.58
5.0	0.92	0.84	0.72
6.0	0.97	0.93	0.83

Pile loads are assumed to be aligned at right angles to the direction of the load.

4. The estimated internal friction angles and associated lateral load design factors for typical fill soils are listed below. Once proposed fill soils are evaluated, more accurate values can be supplied.

Fill Soil	**Effective Friction Angle**	**K_o**	**K_a**	**K_p**	**γ_T**
CLAY FILL	25°	0.6	0.4	2.5	20 kN/m^3
GRAVEL	36°	0.4	0.3	3.8	21 kN/m^3

The K_o condition would be applicable in a situation where no movement of the structure is allowed, such as the proposed bridge. The K_a condition would be applicable where some movement of the structure is allowed for, such as the wing walls of the proposed structure.

The amount of movement required to produce active (or K_a) earth pressure is a function of the height of the structure, 0.02H, where H is the height of the structure in meters.

7.5 Earthquake Design

1. Based on the soils encountered in the testholes, the upper 30 metres of soil at this site is comprised generally of stiff to very stiff clay soils. As such, for structural design

purposes, this site can be classified as Seismic Site Response Site Class D as per Table 4.1.8.4.A in the Alberta Building Code 2006.

7.6 Cement

Tests on selected soil samples indicated negligible concentrations of water soluble soil sulphates in the near surface clay deposits. The following alternatives are advised to address the sulphate content in the soil:

1. Underground Concrete Pipe

   Concrete used for all underground pipes must be constructed of C.S.A. Type HS (high sulphate resistant hydraulic cement).

2. Curbs and Sidewalks

   All concrete for surface improvements such as sidewalks and curbs may be constructed using C.S.A. Type GU (general use hydraulic cement).

3. Foundation Construction

   All concrete used for residential construction and coming into direct contact with the soil may be constructed with CSA Type GU (general use hydraulic cement). In addition, all concrete subject to freezing must be air entrained with 5 to 7 percent air. Individual locations may show lower concentrations of soluble soil sulphates, and thus additional soil testing on particular sites may prove valuable.

8.0 CLOSURE

This report has been prepared for the exclusive and confidential use of Qualico Communities, Stantec Consulting Ltd., City of Edmonton and authorized agents. Use of this report is limited to the subject proposed roadway upgrade and subject bridge only. The recommendations given are based on the subsurface soil conditions encountered during test boring, current construction techniques and generally accepted engineering practices. No other warranty, expressed or implied, is made. Due to geological randomness of many soils formations, no interpolation of soil conditions between or away from the testholes has been made or implied. Soil conditions are known only at the test boring location. Should other soils be encountered during construction or other information pertinent becomes available, the undersigned should be contacted as the recommendations may be altered or modified.

We trust this information is satisfactory. If you should have any further questions, please contact our office.

Respectfully Submitted:
HOGGAN ENGINEERING & TESTING (1980) LTD.

IBRAHIM RAHIME

Abe Rahime
May 29/15

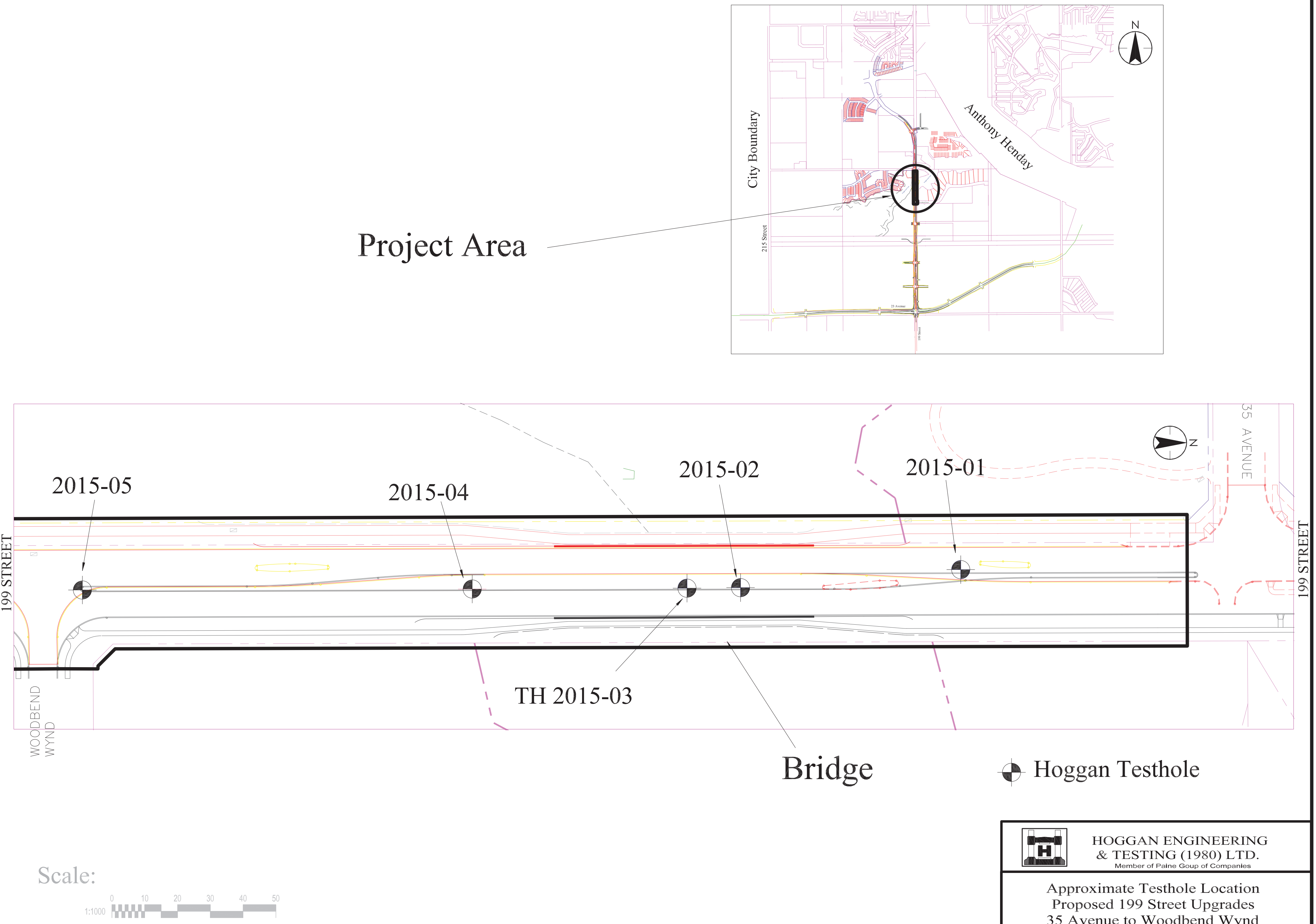
Abe Rahime, P. Eng.

PERMIT TO PRACTICE
HOGGAN ENGINEERING & TESTING (1980) LTD.
Signature R. Evans
Date May 29, 2015
PERMIT NUMBER: P 3691
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

Reviewed By: Rick Evans, P. Eng.

H:\DATA 2015\6004 Stantec Consulting Ltd\6004-38 Proposed 199 Street Upgrade\hr1261sta.doc

APPENDIX I – Site Plan and Testhole Logs



HOGGAN ENGINEERING
& TESTING (1980) LTD.
Member of Paine Goup of Companies

Approximate Testhole Location
Proposed 199 Street Upgrades
35 Avenue to Woodbend Wynd
Edmonton, Alberta

SCALE: 1:1000	DATE: April 2015
FILE #: 6004-38	FIGURE #: 1

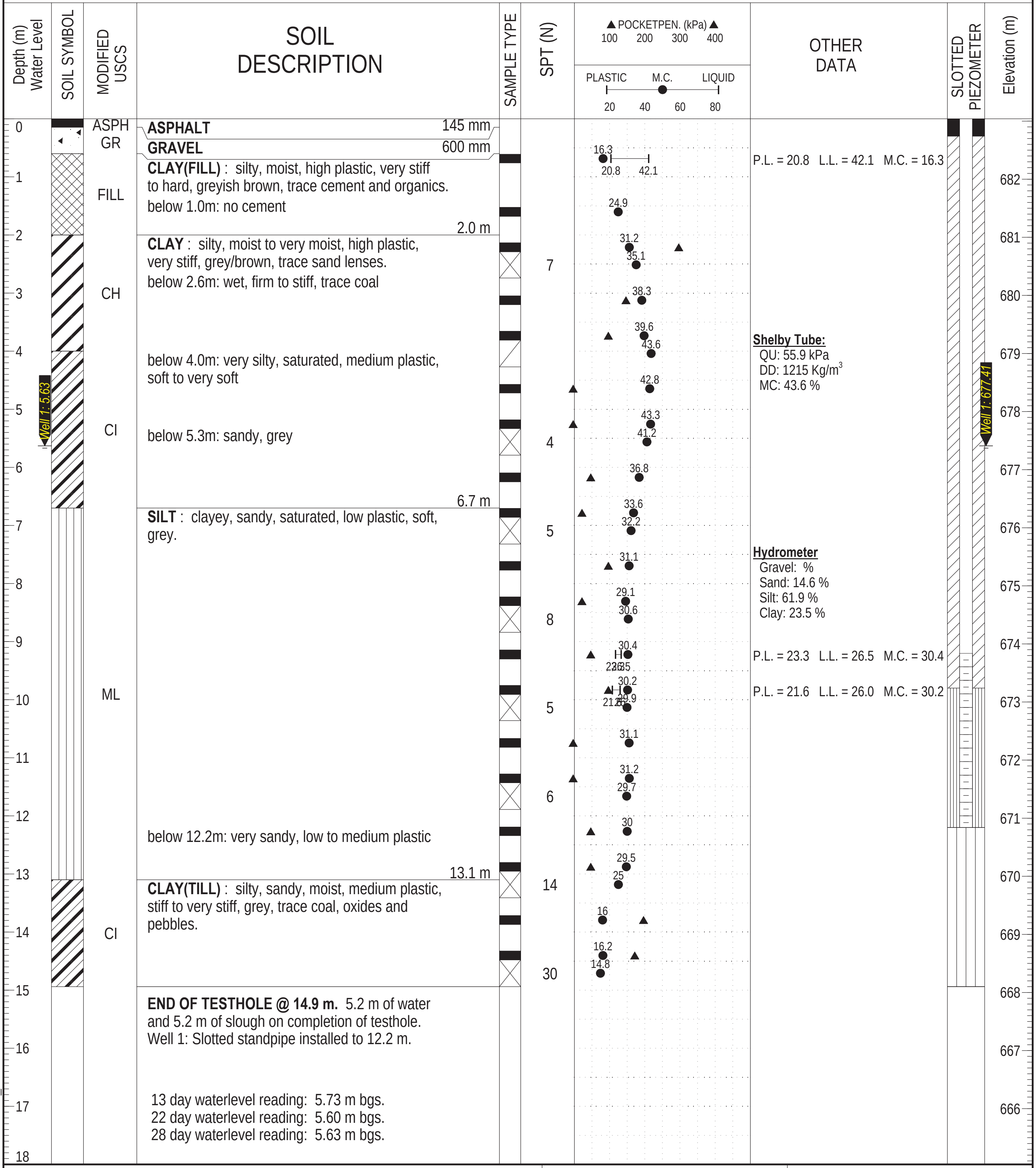
PROJECT: Proposed 199 Street Upgrades | PROJECT NO: 6004-38 | BOREHOLE NO: **2015-01**

CLIENT: Stantec Consulting Ltd | DRILL METHOD: Solid Stem Auger | ELEVATION: 683.04 m

OWNER: Qualico Communities | LOCATION: As per site plan

SAMPLE TYPE: ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁚ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



ASPHALT 145 mm

GRAVEL 600 mm

CLAY(FILL) : silty, moist, high plastic, very stiff to hard, greyish brown, trace cement and organics.
below 1.0m: no cement
2.0 m

CLAY : silty, moist to very moist, high plastic, very stiff, grey/brown, trace sand lenses.
below 2.6m: wet, firm to stiff, trace coal

below 4.0m: very silty, saturated, medium plastic, soft to very soft

below 5.3m: sandy, grey
6.7 m

SILT : clayey, sandy, saturated, low plastic, soft, grey.

below 12.2m: very sandy, low to medium plastic
13.1 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

END OF TESTHOLE @ 14.9 m. 5.2 m of water and 5.2 m of slough on completion of testhole.
Well 1: Slotted standpipe installed to 12.2 m.

13 day waterlevel reading: 5.73 m bgs.
22 day waterlevel reading: 5.60 m bgs.
28 day waterlevel reading: 5.63 m bgs.

OTHER DATA:

P.L. = 20.8 L.L. = 42.1 M.C. = 16.3

Shelby Tube:
QU: 55.9 kPa
DD: 1215 Kg/m^3
MC: 43.6 %

Hydrometer
Gravel: %
Sand: 14.6 %
Silt: 61.9 %
Clay: 23.5 %

P.L. = 23.3 L.L. = 26.5 M.C. = 30.4

P.L. = 21.6 L.L. = 26.0 M.C. = 30.2

JRP 6004-38.GPJ JRPV3_0.GDT 28/05/15

HOGGAN ENGINEERING & TESTING (1980) LTD.

17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime | COMPLETION DEPTH: 14.94 m

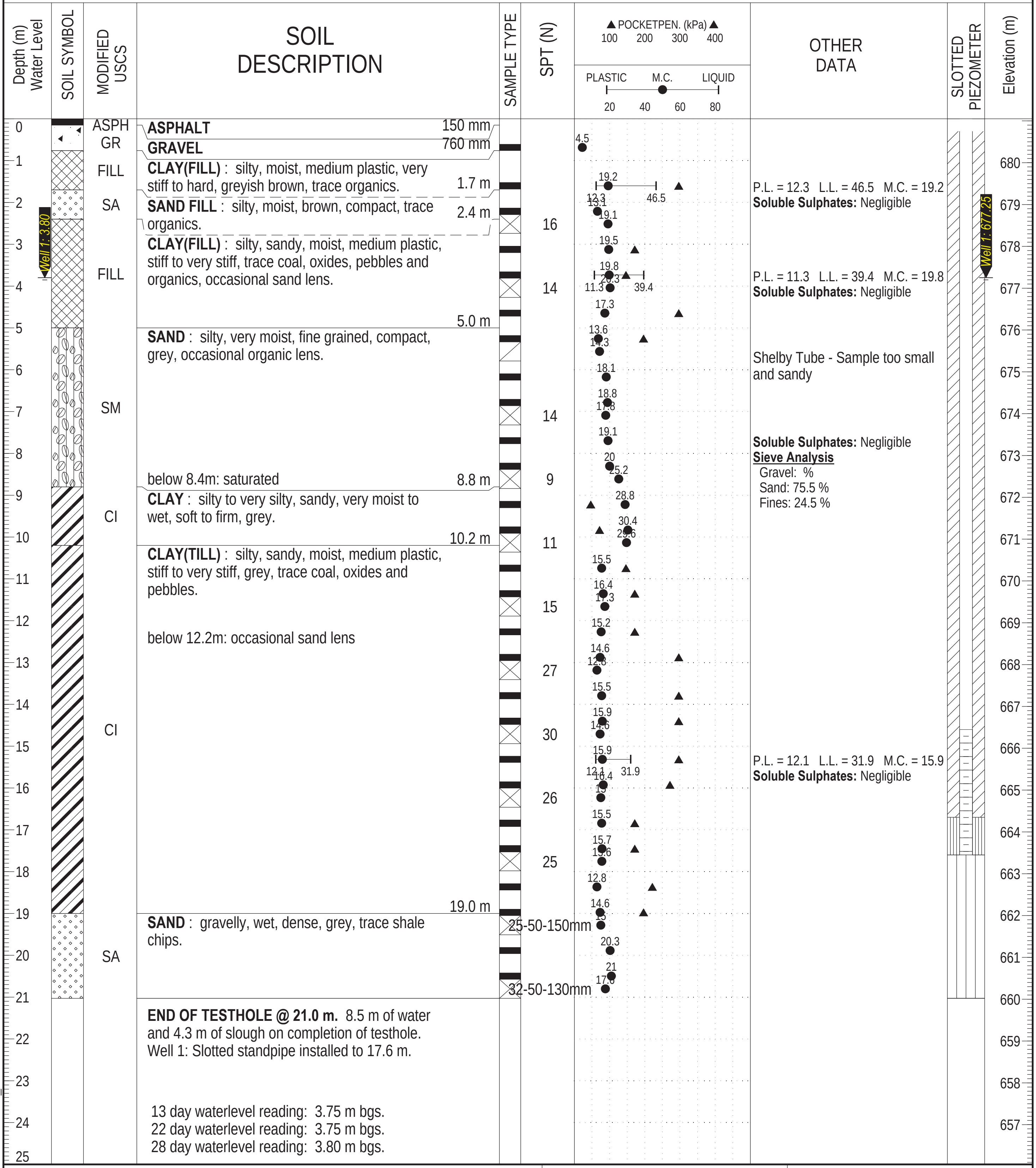
REVIEWED BY: R Evans | COMPLETION DATE: 18/03/15

Fig. No: 2

PROJECT: Proposed 199 Street Upgrades	PROJECT NO: 6004-38	BOREHOLE NO: **2015-02**
CLIENT: Stantec Consulting Ltd	DRILL METHOD: Solid Stem Auger	ELEVATION: 681.05 m
OWNER: Qualico Communities	LOCATION: As per site plan	

SAMPLE TYPE: ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



Depth	Modified USCS	Soil Description	SPT (N)	Other Data
0 – 150 mm	ASPH	**ASPHALT**		
150 mm – 760 mm	GR	**GRAVEL**		
760 mm – 1.7 m	FILL	**CLAY(FILL)** : silty, moist, medium plastic, very stiff to hard, greyish brown, trace organics.		P.L. = 12.3 L.L. = 46.5 M.C. = 19.2 **Soluble Sulphates:** Negligible
1.7 m – 2.4 m	SA	**SAND FILL** : silty, moist, brown, compact, trace organics.	16	
2.4 m – 5.0 m	FILL	**CLAY(FILL)** : silty, sandy, moist, medium plastic, stiff to very stiff, trace coal, oxides, pebbles and organics, occasional sand lens.	14	P.L. = 11.3 L.L. = 39.4 M.C. = 19.8 **Soluble Sulphates:** Negligible
5.0 m – 8.8 m	SM	**SAND** : silty, very moist, fine grained, compact, grey, occasional organic lens. below 8.4m: saturated	14 9	Shelby Tube - Sample too small and sandy **Soluble Sulphates:** Negligible **Sieve Analysis** Gravel: % Sand: 75.5 % Fines: 24.5 %
8.8 m – 10.2 m	CI	**CLAY** : silty to very silty, sandy, very moist to wet, soft to firm, grey.	11	
10.2 m – 19.0 m	CI	**CLAY(TILL)** : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles. below 12.2m: occasional sand lens	15 27 30 26 25	P.L. = 12.1 L.L. = 31.9 M.C. = 15.9 **Soluble Sulphates:** Negligible
19.0 m – 21.0 m	SA	**SAND** : gravelly, wet, dense, grey, trace shale chips.	25-50-150mm 32-50-130mm	

END OF TESTHOLE @ 21.0 m. 8.5 m of water and 4.3 m of slough on completion of testhole.
Well 1: Slotted standpipe installed to 17.6 m.

13 day waterlevel reading: 3.75 m bgs.
22 day waterlevel reading: 3.75 m bgs.
28 day waterlevel reading: 3.80 m bgs.

HOGGAN ENGINEERING & TESTING (1980) LTD. 17505 - 106 Avenue Edmonton, AB T5S 1E7 Phone: (780) 489-0700 Fax: (780) 489-0800	LOGGED BY: A Rahime	COMPLETION DEPTH: 21.03 m
	REVIEWED BY: R Evans	COMPLETION DATE: 18/03/15
	Fig. No: 3	Page 1 of 1

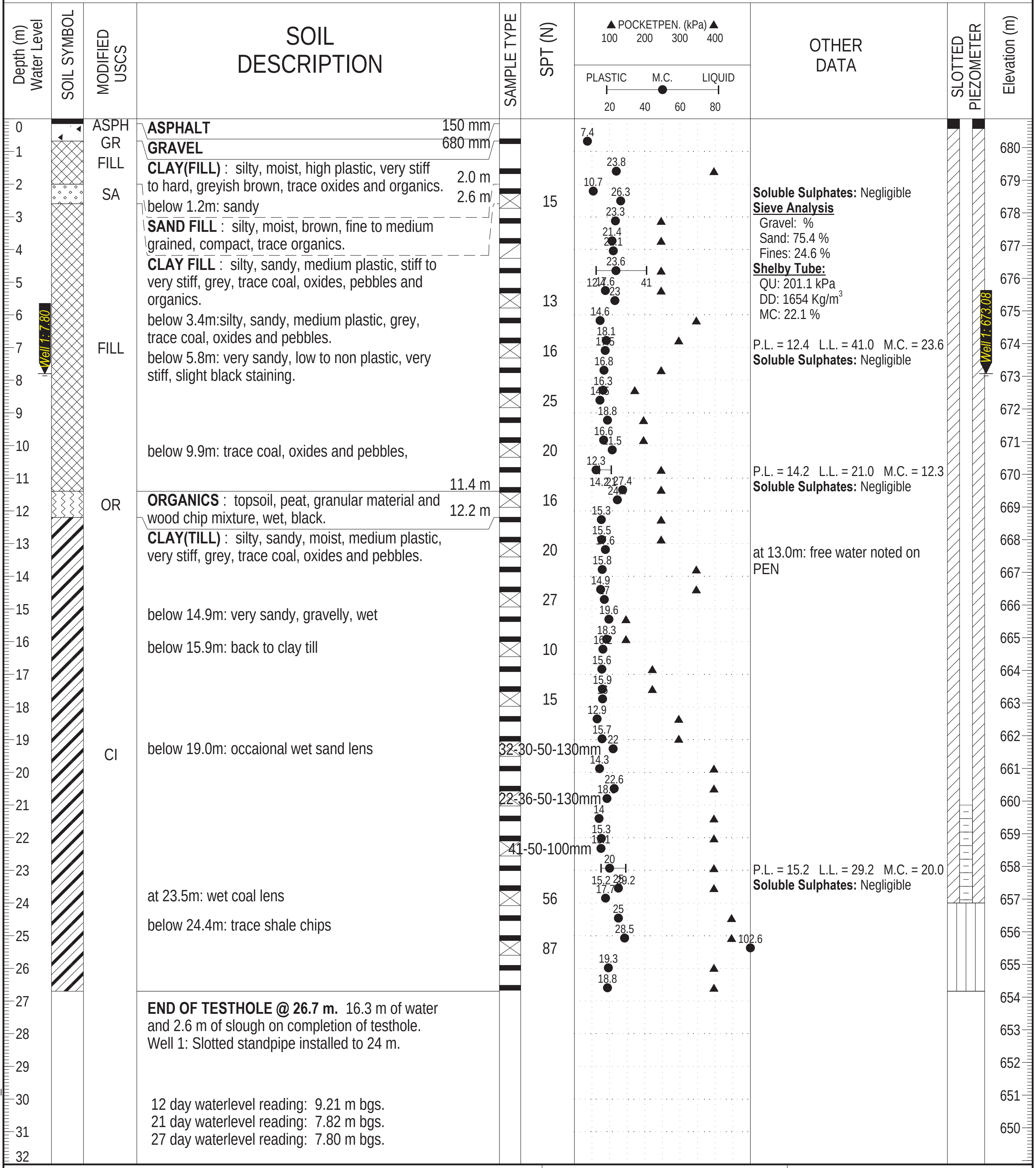
PROJECT: Proposed 199 Street Upgrades | PROJECT NO: 6004-38 | BOREHOLE NO: **2015-03**

CLIENT: Stantec Consulting Ltd | DRILL METHOD: Solid Stem Auger | ELEVATION: 680.88 m

OWNER: Qualico Communities | LOCATION: As per site plan

SAMPLE TYPE: ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



SOIL DESCRIPTION

ASPHALT 150 mm

GRAVEL 680 mm

CLAY(FILL) : silty, moist, high plastic, very stiff to hard, greyish brown, trace oxides and organics. 2.0 m

below 1.2m: sandy 2.6 m

SAND FILL : silty, moist, brown, fine to medium grained, compact, trace organics.

CLAY FILL : silty, sandy, medium plastic, stiff to very stiff, grey, trace coal, oxides, pebbles and organics.

below 3.4m:silty, sandy, medium plastic, grey, trace coal, oxides and pebbles.

below 5.8m: very sandy, low to non plastic, very stiff, slight black staining.

below 9.9m: trace coal, oxides and pebbles, 11.4 m

ORGANICS : topsoil, peat, granular material and wood chip mixture, wet, black. 12.2 m

CLAY(TILL) : silty, sandy, moist, medium plastic, very stiff, grey, trace coal, oxides and pebbles.

below 14.9m: very sandy, gravelly, wet

below 15.9m: back to clay till

below 19.0m: occaional wet sand lens

at 23.5m: wet coal lens

below 24.4m: trace shale chips

END OF TESTHOLE @ 26.7 m. 16.3 m of water and 2.6 m of slough on completion of testhole.
Well 1: Slotted standpipe installed to 24 m.

12 day waterlevel reading: 9.21 m bgs.
21 day waterlevel reading: 7.82 m bgs.
27 day waterlevel reading: 7.80 m bgs.

OTHER DATA

Soluble Sulphates: Negligible
Sieve Analysis
Gravel: %
Sand: 75.4 %
Fines: 24.6 %
Shelby Tube:
QU: 201.1 kPa
DD: 1654 Kg/m^3
MC: 22.1 %

P.L. = 12.4 L.L. = 41.0 M.C. = 23.6
Soluble Sulphates: Negligible

P.L. = 14.2 L.L. = 21.0 M.C. = 12.3
Soluble Sulphates: Negligible

at 13.0m: free water noted on PEN

P.L. = 15.2 L.L. = 29.2 M.C. = 20.0
Soluble Sulphates: Negligible

HOGGAN ENGINEERING & TESTING (1980) LTD.

17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime | COMPLETION DEPTH: 26.70 m
REVIEWED BY: R Evans | COMPLETION DATE: 19/03/15
Fig. No: 4 | Page 1 of 1

JRP 6004-38.GPJ JRPV3_0.GDT 28/05/15

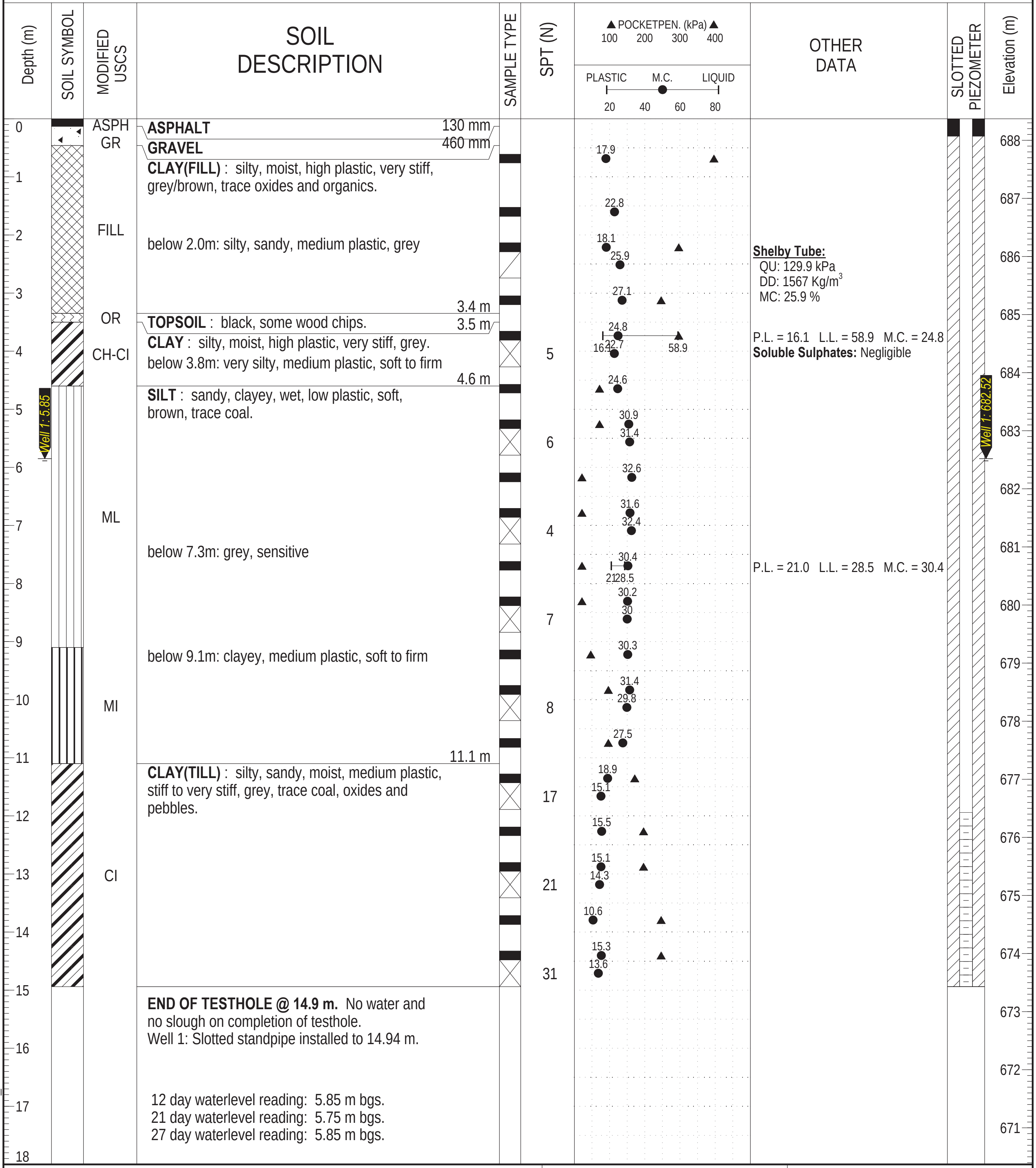
PROJECT: Proposed 199 Street Upgrades | PROJECT NO: 6004-38 | BOREHOLE NO: **2015-04**

CLIENT: Stantec Consulting Ltd | DRILL METHOD: Solid Stem Auger | ELEVATION: 688.37 m

OWNER: Qualico Communities | LOCATION: As per site plan

SAMPLE TYPE: ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁙ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ⁙ SAND



ASPHALT 130 mm

GRAVEL 460 mm

CLAY(FILL) : silty, moist, high plastic, very stiff, grey/brown, trace oxides and organics.

below 2.0m: silty, sandy, medium plastic, grey

3.4 m

TOPSOIL : black, some wood chips. 3.5 m

CLAY : silty, moist, high plastic, very stiff, grey.

below 3.8m: very silty, medium plastic, soft to firm

4.6 m

SILT : sandy, clayey, wet, low plastic, soft, brown, trace coal.

below 7.3m: grey, sensitive

below 9.1m: clayey, medium plastic, soft to firm

11.1 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

END OF TESTHOLE @ 14.9 m. No water and no slough on completion of testhole.
Well 1: Slotted standpipe installed to 14.94 m.

12 day waterlevel reading: 5.85 m bgs.
21 day waterlevel reading: 5.75 m bgs.
27 day waterlevel reading: 5.85 m bgs.

OTHER DATA:

Shelby Tube:
QU: 129.9 kPa
DD: 1567 Kg/m^3
MC: 25.9 %

P.L. = 16.1 L.L. = 58.9 M.C. = 24.8
Soluble Sulphates: Negligible

P.L. = 21.0 L.L. = 28.5 M.C. = 30.4

HOGGAN ENGINEERING & TESTING (1980) LTD.

17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime | COMPLETION DEPTH: 14.94 m

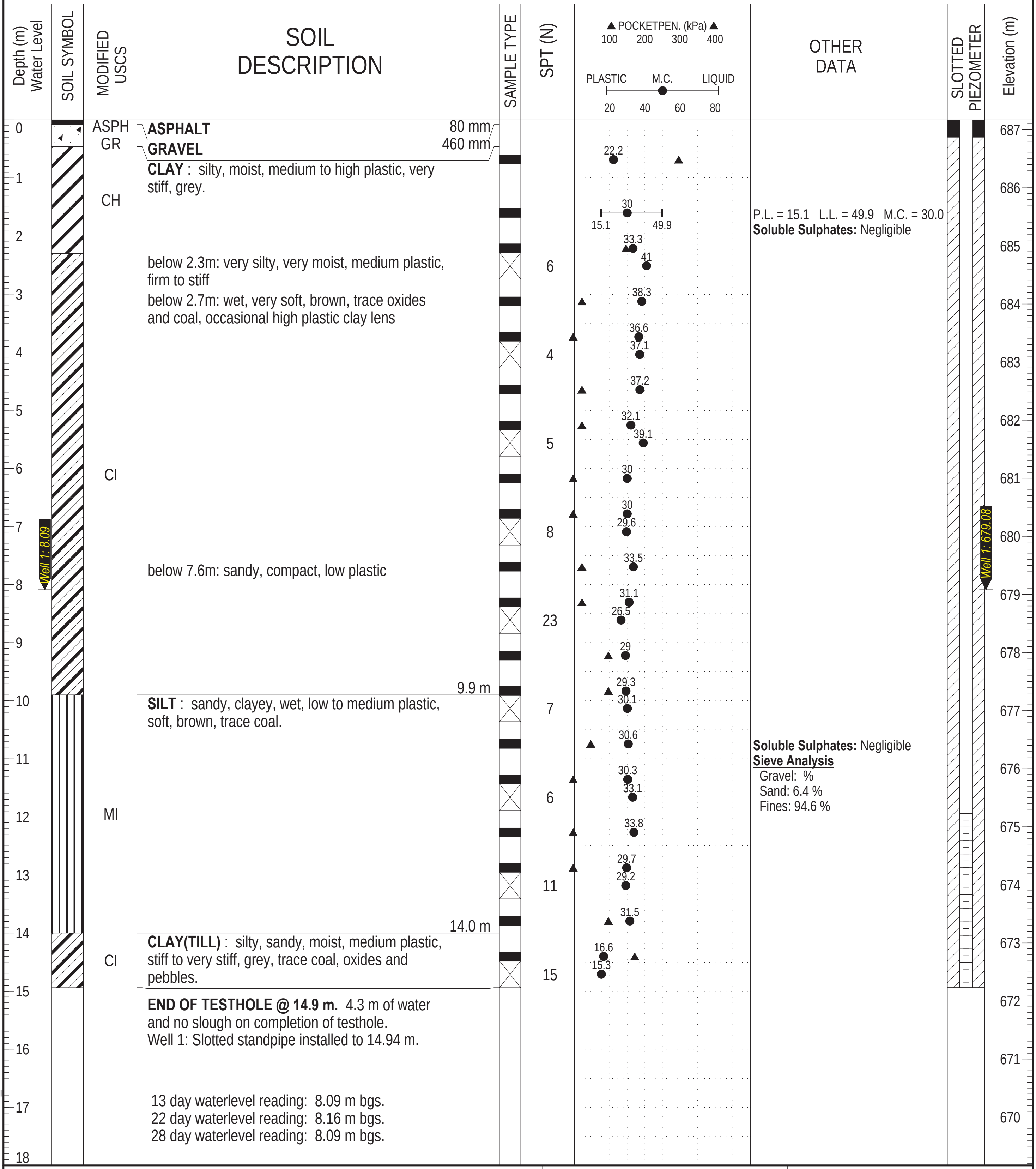
REVIEWED BY: R Evans | COMPLETION DATE: 19/03/15

Fig. No: 5

PROJECT: Proposed 199 Street Upgrades
PROJECT NO: 6004-38
BOREHOLE NO: **2015-05**
CLIENT: Stantec Consulting Ltd
DRILL METHOD: Solid Stem Auger
ELEVATION: 687.17 m
OWNER: Qualico Communities
LOCATION: As per site plan

SAMPLE TYPE: ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



ASPHALT 80 mm

GRAVEL 460 mm

CLAY : silty, moist, medium to high plastic, very stiff, grey.

below 2.3m: very silty, very moist, medium plastic, firm to stiff

below 2.7m: wet, very soft, brown, trace oxides and coal, occasional high plastic clay lens

below 7.6m: sandy, compact, low plastic

9.9 m

SILT : sandy, clayey, wet, low to medium plastic, soft, brown, trace coal.

14.0 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

END OF TESTHOLE @ 14.9 m. 4.3 m of water and no slough on completion of testhole.
Well 1: Slotted standpipe installed to 14.94 m.

13 day waterlevel reading: 8.09 m bgs.
22 day waterlevel reading: 8.16 m bgs.
28 day waterlevel reading: 8.09 m bgs.

P.L. = 15.1 L.L. = 49.9 M.C. = 30.0
Soluble Sulphates: Negligible

Soluble Sulphates: Negligible
Sieve Analysis
Gravel: %
Sand: 6.4 %
Fines: 94.6 %

HOGGAN ENGINEERING & TESTING (1980) LTD.

17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime
REVIEWED BY: R Evans
Fig. No: 6
COMPLETION DEPTH: 14.94 m
COMPLETION DATE: 18/03/15

A P P E N D I X II – Site Photos and G-Slope


Photo 1: 199th Street looking northbound from Woodbend Wynd.


Photo 2: 199th Street looking southbound from 35th Avenue/Edgemont Boulevard.


Photo 3: East side slope and Wedgewood Creek looking down from 199th Street.


Photo 4: East side slope of 199th Street.

Photo 5: Existing culvert output end. Downstream on the east side of 199th Street.

Photo 6: West sideslope of 199th Street looking southwest of Wedgewood Creek.

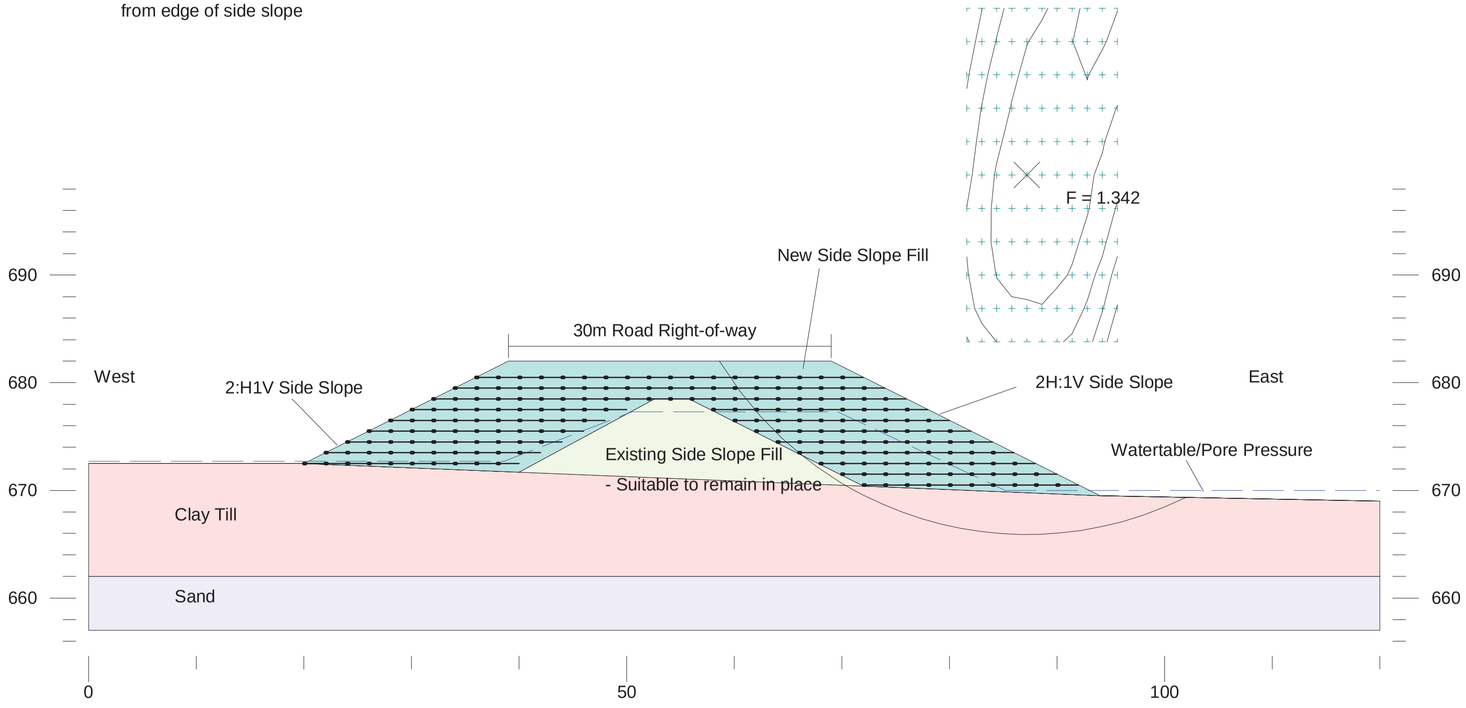
Photo 7: Storm outfall and beaver dam on west sideslope of 199th Street.

Photo 8: Close up of dam west side of 199th Street and input area of culvert.

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
🟩 New Clay Fill	19	0	25	0
Existing Clay Fill	19	2	25	1
🟥 Clay Till	19	5	28	1
Sand	20	0	30	1

Hoggan Engineering & Testing (1980) Ltd.
6004-38
Proposed 199 Street Upgrades
April 5, 2015
Wedgewood Creek Sideslopes
Reinforcement 1m spacing

NOTES:
- Top 3 meters of fill below subgrade, reinforcement shall be placed across extent of fill
- Below 3 meters of fill from subgrade, reinforcement shall be a minimum 20 meters long from edge of side slope

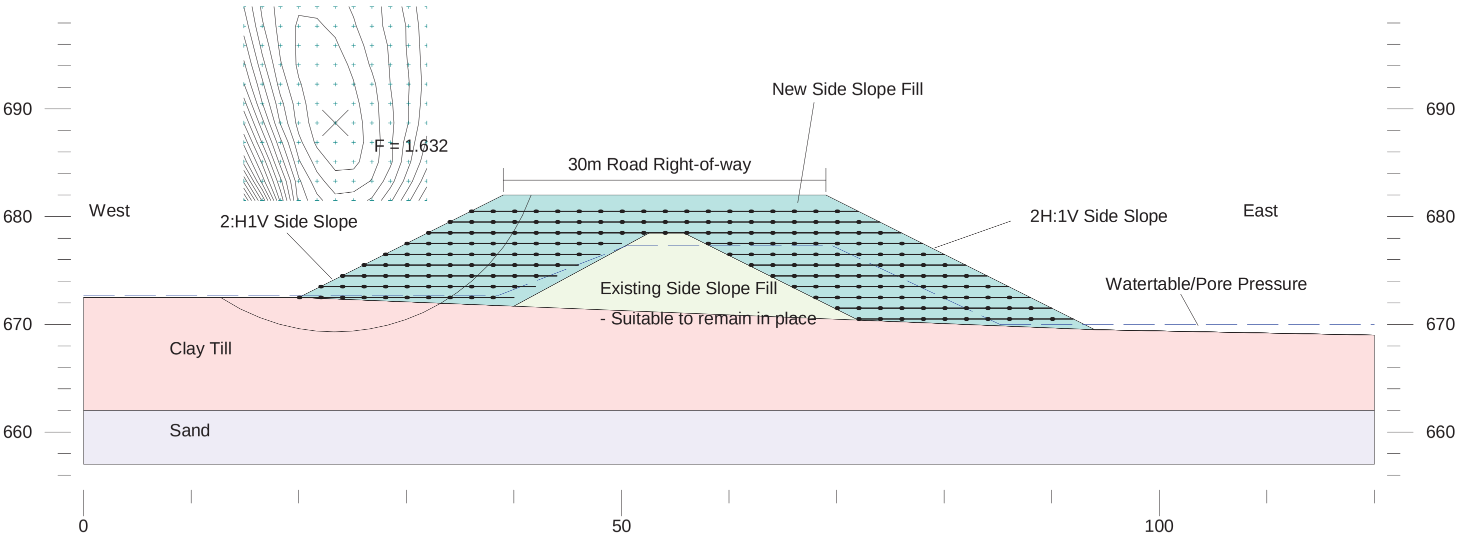


4/22/2015 1:51:57 PM H:\DATA 2015\6004 Stantec Consulting Ltd\6004-38 Proposed 199 Street Upgrade\Gslope\Both Fills Final(East).gsl Hoggan Engineering & Testing (1980) Ltd. F = 1.342

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
New Clay Fill	19	0	25	0
Existing Clay Fill	19	2	25	1
Clay Till	19	5	28	1
Sand	20	0	30	1

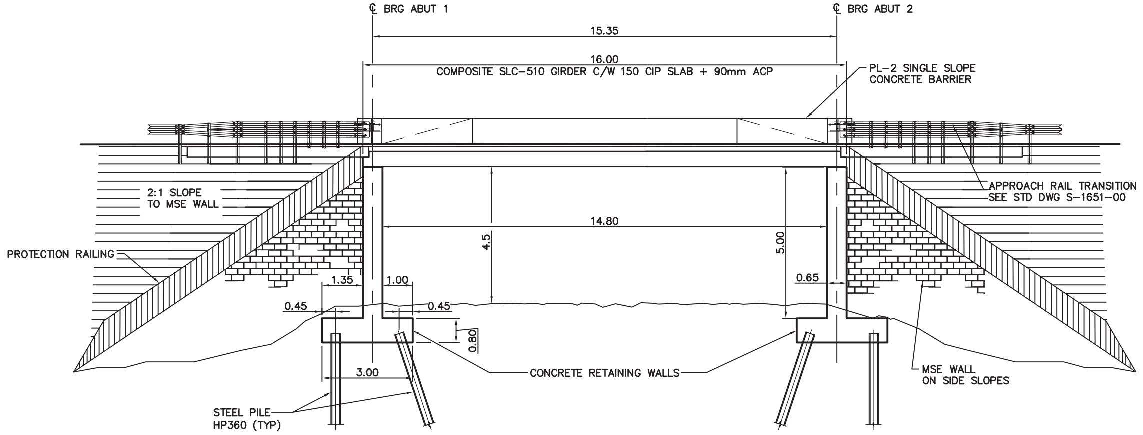
Hoggan Engineering & Testing (1980) Ltd.
6004-38
Proposed 199 Street Upgrades
April 5, 2015
Wedgewood Creek Sideslopes
Reinforcement 1m spacing

NOTES:
- Top 3 meters of fill below subgrade, reinforcement shall be placed across extent of fill
- Below 3 meters of fill from subgrade, reinforcement shall be a minimum 20 meters long from edge of side slope

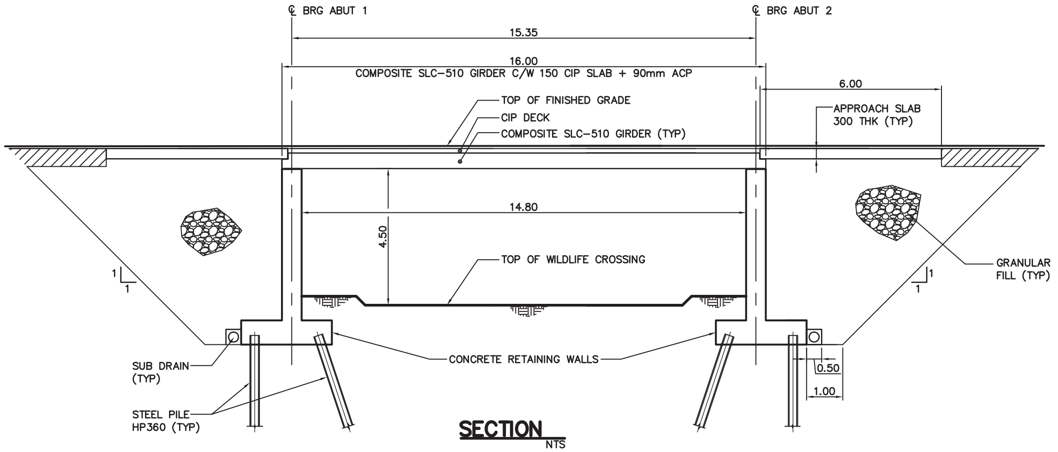


4/22/2015 1:50:00 PM H:\DATA 2015\6004 Stantec Consulting Ltd\6004-38 Proposed 199 Street Upgrade\Gslope\Both Fills Final(west).gsl Hoggan Engineering & Testing (1980) Ltd. F = 1.632

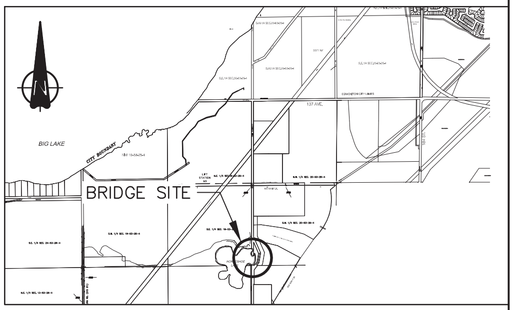
APPENDIX III – Preliminary Bridge Design



SIDE ELEVATION
NTS



SECTION
NTS



SITE PLAN
NTS

ORIGINAL SHEET - ANSI A

Month, Year
04,2015

W:\temporary\199 Street Wildlife Crossing\199 Street.dwg
2015/04/01 2:28 PM By: Pokhrel, Ashish

Stantec

10160 - 112 Street
Edmonton AB Canada
www.stantec.com

Client/Project
RIVERVIEW OWNERS GROUP LTD.
199 STREET
WEDGEWOOD RAVINE

Figure No.
1.0

Title
WILDLIFE CROSSING
CROSS SECTIONS

APPENDIX H

COST ESTIMATES

PROJECT: Wedgewood Creek at 199 Street Culvert Design
BRIDGE FILE:
CONTRACT NO.: **DATE:** 26-Jul-15
JOB NO.: 1161103725 **STANTEC FILE:** 1161103725

Stantec

SPANS & TYPE: 1- 3.0 m dia CSP Culvert **WIDTH :**

LENGTH: 117.5
AREA: 1107.40

COST ESTIMATE TYPE B

	CONTRACT	UNIT	QUANTITY	UNIT PRICE	TOTAL
1	Mobilization	l.sum	1	71,588.00	**71,590**
2	Excavation - Structural	l.sum	1	25,000.00	**25,000**
3	Care of Water (Special Provision)	l.sum	1	40,000.00	**40,000**
4	3000 mm CSP - Supply	m	118	700.00	**82,250**
5	3000 mm CSP - Assembly	m	118	350.00	**41,130**
6	Backfill - Granular	m³	2,000	100.00	**200,000**
7	Backfill - non Granular	m³	375	52.00	**19,500**
8	Heavy Rock Riprap (Class 2)	m³	140	250.00	**35,000**
9	Erosion Control Barrier (Silt Fence)	m	200	15.00	**3,000**
10	Concrete End Treatment	l.sum	2	20,000.00	**40,000**
11	Fish Passage	l.sum	1	20,000	**20,000**
12	Small animal passage	l.sum	1	160,000	**160,000**
13	Remove and dispose of existing Culvert	l.sum	1	50,000	**50,000**
		Total "Contract" :			**$787,470.00**

	Cost-Contract & Materials	$787,470.00
15%	Contingency	$118,120.50
	Total Cost	$905,590.50
	Cost/Area	$817.76

Note: We have not included other project costs such as utility relocation, traffic accommodation during construction and additional Right-of- Way etc.

PROJECT: Wedgewood Creek at 199 Street Culvert Design
BRIDGE FILE:
CONTRACT NO.: **DATE:** 26-Jul-15
JOB NO.: 1161103725 **STANTEC FILE:** 1161103725

Stantec

SPANS & TYPE: 1- 3.05 m dia SPCSP Culvert **WIDTH :**

LENGTH: 117.5

COST ESTIMATE TYPE B

AREA: 1107.40

	CONTRACT	UNIT	QUANTITY	UNIT PRICE	TOTAL
1	Mobilization	l.sum	1	93,091.00	**93,090**
2	Excavation - Subgrade	l.sum	1	25,000.00	**25,000**
3	Care of Water (Special Provision)	l.sum	1	40,000.00	**40,000**
4	3050 mm SPCSP - Supply	m	118	1,800.00	**211,500**
5	3050 mm SPCSP - Assembly	m	118	995.00	**116,910**
6	Backfill - Granular	m³	2,000	100.00	**200,000**
7	Backfill - non Granular	m³	375	52.00	**19,500**
8	Heavy Rock Riprap (Class 2)	m³	140	250.00	**35,000**
9	Erosion Control Barrier (Silt Fence)	m	200	15.00	**3,000**
10	Concrete End Treatment	l.sum	2	20,000.00	**40,000**
11	Fish Passage	l.sum	1	30,000	**30,000**
12	Small animal passage	l.sum	1	160,000	**160,000**
13	Remove and dispose of existing Culvert	l.sum	1	50,000	**50,000**
		Total "Contract" :			**$1,024,000.00**

	Cost-Contract & Materials	$1,024,000.00
15%	Contingency	$153,600.00
	Total Cost	$1,177,600.00
	Cost/Area	$1,063.39

Note: We have not included other project costs such as utility relocation, traffic accommodation during construction and additional Right-of- Way etc.

PROJECT: Wedgewood Creek at 199 Street Culvert Design
BRIDGE FILE:
CONTRACT NO.: **DATE:** 26-Jul-15
JOB NO.: 1161103725 **STANTEC FILE:** 1161103725

SPANS & TYPE: 1- 3.0 m Span X 2.4 m Rise PCC Box Culvert **WIDTH :** 3

Stantec

LENGTH: 117.5

COST ESTIMATE TYPE B

AREA: 1504.00

	CONTRACT	UNIT	QUANTITY	UNIT PRICE	TOTAL
1	Mobilization	l.sum	1	126,848.00	**126,850**
2	Excavation - Subgrade	l.sum	1	25,000.00	**25,000**
3	Care of Water (Special Provision)	l.sum	1	40,000.00	**40,000**
4	3000 mm X 2400 PCC Box Culvert- Supply	m	110	4,867.00	**532,940**
5	3000 mm X 2400 PCC Box Culvert- Installation	l.sum	1	150,000.00	**150,000**
6	Backfill - Granular	m³	2,000	100.00	**200,000**
7	Backfill - non Granular	m³	375	52.00	**19,500**
8	Heavy Rock Riprap (Class 2)	m³	140	250.00	**35,000**
9	Erosion Control Barrier (Silt Fence)	m	200	15.00	**3,000**
10	Bevelled Ends	l.sum	2	11,522.00	**23,040**
11	Fish Passage	l.sum	1	30,000	**30,000**
12	Small animal Passage	l.sum	1	160,000	**160,000**
13	Remove and dispose of existing Culvert	l.sum	1	50,000	**50,000**
	Total "Contract" :				**$1,395,330.00**

	Cost-Contract & Materials	$1,395,330.00
15%	Contingency	$209,299.50
	Total Cost	$1,604,629.50
	Cost/Area	$1,066.91

Note: We have not included other project costs such as utility relocation, traffic accommodation during construction and additional Right-of- Way etc.

Memo

To: Ms. Mikaela Hanley, M.Eng., P.Eng.,
Mr. Alan Mangory, M.Eng., P.Eng.

City of Edmonton Utility Services
600, Century Place
9803 102A Avenue NW
Edmonton, Alberta T5J 3A3

From: Arshed Mahmood
Tony Chiarello

Stantec Consulting Ltd.
10160 112 Street NW
Edmonton, Alberta T5K 2L6

File: 1161103725

Date: May 26, 2016

Reference: **199 Street NW Culvert Crossing at Wedgewood Creek
Additional Information**

Stantec Consulting Ltd. (Stantec) was engaged by Riverview Heights Estates Ltd. c/o Qualico Communities to complete the hydrotechnical investigation for a proposed culvert replacement located at 199 Street NW over Wedgewood Creek in Edmonton, Alberta.

The original Hydrotechnical Summary Report was completed in August 2015. A 3.0m diameter corrugated steel pipe (CSP) closed-bottom culvert was recommended given the existing conditions, Alberta Transportation design guidelines, hydrology and hydraulic analysis, and engineering experience and judgement. The recommended 3.0m diameter CSP culvert designed to accommodate aquatic passage was proposed with first submission detailed design in December 2015. A supplementary memo providing additional information regarding the alignment of the culvert (i.e. aquatic passage) and a summary of the additional options reviewed was completed in April 2016 in response to verbal and written comments received from the City of Edmonton through the review of the Environmental Impact Assessment (EIA).

As a result of the comments received from City Administration on April 28, 2016 and the subsequent meeting that took place on May 04, 2016, a follow-up meeting occurred on May 12, 2016 with Stantec and the City of Edmonton Drainage department (Drainage) to further discuss the design recommendation. At the conclusion of the meeting, it was indicated by Drainage (Mikaela Hanley) that the proposed 3.0m diameter CSP aquatic passage will be supported by City Administration.

This memo provides additional information and responses to the verbal comments made at the meeting with Drainage dated May 12, 2016 related to the proposed aquatic passage design. This memo includes the following:

- Consideration for ice jamming and snow melt in the design of the culvert;
- Expanded modelling results requested by Drainage;
- General recommendations for maintenance;
- Risk review.

For the purpose of this memo, Wedgewood Creek will be referred to as “WWC.” Please read this memo in conjunction with the Hydrotechnical Summary Report (Stantec, August 2015), the culvert design (Stantec, December 2015), and the first supplementary memo (Stantec, April 2016). Figures SK-1 and SK-2 from the Stantec, April 2016 memo have been attached for ease of reference.

Ice Jamming and Snow Melt

Drainage questioned whether ice jamming and snow melt were considered in the design of the culvert. In response, we do not anticipate any ice flow condition for this creek. The creek bed width is narrow and the ice build-up thickness is small due to limited winter flows. We anticipate there would be limited icing around the outfall structure during winter months and that would not impact flows through the culvert. Snow melt in the drainage basin area was also a component of the design flow calculations.

Expanded Modelling Results Requested by City of Edmonton Drainage

Alberta Transportation's software, HydroChan and HydroCulv, were used to calculate velocities and flood elevations in the culvert and model flows through the channel. The detailed modeling results with the revised incremental step are attached to this memo. Please note that the model has limitations with respect to the size of the incremental step. We cannot model with an incremental step less than 0.02 m (in elevation). A summary of the revised modelling results with an incremental step of 0.02 m is presented below in Table 1:

Table 1: Revised HydroChan modeling results for natural channel with an incremental step of 0.02 m

Flood (m3/s)		Modelled Flood (m3/s)	Tail Water Depth of Flow (m)	Tail Water Velocity (m/s)
$Q_{1:100}$ (Design Flood)	14.0	14.14	1.28	2.41
$Q_{1:200}$ (Check Flood)	18.0	18.03	1.42	2.51

The original modeling results with an incremental step of 0.03 m, previously presented, are summarized below in Table 2:

Table 2: Original HydroChan modeling results for natural channel with an incremental step of 0.03 m

Flood (m3/s)		Modelled Flood (m3/s)	Tail Water Depth of Flow (m)	Tail Water Velocity (m/s)
$Q_{1:100}$ (Design Flood)	14.0	14.40	1.29	2.42
$Q_{1:200}$ (Check Flood)	18.0	18.64	1.44	2.52

Reference: 199 Street NW Culvert Crossing at Wedgewood Creek
Additional Information

In Tables 1 and 2, please note the changes in tail water depth of flow and velocity for the two incremental steps. From a modeling point of view, the slight deviation is minor. Due to the limitations of the model, we cannot model the exact flow in the natural channel. However, a review of the attached modeling results shows that the average depth of flows and velocities (to interpolate exact design (1:100 year) and check (1:200 year) flood) for two consecutive incremental steps will be in similar order and do not make a difference in calculations. Note that Manning's roughness for the natural channel was estimated during site visits and was utilized in HydroChan calculations.

The HydroChan results (i.e. tailwater depth of flow and velocity of flow) are input to model culvert flows in HydroCulv.

Table 3: 3.0 m diameter culvert hydraulics using Table 1 information in HydroCulv

Flood (m3/s)	Mean Velocity at Culvert Inlet (m/s)	Mean Velocity at Culvert Outlet (m/s)	Freeboard (m)
14.0	2.3	2.4	0.2
18.0	2.5	2.9	-0.8 (Pipe is submerged at upstream end)

Table 4: 3.0 m diameter culvert hydraulics using Table 2 information in HydroCulv

Flood (m3/s)	Mean Velocity at Culvert Inlet (m/s)	Mean Velocity at Culvert Outlet (m/s)	Freeboard (m)
14.0	2.3	2.4	0.2
18.0	2.5	2.9	-0.7 (Pipe is submerged at upstream end)

Tables 3 and 4 show similar results. Note that Manning's roughness for the culvert to input in HydroCulv is based on Alberta Transportation's guidelines. Note that we are proposing granular substrate to accommodate fish passage (during fish passage flow i.e. 1:2 year flow) and Class 2 rock to hold that substrate in the culvert.

For more information on the model, refer to the original hydrotechnical assessment (Stantec, August 2015), Sections 3.5 and 4.0 and the first supplementary memo sent to the City (Stantec, April 2016).

General Recommendations for Maintenance

Stantec would recommend regular maintenance and debris removal from the culvert inlet when and if required to maintain its full hydraulic capacity. A regular culvert inspection/monitoring plan should be established to determine if any maintenance is required. A suggested inspection plan for this site would be to carry out an inspection every two years and/or after a major flood event of 1:20 year return period or greater. The plan should be adjusted to conform to the City of Edmonton's inspection guidelines for City bridge and culvert structures.

Please note that Alberta Transportation inspects their bridge structures every 18 months or less frequently depending upon the highway standard, etc.

Risk Review

The following risks associated with the proposed 3.0m diameter CSP culvert have been reviewed and are evaluated in terms of very low, low, moderate, and high risk in Table 5 below. Several "check floods" were used to show the risk potential. Please refer to Figure SK-2 attached, and Figure SK-2 from the original hydrotechnical assessment to note the design and check flood elevations.

Table 5: Risk assessment of 3.0m diameter CSP culvert

Risk Number	Risk Description	Risk Tolerance	Mitigation Measures
1	Floods to over top the roadway - *The design (1:100 year) and check floods (1:200 year) do not over top the roadway as shown on the figures noted above.*	Very low risk	N/A
2	Flooding of the small animal passage and large animal passage - *During the 1:200 year flood, the bottoms of the animal passages are above the flood level as shown on Figure SK-2 attached. The animal passages will remain dry.*	Very low risk	N/A
3	Risk to adjacent property - *The Wedgewood Creek ravine is well incised a minimum of 9.0 to 10.0m upstream and downstream of the 199 Street crossing.*	Very low risk	N/A
4	Ice jamming in front of the culvert - *See above*	Very low risk	N/A
5	Channel scour in vicinity of crossing during design flood (1:100 year) - *During the design flood, the mean velocities at the inlet and outlet of the 3.0 m culvert do not exceed than natural channel velocity. Therefore, use of the 3.0 m diameter culvert would not scour or erode downstream channel during a design flood event.*	Low risk	Rip rap armouring is proposed at the culvert outlet to mitigate scour potential.
6	Channel scour in vicinity of crossing during check flood (1:200 year) - *During the check flood (1:200 year), the aquatic passage would be submerged at the upstream end. The velocity at the downstream outlet of the culvert slightly exceeds the velocity of the natural channel.*	Low risk	Rip rap armouring of the culvert inlet (Class 1 with max nominal diameter of 0.45 m) and at the outlet (Class 2 with maximum nominal diameter of 0.8 m) is designed to mitigate lateral channel bank erosion and scour during extreme flood events.

			Erosion and sediment control measures will be installed to protect the roadway embankment.
7	Channel disturbance during construction - *The 3.0 m diameter CSP culvert would allow for a shorter construction time period compared to larger alternative structure options. This structure would also have a smaller construction footprint.*	Low risk (Compared to Alternative Options)	N/A
8	Accumulation of debris at the culvert invert - *High beaver activity exists upstream of the 199 Street crossing.*	Moderate risk	During the design flood, some freeboard will be available to assist in accommodating debris passage. Regular maintenance will be required to maintain the full hydraulic capacity of the culvert. See above for maintenance suggestions.

Closure

This memo was completed using the information available to Stantec to date. Please contact the undersigned should there be further questions or concerns.

Reference: **199 Street NW Culvert Crossing at Wedgewood Creek**
Additional Information

Stantec Consulting Ltd.

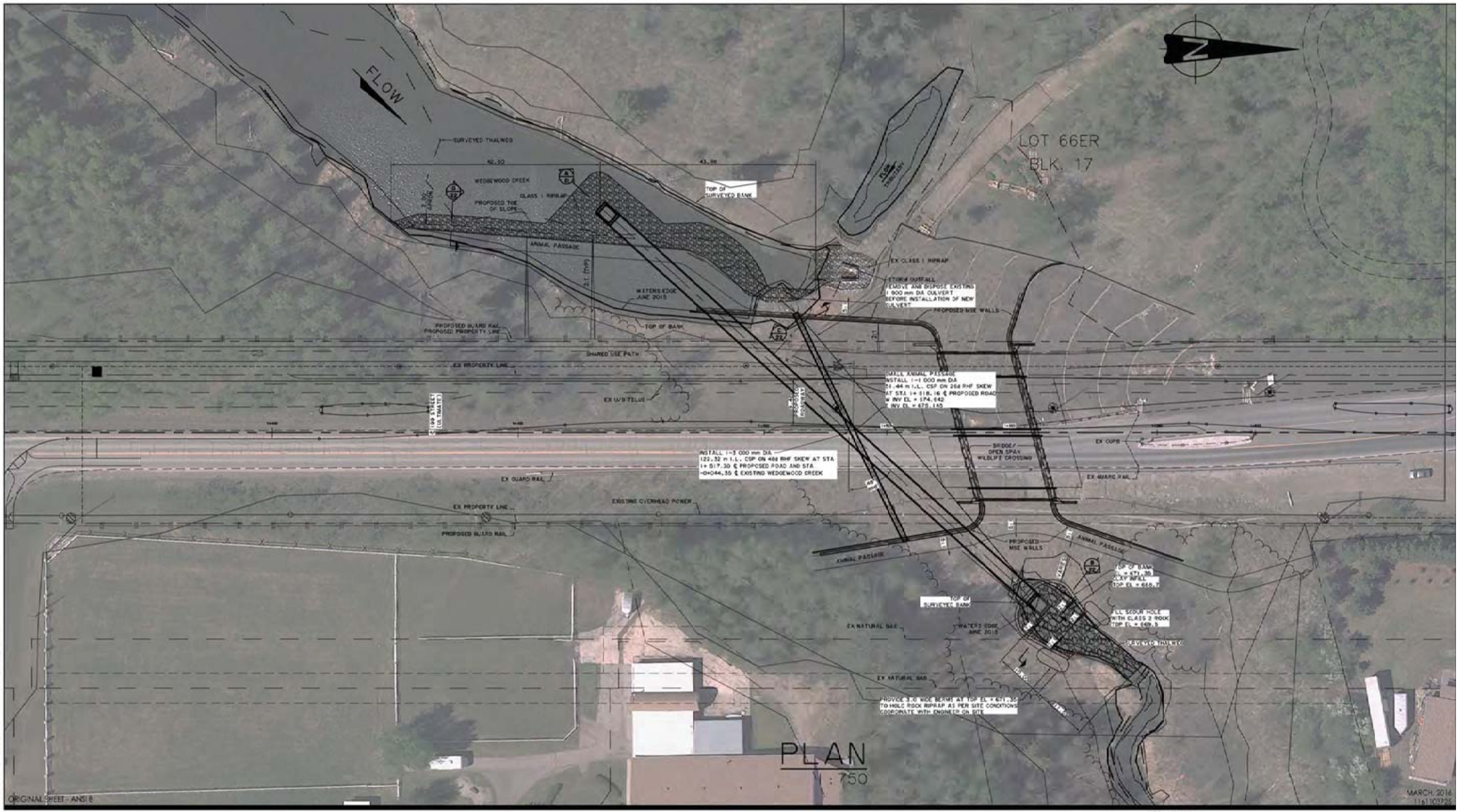
Arshed Mahmood
Bridge Planning and River Engineer
Arshed.Mahmood@stantec.com

Tony Chiarello

Tony Chiarello
Land Development Engineer
Tony.Chiarello@stantec.com

Attachment: Figures SK-1 – Design Culvert Alignment
SK-2 – Option 1 of 3 – 3.0m Diameter Culvert Hydraulics
Additional Modeling Results

c. Micheal Pigeon, Riverview Heights Estates Ltd. c/o Qualico Communities
Reanna Feniak, Riverview Heights Estates Ltd. c/o Qualico Communities
Marc Obert, Stantec
Petrea Chamney, Stantec
Renyuan Cheng, Stantec
Ralph Walters, Stantec



W:\active\1161103725\detailed_design\drawings\sketches\sketch1.dwg
2016/04/01 2:50 PM By: Dressler, Trevor

Stantec

10160 - 112th Street
Edmonton AB
www.stantec.com

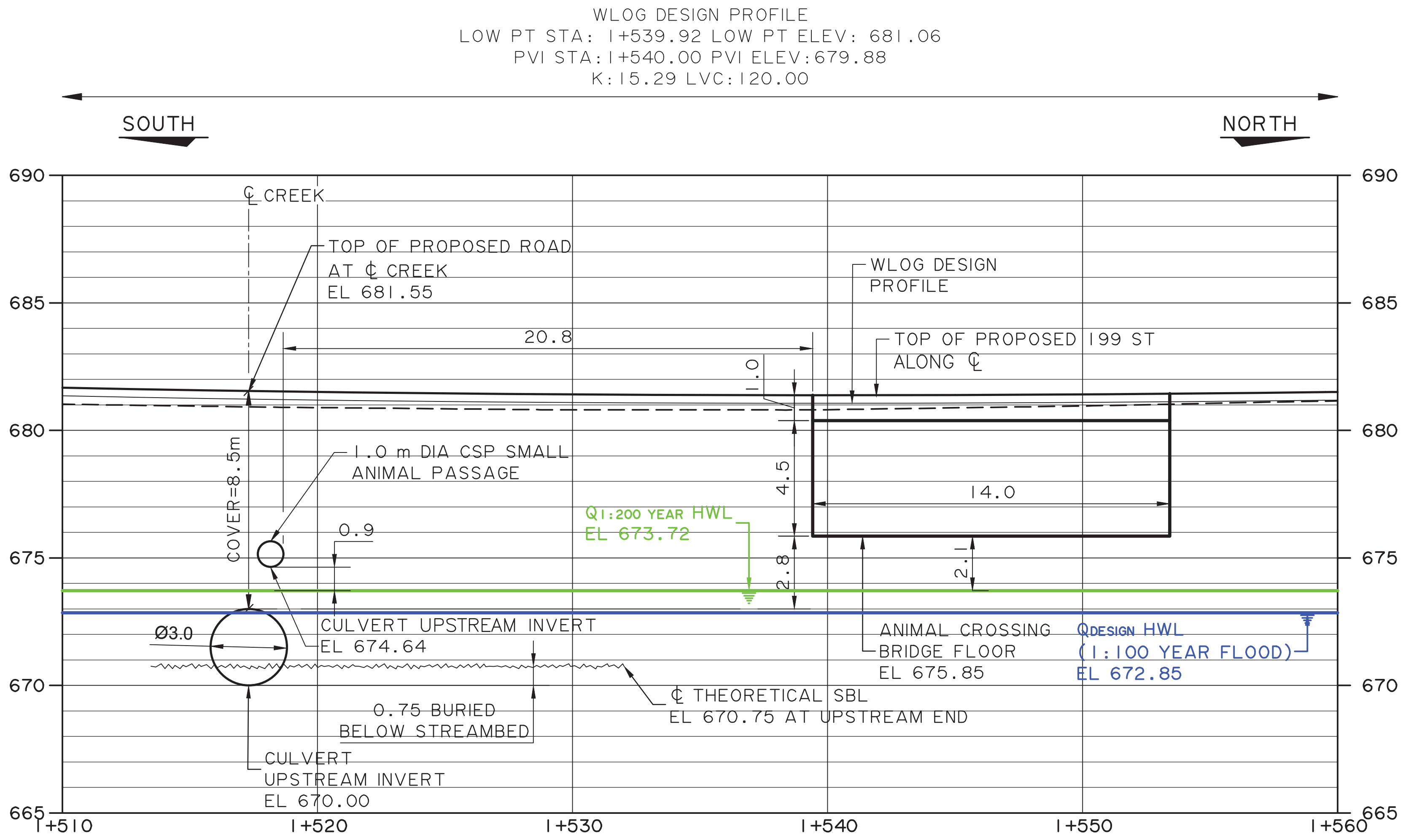
Legend

Notes

Client/Project
RIVERVIEW HEIGHTS ESTATES LTD. C/O QUALICO
199 STREET
CROSSING AT WEDGEWOOD CREEK

Figure No.
SK-1

Title
PLAN - WEDGEWOOD CREEK
CULVERT INSTALLATION UNDER 199 STREET
DESIGNED CULVERT ALIGNMENT



1–3.0 m DIA x 122.0 m LONG CSP CULVERT

MANNINGS ROUGHNESS n = 0.042

1:250

ORIGINAL SHEET - ANSI A

MARCH, 2016
1161103725

W:\active\1161103725\detailed_design\drawings\sketches\sketch2-4.dwg
2016/04/01 3:57 PM By: Dressler, Trevor

Stantec

10160 - 112th Street
Edmonton AB
www.stantec.com

NOTES:

- ALL DIMENSIONS AND ELEVATIONS ARE SHOWN IN METERS
- FLOW ELEVATIONS ARE AT CULVERT INLET

Client/Project
RIVERVIEW HEIGHTS ESTATES LTD. C/O QUALICO
199 STREET
CROSSING AT WEDGEWOOD CREEK

Figure No.
SK-2

Title
OPTION 1 OF 3
3.0 m Ø CULVERT HYDRAULICS

Project Wedgewood Creek at 199 Street_Surveyed Section E (App 50 m d/s from culv

XS Geometry

STA (m)	ELEV (m)
0.00	680.91
1.42	680.05
1.72	679.85
2.52	679.47
4.48	677.72
8.49	674.48
8.59	674.41
8.59	674.41
8.61	674.39
12.64	672.31
14.34	671.67
16.55	671.26
19.00	670.31
20.79	670.23
22.16	670.15
24.67	670.02
27.74	669.69
28.69	669.68
29.25	669.61
29.61	669.36
29.67	669.34
29.81	669.24
32.02	668.74
32.23	668.72
33.14	668.57
34.07	668.41
34.47	668.47
35.38	668.75
35.69	669.07
35.75	669.36
36.05	669.51
36.85	669.86
37.09	669.87
37.94	669.93
38.17	669.93
38.19	669.95
38.68	670.24
40.52	671.23
49.80	674.24
49.92	674.28
49.95	674.29
49.97	674.29
50.02	674.30
58.88	675.37
61.70	675.66

Channel Partition :

Left Overbank	29.2
Right Overbank	36.9

Rating Curve :

Max depth	2
Increment	0.01

Hydraulic Parameters :

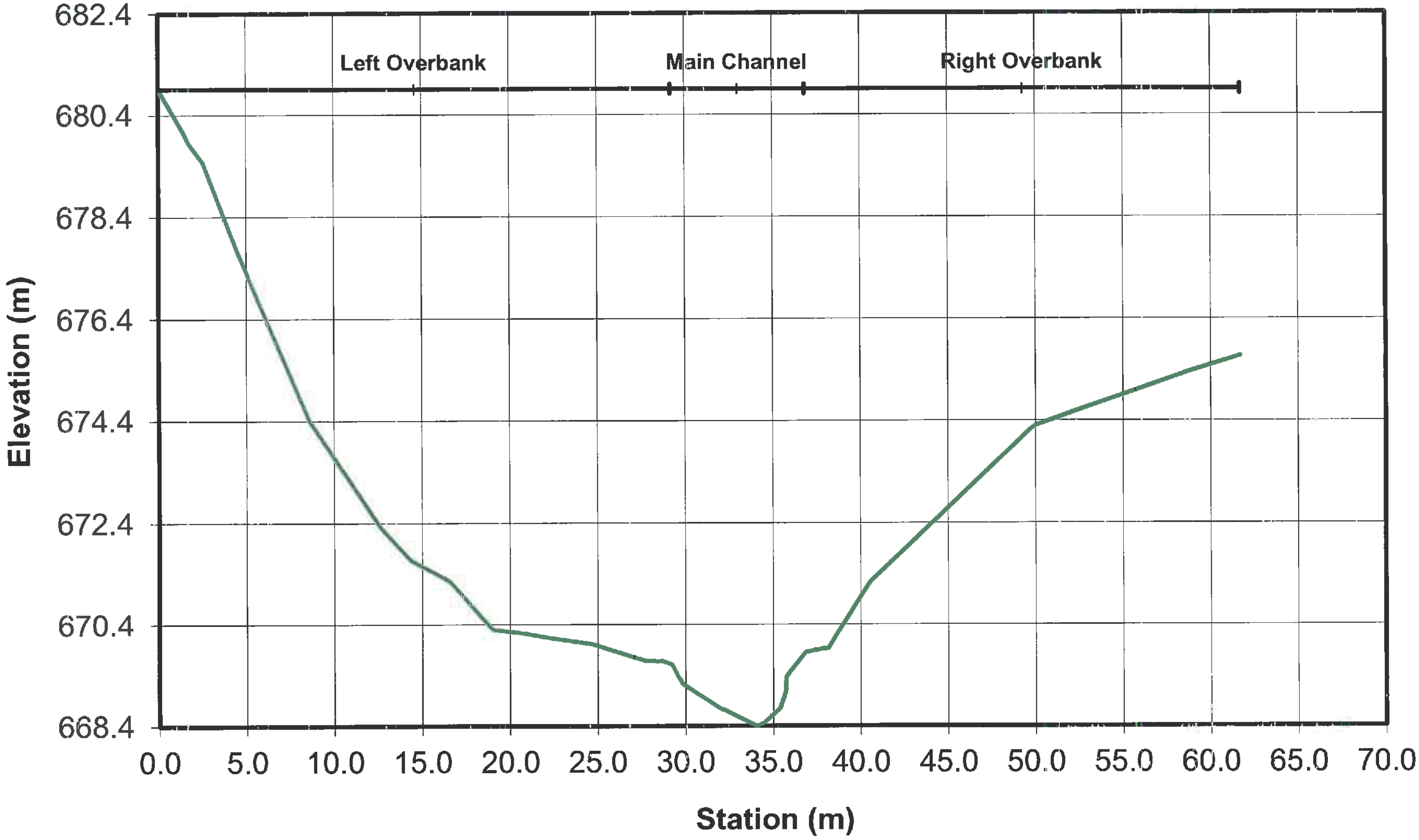
Roughness Type	n
Main Channel Roughness	0.035
Left Overbank Roughness	0.045
Right Overbank Roughness	0.05
Channel Slope	0.011

Boundary Conditions :

	Description	Q (m^3/s)	TW Elev (m)
1	Scenario 1	100	883
2			
3			
4			
5			
6			
7			
8			
9			
10			

	Left Overbank			Main Channel			Right Overbank			Total	Mean
Elevation	A	V	Q	A	V	Q	A	V	Q	Q	V
(m)	(m^2)	(m/s)	(m^3/s)	(m^2)	(m/s)	(m^3/s)	(m^2)	(m/s)	(m^3/s)	(m^3/s)	(m/s)
668.41	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
668.43	0.00	0.00	0.00	0.00	0.14	0.00	0.00	0.00	0.00	0.00	0.14
668.45	0.00	0.00	0.00	0.01	0.22	0.00	0.00	0.00	0.00	0.00	0.22
668.47	0.00	0.00	0.00	0.02	0.29	0.01	0.00	0.00	0.00	0.01	0.29
668.49	0.00	0.00	0.00	0.04	0.36	0.01	0.00	0.00	0.00	0.01	0.36
668.51	0.00	0.00	0.00	0.06	0.42	0.03	0.00	0.00	0.00	0.03	0.42
668.53	0.00	0.00	0.00	0.08	0.48	0.04	0.00	0.00	0.00	0.04	0.48
668.55	0.00	0.00	0.00	0.11	0.53	0.06	0.00	0.00	0.00	0.06	0.53
668.57	0.00	0.00	0.00	0.14	0.58	0.08	0.00	0.00	0.00	0.08	0.58
668.59	0.00	0.00	0.00	0.18	0.62	0.11	0.00	0.00	0.00	0.11	0.62
668.61	0.00	0.00	0.00	0.22	0.67	0.14	0.00	0.00	0.00	0.14	0.67
668.63	0.00	0.00	0.00	0.26	0.71	0.18	0.00	0.00	0.00	0.18	0.71
668.65	0.00	0.00	0.00	0.31	0.75	0.23	0.00	0.00	0.00	0.23	0.75
668.67	0.00	0.00	0.00	0.36	0.79	0.28	0.00	0.00	0.00	0.28	0.79
668.69	0.00	0.00	0.00	0.41	0.83	0.34	0.00	0.00	0.00	0.34	0.83
668.71	0.00	0.00	0.00	0.47	0.86	0.40	0.00	0.00	0.00	0.40	0.86
668.73	0.00	0.00	0.00	0.53	0.90	0.47	0.00	0.00	0.00	0.47	0.90
668.75	0.00	0.00	0.00	0.59	0.92	0.55	0.00	0.00	0.00	0.55	0.92
668.77	0.00	0.00	0.00	0.66	0.97	0.64	0.00	0.00	0.00	0.64	0.97
668.79	0.00	0.00	0.00	0.73	1.02	0.74	0.00	0.00	0.00	0.74	1.02
668.81	0.00	0.00	0.00	0.80	1.06	0.85	0.00	0.00	0.00	0.85	1.06
668.83	0.00	0.00	0.00	0.88	1.10	0.97	0.00	0.00	0.00	0.97	1.10
668.85	0.00	0.00	0.00	0.96	1.14	1.10	0.00	0.00	0.00	1.10	1.14
668.87	0.00	0.00	0.00	1.04	1.18	1.23	0.00	0.00	0.00	1.23	1.18
668.89	0.00	0.00	0.00	1.12	1.22	1.37	0.00	0.00	0.00	1.37	1.22
668.91	0.00	0.00	0.00	1.20	1.26	1.52	0.00	0.00	0.00	1.52	1.26
668.93	0.00	0.00	0.00	1.29	1.30	1.67	0.00	0.00	0.00	1.67	1.30
668.95	0.00	0.00	0.00	1.38	1.33	1.83	0.00	0.00	0.00	1.83	1.33
668.97	0.00	0.00	0.00	1.47	1.37	2.01	0.00	0.00	0.00	2.01	1.37
668.99	0.00	0.00	0.00	1.56	1.40	2.18	0.00	0.00	0.00	2.18	1.40
669.01	0.00	0.00	0.00	1.66	1.43	2.37	0.00	0.00	0.00	2.37	1.43
669.03	0.00	0.00	0.00	1.75	1.46	2.57	0.00	0.00	0.00	2.57	1.46
669.05	0.00	0.00	0.00	1.85	1.50	2.77	0.00	0.00	0.00	2.77	1.50
669.07	0.00	0.00	0.00	1.95	1.53	2.98	0.00	0.00	0.00	2.98	1.53
669.09	0.00	0.00	0.00	2.06	1.56	3.21	0.00	0.00	0.00	3.21	1.56
669.11	0.00	0.00	0.00	2.16	1.59	3.44	0.00	0.00	0.00	3.44	1.59
669.13	0.00	0.00	0.00	2.27	1.62	3.68	0.00	0.00	0.00	3.68	1.62
669.15	0.00	0.00	0.00	2.38	1.65	3.93	0.00	0.00	0.00	3.93	1.65
669.17	0.00	0.00	0.00	2.49	1.68	4.19	0.00	0.00	0.00	4.19	1.68
669.19	0.00	0.00	0.00	2.60	1.71	4.45	0.00	0.00	0.00	4.45	1.71
669.21	0.00	0.00	0.00	2.72	1.74	4.72	0.00	0.00	0.00	4.72	1.74
669.23	0.00	0.00	0.00	2.83	1.77	5.01	0.00	0.00	0.00	5.01	1.77
669.25	0.00	0.00	0.00	2.95	1.80	5.31	0.00	0.00	0.00	5.31	1.80
669.27	0.00	0.00	0.00	3.07	1.84	5.64	0.00	0.00	0.00	5.64	1.84
669.29	0.00	0.00	0.00	3.19	1.87	5.98	0.00	0.00	0.00	5.98	1.87
669.31	0.00	0.00	0.00	3.31	1.91	6.32	0.00	0.00	0.00	6.32	1.91
669.33	0.00	0.00	0.00	3.43	1.94	6.68	0.00	0.00	0.00	6.68	1.94
669.35	0.00	0.00	0.00	3.55	1.98	7.02	0.00	0.00	0.00	7.02	1.98
669.37	0.00	0.00	0.00	3.68	2.01	7.38	0.00	0.00	0.00	7.38	2.01

XS Plot - Wedgewood Creek at 199 Street_Surveyed Section E (App
50 m d/s from culvert outlet)



669.39	0.00	0.00	0.00	3.80	2.03	7.73	0.00	0.00	0.00	7.73	2.03
669.41	0.00	0.00	0.00	3.93	2.06	8.10	0.00	0.00	0.00	8.10	2.06
669.43	0.00	0.00	0.00	4.05	2.09	8.48	0.00	0.00	0.00	8.48	2.09
669.45	0.00	0.00	0.00	4.18	2.12	8.86	0.00	0.00	0.00	8.86	2.12
669.47	0.00	0.00	0.00	4.31	2.15	9.25	0.00	0.00	0.00	9.25	2.15
669.49	0.00	0.00	0.00	4.44	2.17	9.65	0.00	0.00	0.00	9.65	2.17
669.51	0.00	0.00	0.00	4.58	2.20	10.06	0.00	0.00	0.00	10.06	2.20
669.53	0.00	0.00	0.00	4.71	2.22	10.48	0.00	0.00	0.00	10.48	2.22
669.55	0.00	0.00	0.00	4.85	2.25	10.90	0.00	0.00	0.00	10.90	2.25
669.57	0.00	0.00	0.00	4.98	2.27	11.33	0.00	0.00	0.00	11.33	2.27
669.59	0.00	0.00	0.00	5.12	2.30	11.78	0.00	0.00	0.00	11.78	2.30
669.61	0.00	0.00	0.00	5.26	2.32	12.23	0.00	0.00	0.00	12.23	2.32
669.63	0.00	0.07	0.00	5.40	2.34	12.66	0.00	0.00	0.00	12.66	2.34
669.65	0.00	0.14	0.00	5.55	2.37	13.14	0.00	0.00	0.00	13.14	2.37
669.67	0.01	0.19	0.00	5.69	2.40	13.64	0.00	0.00	0.00	13.64	2.39
669.69	0.02	0.14	0.00	5.83	2.42	14.14	0.00	0.00	0.00	14.14	2.41
669.71	0.06	0.23	0.01	5.98	2.45	14.65	0.00	0.00	0.00	14.66	2.43
669.73	0.09	0.30	0.03	6.13	2.47	15.16	0.00	0.00	0.00	15.19	2.44
669.75	0.13	0.36	0.05	6.27	2.50	15.69	0.00	0.00	0.00	15.73	2.46
669.77	0.17	0.40	0.07	6.42	2.53	16.22	0.00	0.00	0.00	16.29	2.47
669.79	0.22	0.45	0.10	6.57	2.55	16.76	0.00	0.00	0.00	16.86	2.48
669.81	0.27	0.49	0.13	6.72	2.57	17.31	0.00	0.00	0.00	17.44	2.49
669.83	0.32	0.53	0.17	6.87	2.60	17.86	0.00	0.00	0.00	18.03	2.51
669.85	0.38	0.56	0.21	7.03	2.62	18.43	0.00	0.00	0.00	18.64	2.52
669.87	0.44	0.60	0.26	7.18	2.65	19.02	0.00	0.06	0.00	19.28	2.53
669.89	0.51	0.63	0.32	7.33	2.68	19.64	0.01	0.13	0.00	19.96	2.54
669.91	0.58	0.66	0.38	7.48	2.71	20.26	0.02	0.18	0.00	20.65	2.55
669.93	0.65	0.69	0.45	7.64	2.74	20.90	0.04	0.22	0.01	21.35	2.56
669.95	0.73	0.72	0.52	7.79	2.76	21.53	0.07	0.27	0.02	22.07	2.57
669.97	0.81	0.74	0.60	7.94	2.79	22.17	0.09	0.33	0.03	22.80	2.58
669.99	0.89	0.77	0.69	8.10	2.82	22.82	0.12	0.38	0.05	23.55	2.59
670.01	0.98	0.80	0.78	8.25	2.85	23.47	0.15	0.43	0.06	24.32	2.59
670.03	1.07	0.81	0.87	8.40	2.87	24.13	0.18	0.47	0.08	25.08	2.60
670.05	1.17	0.82	0.95	8.56	2.90	24.80	0.21	0.51	0.11	25.86	2.60
670.07	1.28	0.83	1.05	8.71	2.92	25.46	0.24	0.55	0.13	26.65	2.61
670.09	1.39	0.84	1.17	8.86	2.95	26.14	0.27	0.58	0.16	27.46	2.61
670.11	1.51	0.85	1.29	9.01	2.97	26.81	0.30	0.61	0.18	28.29	2.61
670.13	1.64	0.87	1.42	9.17	3.00	27.50	0.33	0.64	0.21	29.13	2.61
670.15	1.78	0.88	1.57	9.32	3.02	28.18	0.37	0.67	0.25	30.00	2.62
670.17	1.93	0.90	1.74	9.47	3.05	28.87	0.40	0.70	0.28	30.89	2.62
670.19	2.08	0.92	1.92	9.63	3.07	29.57	0.44	0.72	0.31	31.81	2.62
670.21	2.24	0.94	2.11	9.78	3.10	30.27	0.47	0.75	0.35	32.74	2.62
670.23	2.40	0.96	2.32	9.93	3.12	30.98	0.51	0.77	0.39	33.68	2.62
670.25	2.57	0.98	2.51	10.09	3.14	31.69	0.54	0.79	0.43	34.63	2.62
670.27	2.76	0.99	2.72	10.24	3.16	32.40	0.58	0.82	0.47	35.59	2.62
670.29	2.95	1.00	2.94	10.39	3.19	33.12	0.62	0.84	0.52	36.57	2.62
670.31	3.15	1.02	3.20	10.54	3.21	33.84	0.66	0.86	0.56	37.60	2.62
670.33	3.35	1.06	3.54	10.70	3.23	34.56	0.70	0.88	0.61	38.71	2.62
670.35	3.56	1.09	3.89	10.85	3.25	35.29	0.74	0.90	0.66	39.84	2.63
670.37	3.76	1.13	4.25	11.00	3.27	36.03	0.78	0.91	0.71	40.99	2.64
670.39	3.97	1.17	4.63	11.16	3.30	36.76	0.82	0.93	0.77	42.16	2.64
670.41	4.18	1.20	5.02	11.31	3.32	37.50	0.86	0.95	0.82	43.35	2.65

INTERPOLATION
CAN BE CARRIE
OUT TO FIND
VALUES FO
DESIGN
FLOOD
AN
CHECK FL

Project Wedgewood Creek_at 199 street proposed _3000 mm CSP

Culvert Data

Pipe No.	1	2	3	4
Include (Y/N)	Y	Y		
Station (m)	380.086	380.086		
U/S Invert El (m)	670.000	675.850		
D/S Invert El (m)	668.700	675.500		
Length (m)	122.30	32.60		
Roughness n	0.042	0.035		
Ent. Loss Coeff.	0.7	0.7		
Exit Loss Coeff.	1	1		
Shape	R	B		
Rise (m)	3.00	4.50		
Span (m)		14.00		

slope=	0.0106296	0.0107362

Boundary Conditions :

	Description	Q (m^3/s)	TW Elev (m)	D/S Vel (m/s)
1	Q design	14	670.73	2.41
	Q1:200	18	670.87	2.51
3				
5				
6				
7				
8				
9				
10				

Performance Curve Parameters

Channel Data

S	
B	
h	
T	
Rough	
D/S Bed EL	
U/S Bed El	

Flow Curve Data

Min Y	
Max Y	
Y inc	

Size Curve Data

BC Type
BC Value
Min D
Max D
D inc

Output Summary - Wedgewood Creek_at 199 street proposed _3000 mm CSP

BC No.	1	2					
Q (cms)	14.0	18.0					
TW (m)	670.73	670.87					
Vds (m/s)	2.41	2.51					
HW (m)	672.85	673.76					
Headloss (m)	1.82	2.57					

BC No. 1 - Q design

	Pipe 1	Pipe 2			
Q (cms)	13.99	0.00			
Freeboard (m)	0.15	0.00			
Ynorm (m)	2.40	0.00			
Ycrit (m)	1.62	0.00			
Vout (m/s)	2.38	0.00			
Vin (m/s)	2.33	0.00			
Flow Desc.	M2	No Flow			

BC No. 2 - Q1:200

	Pipe 1	Pipe 2			
Q (cms)	17.98	0.00			
Freeboard (m)	-0.76	0.00			
Ynorm (m)	3.00	0.00			
Ycrit (m)	1.85	0.00			
Vout (m/s)	2.87	0.00			
Vin (m/s)	2.54	0.00			
Flow Desc.	M2 - Full	No Flow			

Wedgewood Creek Crossing at 199 Street: Evaluation of Wildlife Passage – Preliminary Design

Prepared for:
Riverview Owners Group
(Qualico Communities, Walton
Development and Management
LP, Melcor Developments Ltd,
and Sunwapta Holdings
Corporation)

Prepared by:
William L. Harper, R.P.Bio.

October 23, 2015

Table of Contents

LIST OF FIGURES

1.0 BACKGROUND

Stantec Consulting Ltd. (Stantec) was retained by Riverview Owners Group (the Client) to provide environmental consulting services and recommendations for wildlife passage as part of the 199 Street Widening within the Riverview Neighbourhood 2 (the Project). In an effort to minimize the impacts on wildlife movement from transportation infrastructure, the City of Edmonton commissioned the development of the Wildlife Passage Engineering Design Guidelines (WPEDG) (City of Edmonton 2010). The objective of these guidelines is to reduce human-wildlife conflict through improved awareness, safety, and collision reduction while also aiding in the maintenance of habitat connectivity and reduced genetic isolation.

The 199 Street Concept Planning Report determined that 22% of all vehicle collisions and 30% of collisions at midblocks (between intersections) were animal-vehicle collisions (CIMA 2014a). These were attributed to the presence of white-tailed deer in the Project area and the lack of wildlife passage across 199 Street at the Wedgewood Ravine (CIMA+ 2014a).

As part of the Riverview Neighbourhood 2 development, 199 Street will be widened from its current 2-lane rural configuration to a 4-lane arterial roadway configuration (CIMA+ 2014a). The widened road, along with projected increases in traffic volume and vehicle speed, will increase the barrier effect of the road on wildlife. For this reason, and to reduce animal-vehicle collisions, provisions for wildlife movement where 199 Street crosses the Wedgewood Ravine were developed.

At the conceptual design stage, the City of Edmonton requested additional information pertaining to wildlife passage associated with the Project (City of Edmonton 2014a; 2014b). These were addressed in a letter report (CIMA+ 2014b), the 2nd submission of the 199 Street concept planning report (CIMA+ 2014c), and three earlier reports on wildlife passage design (Stantec Consulting Ltd. 2014a; 2014b; 2014c).

The current preliminary design (3rd submission) for the proposed widening over Wedgewood Creek includes:

- an open-span bridge structure (14 m x 4.65 m) to accommodate large, medium and small terrestrial species,
- a dry-passage culvert (1 m diameter), to accommodate medium and small terrestrial species, and
- a wet-passage culvert (3 m diameter) to accommodate aquatic species.

2.0 OBJECTIVES

The objective of this report is to evaluate the potential for the three proposed wildlife crossing structures to maintain landscape permeability for the Ecological Design Groups (EDGs) predicted to occur in the area, and to respond to questions/concerns outlined by the City of Edmonton during the preliminary design stage (City of Edmonton 2015a; 2015b; 2015c –see *Appendix A. Responses to City of Edmonton Comments on the Preliminary Design*). This report should be considered as follow-up to the three earlier reports on wildlife passage conceptual design (Stantec Consulting Ltd. 2014a; 2014b; 2014c).

3.0 OPEN-SPAN BRIDGE STRUCTURE FOR LARGE, MEDIUM AND SMALL TERRESTRIAL SPECIES

The September 2015 preliminary design (3rd submission) for the large open-span crossing structure utilises a standard bridge cross section with an opening that is 14 m wide and 4.65 m deep under the structure. The total length of the structure is estimated at 32.3 m (Figure 1). There is also a 14 m^2 skylight in the median between the traffic lanes to increase natural light inside the structure. The design of this wildlife crossing structure also includes wing-walls at either end to minimize the length of the structure and help guide animals to the entrances. At this time, these wing-walls are planned to be constructed from mechanically stabilized earth (MSE), but this remains to be confirmed by the structural design team during the final design stage (Figure 2).

The large open-span crossing is situated at a 15 degree skew from perpendicular in order to improve sight lines for animals approaching the structure (Figure 1). The skew angle was modified from the 25 degrees indicated in the conceptual design (Stantec 2014c) since prefabricated bridge girders only come in angles of either 15 or 30 degrees. A 15 degree skew was chosen over a 30 degree skew to avoid lengthening the structure. As well, a 15 degree skew is preferred because it moves the western approach further from rip-rap associated with the existing storm outfall. Both of these factors should result in a more effective wildlife crossing structure with the 15 degree skew.

Open-span structures such as this have been shown to be effective for both large wildlife (e.g., deer, bears) and a variety of smaller species (Ruediger and DiGiorgio 2007). The dimensions of this large below-grade crossing structure is within the large animal design recommendations for length (<37 m; Cramer 2012), width (>12 m; Clevenger and Huijser 2011), and height (>4 m Clevenger and Huijser 2011).

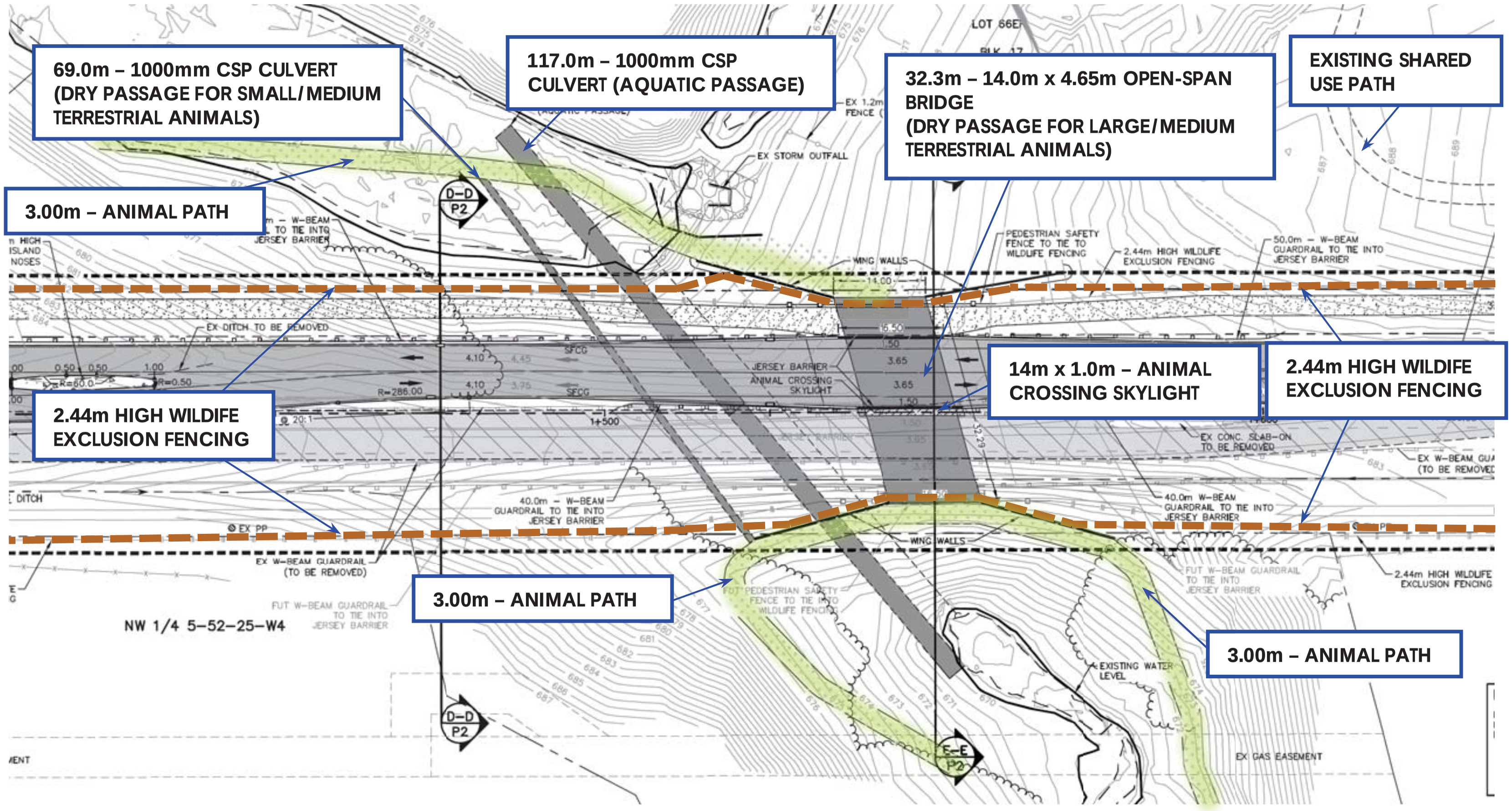


Figure 1 Wedgewood Creek Wildlife Crossings at 199 Street (Preliminary Engineering Plan – September 2015)

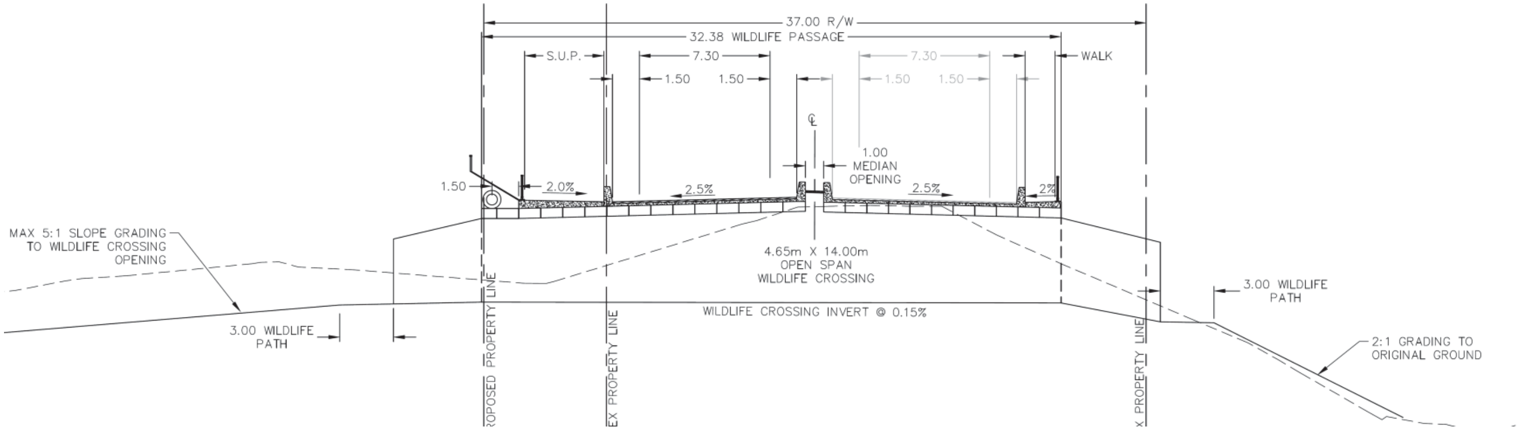


Figure 2 Preliminary Cross-Section of Large Open-Span Wildlife Crossing Structure (October 2015)

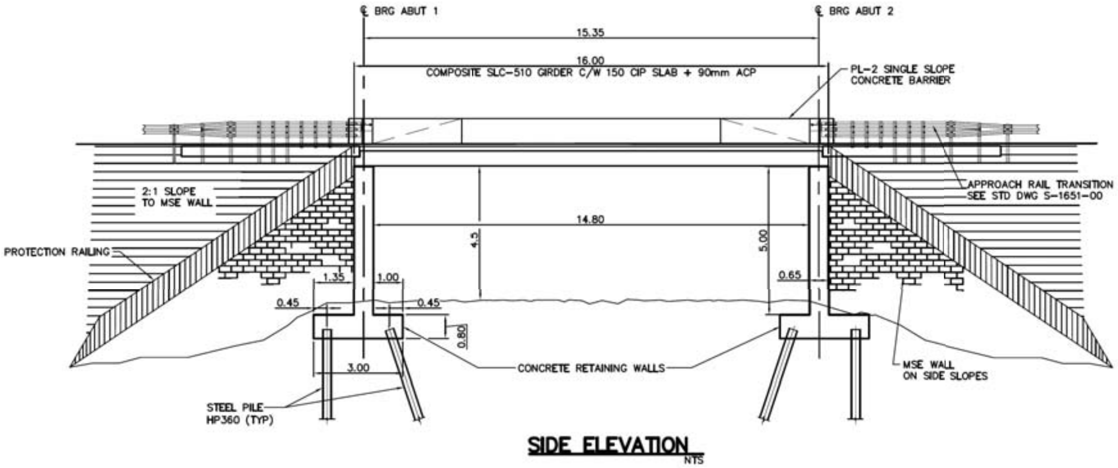


Figure 3 Preliminary Side Elevation of Large Open-Span Wildlife Crossing Structure (April 2015)[1]

---

[1] Appendix G in Stantec 2015

Although Clevenger and Huijser (2011) do not recommend the use of openness indices in planning and designing wildlife crossing structures, this metric has been calculated to provide a context for comparison to the Wildlife Passage Engineering Design Guidelines (WPEDG; City of Edmonton 2010). The WPEDG indicate an "optimal passage openness" of 1.5 is preferred for the Large Terrestrial EDG.

- The openness index for the November 2014 concept design of the large open-span bridge (14 x 4.5 x 30.9 m) was estimated at 2.04.

- The openness index for the April 2015 preliminary design (1st submission) of the large open-span bridge (14 x 4.5 x 36.9 m) was estimated at 1.71 due to an increase in the total length of 6 m to accommodate a skylight in the middle of the structure.

- The openness index for the September 2015 preliminary design (3rd submission) of the large open-span bridge (14 x 4.65 x 32.3 m) was estimated at 2.02. This was a result of a reduction in the total length of 4.5 m (due to a change in the skew and a reduced width of skylight to 1 m) and a small increase in the height of the structure.

The estimated openness index is well above the City of Edmonton (2010) recommendation of 1.5, even with the inclusion of the skylight.

Although excessive noise levels in wildlife crossing structures have the potential to reduce the crossing frequency of wildlife species, separating this effect from the other environmental characteristics has proven difficult. In Spain, the effect of noise on the use of 19 crossing structures by vertebrates was investigated along a major highway (Iglesias et al 2012). However, the diversity of species use and the crossing frequencies of lagomorphs and foxes were not correlated with any of the noise indicators. The only significant correlations found were positive; between the crossing frequencies of *Canis* sp. and small mustelids and maximum noise levels (Iglesias et al 2012). On the Trans-Canada highway in Banff National Park, noise was not a significant factor in the crossing performance for black bear, wolf and cougar (Clevenger and Waltho 2005). However, noise was negatively correlated for other species, and explained between 16 and 28% of the variation in the crossing performance of grizzly bear, elk and deer (Clevenger and Waltho 2005).

Noise levels on 199th Street are not expected to be as high as the two studies mentioned above, since both traffic frequency and average vehicle speed will be much less than on a major highway. As well, peak traffic levels on 199th Street are expected during daylight hours, which is outside the evening and crepuscular time periods that most animals are expected to use this structure. Regardless, potential design modifications of the skylight to reduce noise transmission into the crossing structure will be considered during final design phase.

The location of pedestrian and wildlife fencing has been modified from the 1st and 2nd submissions and has been configured to direct wildlife towards the large open-span crossing structure. The ends of the fence are located as close to the paved surface of 199th Street as safety tolerances allow. The fence ends do not angle away from highway since this would tend to funnel animals traveling parallel to 199th Street onto the paved surface and increase the potential for animal-vehicle collisions.

The preliminary design includes 3 m wide pathways to facilitate wildlife use of the large open-span crossing structure. Similar wildlife pathways have been used successfully east of Golden, British Columbia to facilitate wildlife approaches to crossing structures on the Trans-Canada Highway. The Washington State Department of Transport include pathways in the "Passage Enhancement Toolbox" as a way to improve the permeability of crossing structures for terrestrial wildlife (Washington Department of Transport 2015). These animal pathways are designed to be effectively used by large, medium, and small animals.

As indicated in the May 2014 conceptual design report (Stantec 2014a), passage requirements for Large Terrestrial, Medium Terrestrial, and Small Terrestrial EDGs will be addressed in this large open-span bridge crossing structure. Provision of hiding cover (e.g., tree branches and tree trunks) inside the structure will be included to encourage use of this structure by small mammals and reptiles (Connolly-Newman 2013). Specifications for small animal hiding cover will be provided as part of the landscaping plan to be developed during the final design stage.

4.0 DRY PASSAGE CULVERT FOR SMALL AND MEDIUM TERRESTRIAL SPECIES

The September 2015 preliminary design (3rd submission) proposes to use 1.0 m diameter CSP culvert (Figure 3) to provide dry passage for small and medium terrestrial species. The total length of the structure is estimated at 92.5 m (Figure 1).

The 2014 conceptual design reports (Stantec 2014a; 2014b; 2014c) suggested that passage requirements for the Small Terrestrial, Amphibian, and Aquatic Species EDGs could be addressed in a modified drainage culvert. However, due to the high levels of beaver activity in the area, it is likely that construction of a shelf along the culvert length, with ramps at either end to allow small terrestrial animal access, would cause an accumulation of debris at culvert inlet that would likely damage the shelf (Stantec 2015). This debris would also need to be periodically removed in order to maintain culvert function (Stantec 2015). Therefore, the current preliminary design has been modified, with dry passage proposed for Small and Medium Terrestrial EDGs in a crossing structure separate from the drainage culvert.

The 1.0 m diameter culvert is considered adequate for passage of small- and medium-sized animals (City of Edmonton 2010; Clevenger and Huijser 2011; Phillips et al. 2012). The 1.0 m diameter culvert is considered a "Class 1 Small Underpass" within Kintsch and Cramer's (2011) *Passage Assessment System* and has the potential to provide passage for the species movement guilds that include the target EDGs at this site (Medium Terrestrial and Small Terrestrial). Clevenger and Huijser (2011) have similar species guilds to Kintsch and Cramer (2011) and their "small to medium-size mammal underpass" includes drainage culverts up to 1.2 m in diameter. In California, coyote passage through two 60 m long, 1070 mm diameter drainage culverts has been documented (Phillips et al. 2012). As Clevenger and Huijser (2011) point out, high mobility medium-size mammals (includes coyote and fox) "*will typically use underpass or culvert designs sufficiently large enough so they can move through them*".

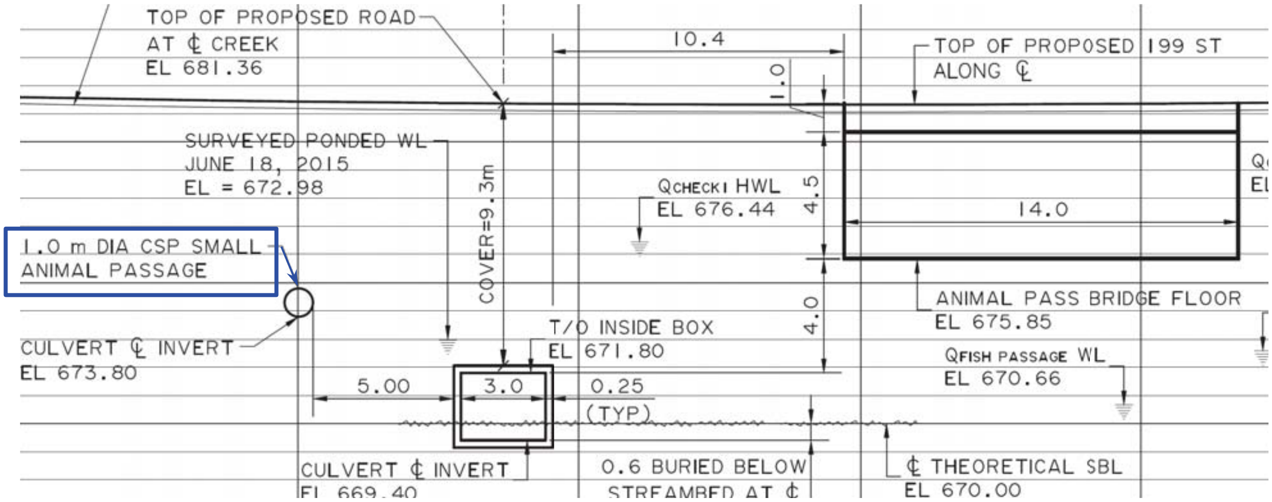


Figure 4 **Cross-sectional representation of the 1.0 m diameter small animal culvert, stream culvert (Concrete Box Culvert option) and open-span bridge.** [2]

5.0 WET PASSAGE CULVERT FOR AQUATIC SPECIES

Wood frog and boreal chorus frog have been detected in the vicinity of Wedgewood Ravine (Ecoventure 2013). Installation of an appropriately-sized concrete box or round culvert with substrate installed that addresses both hydrotechnical and fish passage concerns will adequately address passage requirement for amphibians. The *Hydrotechnical Summary Report* (Stantec 2015) proposed three options to replace the existing 1.8 m diameter closed bottom Structural Plate Corrugated Steel Pipe (SPCSP) Culvert:

- 3.0 m diameter closed-bottom Corrugated Steel Pipe (CSP) Culvert (117.5 m long)

---

[2] Appendix F in Stantec 2015

- 3.05 m diameter closed-bottom SPCSP Culvert (117.5 m long)
- 3.0 m span x 2.4 m rise Concrete Box Culvert (117.5 m long)

All of these proposed structures are designed to provide fish passage (Stantec 2015), and they conform to Kintsch and Cramer's (2011) "Class 1 Small Underpass", which includes drainage culverts. According to their system, this type of culvert has the potential to provide passage for the species movement guilds that include the target EDGs at this site, Amphibians and Aquatic Species. This type of structure is considered to be adequate to allow passage of small aquatic animals (City of Edmonton 2010; Clevenger and Huijser 2011; Phillips et al. 2012).

Aquatic Species EDGs are particularly sensitive to poorly designed crossing structures (City of Edmonton 2010). Issues of increased water velocity and poorly embedded structures can create a barrier to upstream movement of aquatic species. The three culvert options (concrete box, SPCSP, or CSP) have all been sized and positioned to minimize flow velocities, avoid confining the channel, and be sufficiently embedded in the stream channel to provide a natural substrate at the bottom of the culvert (Stantec 2015).

Due to high beaver dam activity in area, the aquatic crossing at 199 Street has also been designed to accommodate debris passage through the culvert (Stantec 2015). This accommodation of debris passage renders the raised platform to allow dry passage of small-sized animals through the aquatic culvert infeasible.

6.0 CONCLUSION

The City of Edmonton (2010) has identified 11 Ecological Design Groups (EDGs) to be addressed when planning and designing wildlife passage: Large Terrestrial, Medium Terrestrial, Small Terrestrial, Amphibians, Aerial Mammals, Aquatic Species, Scavenger Birds, Birds of Prey, Water Birds, Ground Dwelling Birds, and Other Birds. It is expected that one or more species within all 11 EDGs are predicted to occur in the vicinity of Wedgewood Creek (Stantec 2014a).

One of the best ways to maximize permeability of roads for wildlife is to include frequently spaced culverts of mixed size classes (Clevenger et al. 2001). The three crossing structures proposed for Wedgewood Creek provide a good example of this. Together, they are considered adequate to accommodate the passage requirements for all of the EDGs identified in the Stantec (2014a) report. Passage requirements for the Large Terrestrial, Medium Terrestrial, and Small Terrestrial EDGs are provided in the large open-span wildlife crossing structure. This structure is located approximately 5 m below the roadway, near the natural travel area for deer at the top of Wedgewood Ravine that was observed during the field assessment (Stantec 2014b).

Passage requirements for Medium Terrestrial and Small Terrestrial EDGs are also provided in the 1.0 m diameter dry passage culvert. Passage requirements for Amphibian and Aquatic Species EDGs are provided in the 3.0 m diameter drainage culvert associated with Wedgewood Creek. Passage requirements for the Aerial Mammals, Scavenger Birds, Birds of Prey, Ground Dwelling Birds, Water Birds and Other Birds EDGs will be adequately addressed above-grade in the Recommendations for Reducing Bird and Bat Vehicle Collision Risk (see Stantec 2014a). This involves natural vegetation and tree plantings that are used to direct the flight paths of birds and bats higher over the road, above the traffic (Stantec 2014a). This measure will also minimize the reduction in habitat created by the road right-of-way, and maintain the aesthetics of the area. To accomplish this measure, clearing of trees and vegetation will be minimized along 199 Street and tree plantings will be designed to grow taller than the highest vehicles using the road. Specifications for vegetation plantings will be included in the landscaping plan to be developed during the final design stage.

7.0 CLOSURE

This evaluation of conceptual design for wildlife passage on the 199 Street Widening Project within Riverview Neighbourhood 1 was prepared by Stantec Consulting Ltd. for the Riverview Owners Group. The material in it reflects Stantec's best judgment in light of the information available to it at the time of preparation. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibilities of such third parties. Stantec Consulting Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

Stantec has endeavored to incorporate the principles of the WPEDG into the 199 Street wildlife passage design and the constraints associated with the physical site characteristics and available materials. We trust that this information is sufficient to support the submission of the initial concept.

Respectfully submitted,

STANTEC CONSULTING LTD.

Bill Harper

William L. Harper, M.Sc., R.P.Bio.
Senior Wildlife Biologist

8.0 REFERENCES

CIMA+. 2014a. 199 Street concept planning report: 23 Avenue to 35 Avenue – Draft Submission. Draft concept plan report prepared by G. Campbell, P.Eng. for the Riverview Ownership Group, April 2014. 88 pp.

CIMA+. 2014b. 199 Street (23 Avenue to 35 Avenue), comments to your correspondence dated October 15, 2014. Letter to C. Wintle dated November 18, 2014 from G. Campbell. Includes wildlife evaluation of conceptual design (see Stantec Consulting Ltd. 2014c).

CIMA+. 2014c. 199 Street concept planning report: 23 Avenue to 35 Avenue – 2nd Submission. Draft concept plan report prepared by G. Campbell, P.Eng. for the Riverview Ownership Group, December 2014.

City of Edmonton. 2010. Wildlife passage engineering design guidelines. Report prepared by Stantec Consulting Ltd. for the Office of Natural Areas, City of Edmonton, AB. 249 pp. Accessed October 14, 2015, from: http://www.edmonton.ca/city_government/documents/PDF/WPEDG_FINAL_Aug_2010.pdf

City of Edmonton. 2014a. Review of Stantec's Wildlife Crossing Report by Facility and Capital Planning (FCP) and Office of Biodiversity (OBD). Letter to CIMA+ dated August 6, 2014 by N. Lazurko, Transportation Services, City of Edmonton, AB. 3 pp.

City of Edmonton. 2014b. Comments for document titled 199 Street (23 Avenue to 35 Avenue). Memo to N. Lazurko dated October 15, 2014 from C. Shier, City of Edmonton, AB. 3 pp.

City of Edmonton. 2014c. Comments for 199 Street crossing of Wedgewood Creek (May 15, 2014 concept drawings from CIMA). Memo to N. Lazurko dated December 4, 2014 from C. Shier, City of Edmonton, AB. 2 pp.

City of Edmonton. 2015a. Comments for 199 Street crossing of Wedgewood Creek (April 7, 2015 preliminary drawings from Stantec). Memo to F. Wambugu dated April 27, 2015 from C. Shier, City of Edmonton, AB. 2 pp.

City of Edmonton. 2015b. Comments for the EIA on the 199 street widening at Wedgewood Creek (GB15-10, received May 28, 2015). Memo to G. Buffalo dated June 19, 2015 from C. Shier, City of Edmonton, AB. 5 pp.

City of Edmonton. 2015c. 199 Street Preliminary Plan: 35 Avenue to 23 Avenue –Second Submission. Letter to P. Chamney dated August 11, 2015 from F. Wambugu, City of Edmonton, AB. 4 pp.

Clevenger, A.P., B. Chruszcz and K Gunson. 2001. Drainage culverts as habitat linkages and factors affecting passage by mammals. Journal of Applied Ecology 48: 1340-1349. Accessed on September 3, 2015, from: http://onlinelibrary.wiley.com/doi/10.1046/j.0021-8901.2001.00678.x/abstract

Clevenger, A.P. and N. Waltho. 2005. Performance indices to identify attributes of highway crossing structures facilitating movement of large mammals. Biological Conservation 121: 453–464. Accessed on October 14, 2015, from: http://www.sciencedirect.com/science/article/pii/S0006320704002319

Clevenger, A.P. and M.P. Huijser. 2011. Wildlife crossing structure handbook: Design and evaluation in North America. Report FHWA-CFL/TD-11-003 by the Western Transportation Institute, Bozeman, MT for the U.S. Federal Highway Administration, Central Federal Lands Highway Division, Lakewood, CO. 224 pp. Accessed on October 14, 2015, from: http://roadecology.ucdavis.edu/files/content/projects/DOT-FHWA_Wildlife_Crossing_Structures_Handbook.pdf

Connolly-Newman, H.R., M.P. Huijser, L. Broberg, C.R. Nelson, and W. Camel-Means. 2013. Effect of cover on small mammal movements through wildlife underpasses along us Highway 93 North, Montana, USA. Proceedings of the 2013 International Conference on Ecology and Transportation (ICOET 2013). Accessed on October 14, 2015, from: http://scholarworks.umt.edu/cgi/viewcontent.cgi?article=1369&context=etd

Cramer, P. 2012. Determining wildlife use of wildlife crossing structures under different scenarios. Report No. UT-12.07 prepared by Department of Wildland Resources and Utah Transportation Center, Utah State University, Logan, UT for the Utah Department of Transportation, Salt Lake City, Utah. 181 pp. Accessed on October 14, 2015, from: http://www.udot.utah.gov/main//uconowner.gf?n=10315521671291686

Ecoventure. 2013. Environmental screening report – Edgemont storm water management system. Report for Walton Edgemont Development Corporation, prepared by Ecoventure Inc., Edmonton, AB. 137 pp.

Iglesias, C., C. Mata and J.E. Malo. 2012. The influence of traffic noise on vertebrate road crossing. AMBIO 41: 193-201. Accessed on October 2, 2015, from: http://www.ncbi.nlm.nih.gov/pmc/articles/PMC3357842/

Kintsch, J. and P.C. Cramer. 2011. Permeability of existing structures for terrestrial wildlife: A passage assessment system. Research Report No. WA-RD 777.1. Washington State Department of Transportation, Olympia, WA. 188 pp. Accessed on October 14, 2015, from: http://www.wsdot.wa.gov/research/reports/fullreports/777.1.pdf

Phillips, J., R. Phillips, N. Srinivasan, D. Aso, W. Lao and P. Cornely. 2012. Safe passage for Coyote Valley: a wildlife linkage for the Highway 101 corridor. Kirsch Center for Environmental

Studies, De Anza College, Cupertino, CA. 35pp. Accessed on October 14, 2015, from: https://www.deanza.edu/es/wildlifecorrproj/Safe%20Passagelowres.pdf

Ruediger, B. and M. DiGiorgio. 2007. Safe Passage: A User's Guide to Developing Effective Highway crossings for Carnivores and other Wildlife. USDA Forest Service and Wilberforce. 19 pp. Accessed on October 14, 2015, from: http://www.wildlifeconsultingresources.com/pdf/Carnivore%20Safe%20PassageFinalSMALL.pdf

Stantec Consulting Ltd. 2014a. Wildlife passage design recommendations—199 Street widening project within Riverview Neighbourhood. Report prepared for the Riverview Owners Group, dated May 20, 2014. 15 pp.

Stantec Consulting Ltd. 2014b. Wildlife passage design—199 Street widening project within Riverview Neighbourhood 1. Report prepared for the Riverview Owners Group, dated September 18, 2014. 11 pp.

Stantec Consulting Ltd. 2014c. Appendix A – Evaluation of November 2014 conceptual design for wildlife passage design—199 Street widening project within Riverview Neighbourhood 1. Report prepared as part of CIMA+ letter to the City of Edmonton (2014b), dated November 18, 2014. 9 pp.

Stantec Consulting Ltd. 2015. Wedgewood Creek Crossing at 199 Street (NE 6-52-25-W4 and NW 5-52-25-W4, Edmonton, AB). Hydrotechnical Summary Report. Prepared for Qualico Communities. 22pp. + Appendices.

Washington Department of Transport. 2015. Passage Enhancement Toolbox (website). Accessed September 25, 2015, from: http://www.wsdot.wa.gov/NR/rdonlyres/AECC63E5-76FA-411B-9B28-15E1FB9388EF/0/PassageEnhanceToolbox.pdf

9.0 APPENDIX A. RESPONSES TO CITY OF EDMONTON COMMENTS ON THE PRELIMINARY DESIGN THAT PERTAIN TO WILDLIFE PASSAGE

The City of Edmonton has requested additional information pertaining to wildlife passage associated with the Project (City of Edmonton 2015a; 2015b; 2015c). The following is in response to these questions/concerns with the earlier preliminary designs (1st and 2nd submissions), particularly as they apply to open-span bridge structure[3].

1. ***City Comment on bridge structure (underpass):*** *What is the length of this structure? We require this to confirm that the openness ratio at full build is 2.0? (City of Edmonton 2015a; 2015b)*

   Stantec: The length of the large open-span crossing structure is 32.3 m, resulting in an estimated openness index at full build of 2.02.

2. ***City Comment on bridge structure (underpass):*** *Suggested changes to improve line of sight: Grading leading to (and out of) the passage was to be reviewed further and illustrated in the preliminary drawings. We indicated we would be looking for an evaluation of other ways to improve the line of sight, for example, through potential modifications to the wing walls and additional earth work with respect to grading. As expressed in the field, our concerns lie mainly with the western opening of the passage and its steep slope, its relation to the outfall, and pedestrian trail at top of bank. We note that the grading has been changed from 4:1 to 5:1 and that a 3 m wide animal path has been incorporated on both sides of the road. This "animal path" technique is new to our office. Has it been proven an effective technique in other structures, and is it to be designed for all EDG's or just large animals? Also, how with this animal path interact with the rip rap associated with the outfall on the west side of the road? Is the rip rap to be buried and covered with topsoil and vegetated? (City of Edmonton 2015a; 2015b; 2015c)*

   Stantec: The preliminary design includes 3 m wide pathways to facilitate wildlife use of the large open-span crossing structure. Similar wildlife pathways have been used successfully east of Golden, British Columbia to facilitate wildlife approaches to crossing structures on the Trans-Canada Highway. The Washington State Department of Transport include pathways in the "Passage Enhancement Toolbox" as a way to improve the permeability of crossing structures for terrestrial wildlife (Washington Department of Transport 2015). These animal pathways are designed to be effectively used by large,

---

[3] Stantec responses were developed with input from various discipline specialists, including Bill Harper and Stephanie Grossman (wildlife), Marc Obert (environmental science), and Petrea Chamney and Don Hall (engineering).

medium, and small animals. Since the rip rap associated with the storm water outfall does not interfere with the effectiveness of the wildlife crossing structure, it will not be necessary to treat the rip rap in any way.

3. ***City Comment on bridge structure (underpass):*** *We require an understanding of how "open" the structure truly is: While the passage itself (4.5m x 14m) may produce an openness ratio of 2.0 (depending on its length), we are interested to know what impact, if any, such large wing walls have on the functionality of the passage (or the perception of openness by wildlife). (City of Edmonton 2015a; 2015b)*
    a. *Please outline wing walls on the cross section of the passage.*
    b. *The SW wing wall has been modified since the concept drawings. Such a modification potentially reduces line of sight and as such is contrary to our earlier direction. Also, it directs wildlife to a small tributary across from which is the steepest part of the valley, so it is unclear how this improves passage.*
    c. *Also, there is mention on page 2.5 that a water main may be "supported on the bridge abutment walls" – will this in anyway decrease the size of the opening?*

    Stantec:

    a) The wing walls are outlined on Figures 2 and 3.

    b) The bridge structure (underpass) has been modified from the 25 degree skew indicated in the conceptual drawing, since prefabricated bridge girders only come in angles of either 15 or 30 degrees. A 15 degree skew was chosen over a 30 degree skew in order to avoid lengthening the structure. As well, a 15 degree skew is preferred because it moves the western approach further from rip-rap associated with the existing storm outfall. Both of these factors should result in a more effective wildlife crossing structure using the 15 degree skew.
    c) The water main will be resting on a girder that is the same depth as the bridge girders and will therefore have no impact on the size of the large animal underpass opening.

4. ***City Comment on bridge structure (underpass):*** *An open median will be a requirement and we will be looking for the applicant to safely narrow the cross-section of the road within the ravine. (City of Edmonton 2015a; 2015b)*

    Stantec: The cross-section of the road within the ravine has been narrowed.

5. ***City Comment on bridge structure (underpass):*** *Ensure appropriate fencing for both wildlife and people management. (City of Edmonton 2015a; 2015b)*
    a. *We note that the wildlife fence does little to direct wildlife to the passage.*

b. *Please ensure that the ends of the fence angle away from the road and follows along top of bank (or earlier if required)*

c. *Please confirm in this EIA there is no conflict between top of bank pedestrian trails and the wildlife fence. On page 7.13 it indicates that there is potential for pedestrian path users to influence wildlife use of the crossing structure. Please explain further as it is unclear if this is in reference to the TOB SUP or some other human use.*

Stantec: The location of pedestrian and wildlife fencing has been modified from the 1st and 2nd submissions and has been configured to direct wildlife towards the large open-span crossing structure. The ends of the fence are located as close to the paved surface of 199th Street as safety tolerances allow. The fence ends do not angle away from highway since this would tend to funnel animals traveling parallel to 199th Street onto the paved surface and increase the potential for animal-vehicle collisions. The potential for conflict between the top of bank pedestrian trail and wildlife fence will be addressed in an updated EIA during the final design stage.

6. ***City Comment on bridge structure (underpass):*** *Please confirm in this EIA that the only drainage access required can be provided from the arterial roadway through a locked gate in the wildlife exclusionary fence. (City of Edmonton 2015a; 2015b)*

   Stantec: At this stage of the design, it is not anticipated that drainage access will be required from the arterial roadway (199th Street) since a shared use path (Figure 1) exists just north of Wedgwood Creek. However, if drainage access is required from 199th Street it will be through a locked gate in the wildlife exclusion fence.

7. ***City Comment on bridge structure (underpass):*** *Given the passage is directly under the road (and is more enclosed than a full span bridge would be), is noise to be a deterrent to wildlife use? (City of Edmonton 2015a; 2015b)*

   Stantec: Although excessively noise levels in wildlife crossing structures has the potential to reduce the crossing frequency of wildlife species, separating this effect from the other environmental characteristics has proven difficult. In Spain, the effect of noise on the use by vertebrates of 19 crossing structures was investigated along major highway (Iglesias et al 2012). However, the diversity of species use and the crossing frequencies of lagomorphs and foxes were not correlated with any of the noise indicators. The only significant correlations found were positive; between the crossing frequencies of *Canis* sp. and small mustelids and maximum noise levels (Iglesias et al 2012). On the Trans-Canada highway in Banff National Park, noise was not a significant factor in the crossing performance for black bear, wolf and cougar (Clevenger and Waltho 2005). However, noise was negatively correlated for other species, and explained between 16 and 28% of the variation in the crossing performance of grizzly bear, elk and deer (Clevenger and Waltho 2005.

Noise levels on 199th Street are not expected to be as high as the two studies mentioned above, since both traffic frequency and average vehicle speed will be much less than on a major highway. Regardless, potential design modifications of the skylight to reduce noise transmission into the crossing structure will be considered during final design phase.

8. **City Comment on bridge structure (underpass):** *Please outline design considerations (to be implemented at detailed design) in the bridge structure for the use of this passage for small/medium EDGs. (City of Edmonton 2015a; 2015b)*

   Stantec: Provision of hiding cover (e.g., tree branches and tree trunks) inside the structure will be included to encourage use of this structure by small mammals and reptiles. Specifications for small animal hiding cover will be provided as part of the landscaping plan to be developed during the final design stage.

9. **City Comment on bridge structure (underpass):** *Provide clear recommendations for vegetation/landscaping of the wing walls and fence leading up to the structure to make the passage appear as natural as possible. We are looking at the option of a mechanically stabilized earth (MSE) wall at a similar passage, has this been considered here? A landscape plan has been included in all other wildlife passage bridge structure EIAs – this EIA would greatly benefit from such a figure. Outline potential locations for habitat restoration around proposed crossings to further offset the negative impacts of having the road widened (City of Edmonton 2015a; 2015b: 2015c)*

   Stantec: The wing-walls at either end of the large open-span crossing structure are designed to minimize the length of the structure and help guide animals to the entrances. The current design is for these wing-walls will be constructed from mechanically stabilized earth (MSE). Specifications for vegetation plantings to provide security cover and achieve other habitat restoration objectives will be included in the landscaping plan to be developed during the final design stage.

10. **City Comment on bridge structure (underpass):** *Significant issue: please outline where road drainage is to be directed and if it has the potential to negatively impact the wildlife passage. (City of Edmonton 2015a; 2015b)*
    a. *There is little in the report that speaks to the impact of the potential road drainage proposals on the functionality of the wildlife passage. Our office is particularly concerned with:*
        i. *the addition of any further rip rap to deal with road drainage (either from a new outfall or a new major drainage overflow route) and its impacts of wildlife movement in an area that already has a number of significant site constraints for wildlife passage (e.g. manhole, outfall and associated rip rap, poor grades on the west side of the passage), and*
        ii. *The installation of underground infrastructure that may impact the ability to adjust grading leading out of the terrestrial wildlife passage (as outlined above).*

   b. *Please clearly outline on the plan the extent of the major drainage overflow route, what infrastructure (e.g. rip rap) is required to support it, and evaluate how this would impact the wildlife structure and wildlife passage in the area.*
   c. *Note that we are not in a position to support this option until the impacts to the wildlife passage are understood.*
   d. *Please also outline the impacts to the creek due to the increased velocity from 1.01 m/s to 1.35 m/s. The ESR associated with this outfall and its impacts did not address (nor allow for) such a change in velocity.*

Stantec: Road drainage is not anticipated to adversely affect wildlife movements at the approaches of the wildlife crossing structures. Provisions for road drainage will be detailed at the final design stage, and wildlife access to all the wildlife crossing structures will be considered during road drainage design. Should installation of additional rip-rap be required, alternatives such as culverts will be pursued if it is determined that the rip-rap would jeopardize the function of the wildlife crossing structures.

The exit velocity from the pipe exiting to the rip rap apron is 1.35 m/s maximum. A rip rap apron will be installed to allow the flow to spread and decrease in height such that the velocity at the end of the apron entering the stream flow is reduced to 0.90 m/s.

11. ***City Comment on bridge structure (underpass):*** *Provide direction on how to minimize light pollution including recommended placement of light standards (that do not impact road safety. (City of Edmonton 2015a; 2015b)*

    Stantec: Street lighting with reduced spill and glare will be incorporated in the final design. In addition, within constraints required to appropriately illuminate the street, the lighting design will avoid illuminating the entrances of the wildlife crossing structures and nearby natural features. These details will be developed during the final design stage.

12. ***City Comment on bridge structure (underpass):*** *Provide direction on diversionary strategies for birds and bats (to move them up and over the road). (City of Edmonton 2015b)*

    Stantec: Natural vegetation and tree plantings will be used to direct the flight paths of birds and bats higher over the road, above the traffic (Stantec 2014a). This measure will also minimize the reduction in habitat created by the road right-of-way, and maintain the aesthetics of the area. To accomplish this measure, clearing of trees and vegetation will be minimized along 199 Street and tree plantings will be designed to grow taller than the highest vehicles using the road. Specifications for vegetation plantings will be included in the landscaping plan to be developed during the final design stage.

13. ***City Comment on aquatic passage (culvert):*** *What is the slope and discharge velocity of this culvert? Is it acceptable for fish passage? (City of Edmonton 2015a; 2015b*

Stantec: The slope of this culvert is 1.40%, which maintains a discharge velocity of 2.4 cubic meters per second.

14. ***City Comment on aquatic passage (culvert):*** *Open-bottom culverts with natural substrate are preferred to the option presented (2.4m corrugated pipe). Analysis on this option needs to be completed. (City of Edmonton 2015a; 2015b)*

    Stantec: Our office has prepared a Hydrotechnical Summary Report for the Wedgewood Creek Crossing at 199 Street (Stantec 2015) which details the options evaluated in the design of the aquatic passage, and the justification behind the option recommended by Stantec.

15. ***City Comment on aquatic passage (culvert):*** *How deep will the replacement culvert be embedded? (City of Edmonton 2015a; 2015b)*

    Stantec: The replacement culvert will be embedded 0.75m or ¼ of the pipe.

Stantec

Memo

To:	Catherine Shier City of Edmonton	From:	Marc Obert Stantec Consulting Ltd.
File:	1161103725	Date:	April 4, 2016

Reference: **199 Street NW/Wedgewood Creek EIA Comment Response Support (GB15-10) - Small Terrestrial Wildlife Passage Options**

The small wildlife passage crossing structure is one component of a three wildlife crossing structure system at Wedgewood Creek that is designed to provide passage for small mammals. This structure may also function at some level for medium-size animals but that was not the target ecological design group (EDG) for this particular structure. The three wildlife crossing structure system at Wedgewood Creek is consistent with recommendations of Clevenger et al.[1], "*To maximize connectivity across roads for mammals, future road construction schemes should include frequently spaced culverts of mixed size classes and should have abundant vegetative cover present near culvert entrances*." Studies in Banff National Park have found that species that travel in burrows and runway systems (e.g., weasels and rodents) prefer less open culverts, while other species (e.g., coyote and snowshoe hare) prefer more open structures[1]. McDonald and St. Clair[2] also found smaller culverts (0.3 m diameter) were more effective in maintaining permeability for small rodents (deer mice and voles) than larger 3 m diameter structures.

In order to address the following comment: … "the use of a 1 m CSP for the small/medium mammal passage is not recommended by the Wildlife Passage Engineering Design Guidelines. This passage is to be designed as either an open bottom culver or box culvert with acceptable openness ratio and substrate." We have evaluated the current location and have generated additional options to consider (including the original one presented). The framework used to generate the options included:

- Different sizes of culverts (1 m diameter, 1.2 x 1.8 m concrete box, bottomless arch)
- Different geographic locations (both north and south of the open span large terrestrial wildlife passage)
- Different elevations (same elevation as the open span bridge versus closer to creek level where there is likely more small mammal activities)
- No culvert

From these criterions, eight options were generated and compared (see Table 1, attached). To understand the site constraints, a cross section (Figure 1, attached) depicting the utility, location, and water level (i.e. 1:100 and 1:200) constraints that limit the size/location of culvert that can be installed has been provided for illustration purposes. In addition, all the options presented have been shown in plan view (Figure 2, attached) in relation to the proposed large terrestrial and aquatic passage alignments.

---

[1] Clevenger, A.P., B Chruszcz, and K. Gunson. 2001. Drainage culverts as habitat linkages and factors affecting passage by mammals. Journal of Applied Ecology 38: 1340–1349.

[2] McDonald, W. and C.C. St. Clair. 2004. Elements that promote highway crossing structure use by small mammals in Banff National Park. Journal of Applied Ecology 41: 82–93.

All the proposed designs for the small wildlife passage in Table 1 (attached) are consistent with the "small-to-medium-sized mammal underpass" described in the Wildlife Crossing Structure Handbook[3] that is considered a "recommended/optimal solution" for fisher, marten, weasel, small mammals and reptiles (Table 5 on p. 63[3]). The dimensions of Clevenger and Huijser's[3] "small-to-medium-sized mammal underpass" generally range from 0.4 to 1.2 m in diameter. At the larger end of this range, it is recommended that cover requirements for smaller fauna be met by placing pipes of varying diameter in the culvert that span the entire length[3].

Of the eight options presented, four are considered the best at maintaining permeability for small mammals as part of the three wildlife crossing structure system. From a wildlife passage standpoint, the preferred options are:

- Option 1 (1 m diameter culvert currently proposed)
- Option 2 (1.2 x 1.8 m concrete box)
- Option 5 (1 m diameter culvert nearer creek)
- Option 6 (1.2 x 1.8 m concrete box nearer creek)

The other four options are not preferred, since the size or location of these structures does not function as well at maintaining permeability for small mammals (the target EDG for this structure).

Please note, Options 1 and 2 ensure that the bottom of the mid slope terrestrial passage will be above the design water level (i.e. 1:100) and the 1:200 year flood.

Sincerely,
STANTEC CONSULTING LTD.

Bill Harper

William L. Harper, M.Sc., R.P.Bio.
Senior Wildlife Biologist
Phone: (250) 655-5394
bill.harper@stantec.com

Marc Obert B.Sc., P. Biol., P.Ag.
Environmental Scientist, Environmental Services
Phone: (780) 969 2194
marc.obert@stantec.com

Attachments: Figure 1 – Site Constraints
Figure 2 - mid slope Wildlife Passage Options
Table1 - 199 Street NW Mid slope Wildlife Passage, Comparison of Options

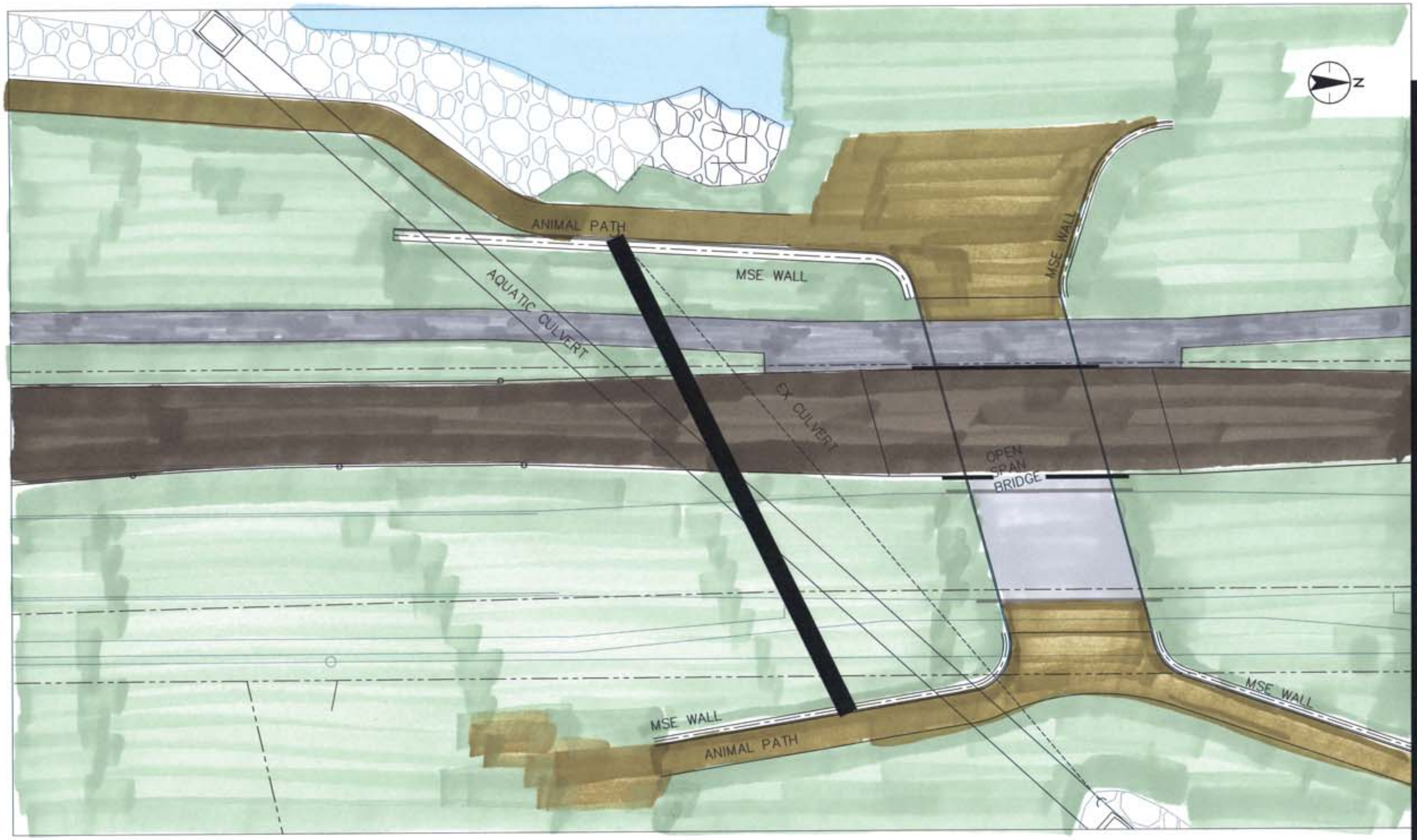
---

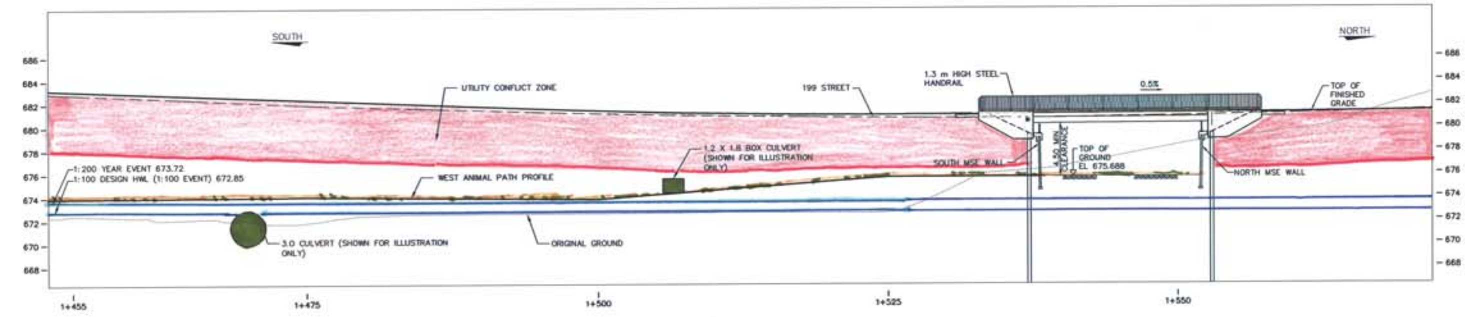
[3] Clevenger, A.P. and M.P. Huijser. 2011. Wildlife crossing structure handbook: Design and evaluation in North America. Report FHWA-CFL/TD-11-003 by the Western Transportation Institute, Bozeman, MT for the U.S. Federal Highway Administration, Central Federal Lands Highway Division, Lakewood, CO. 224 pp.

ATTACHMENTS

TABLE 1: 199 STREET NW MID SLOPE WILDLIFE PASSAGE - COMPARISON OF OPTIONS

	CROSS SECTION	MATERIAL	SIZE	LENGTH	PROS	CONS
OPTION 1 (current design)	Circular	CSP	1.0m dia	51.4m	-Optimal placement to minimize length -Both entrances are above the design water level -Good permeability for small mammals due to smaller opening size	-Reduces permeability for medium-sized animals due to smaller opening size (note that the large terrestrial passage is available approximately 3 m (meter) north of small terrestrial passage
OPTION 2 (same length as current design)	Box	Concrete	1.2m rise x 1.8m wide	51.4m	-Optimal placement to minimize length -Both entrances are above the design water level -Increases permeability for medium-sized mammals due to larger opening size -Similar size was approved for construction to the north along 199 Street NW	-Reduces permeability for small mammals due to larger opening size
OPTION 3	No Structure				-Zero maintenance	-Greatly reduces permeability for small mammals with loss of structure specifically designed for small mammals. Also somewhat reduces permeability for medium-sized mammals, but large open span structure is also designed for medium-sized animals.
OPTION 4 (north of open span passage)	Box	Concrete	1.2m rise x 1.8m wide	45.0m	-Away from the design water level -Small increase in permeability for medium-sized mammals due to shorter length -Similar size was approved for construction to the north along 199 Street NW	-Creates redundancies due to close proximity to large terrestrial passage -Interferes with the MSE wall -Potential proximity to human conflict -Juxtaposition with open span crossing structure reduce overall wildlife permeability (2 structures at the same location)
OPTION 5 (lowered closer to design water level)	Circular	CSP	1.0m dia.	77.0m	-Relocated closer to creek level -Increases permeability for small mammals due to smaller opening size and since it is closer to riparian habitats associated with the creek.	-Length increases compared to current design -CSP culvert is used -Reduces permeability for medium-sized mammals due to increased length and smaller opening size
OPTION 6 (lowered closer to design water level)	Box	Concrete	1.2m rise x 1.8m wide	77.0m	-Relocated closer to creek level-Increases permeability for medium-sized mammals (larger opening size) and increases permeability for small mammals since it is closer to riparian habitats associated with the creek. -Similar size was approved for construction to the north along 199 Street NW	-Length increases compared to current design-Reduces permeability for medium-sized mammals due to increased length
OPTION 7 (Hold west side and re-align east side)	Box	Concrete	1.2m rise x 1.8 wide	49.0m	-Slightly shorter length compared to current design -Small increase in permeability for medium-sized mammals due to shorter length -Similar size was approved for construction to the north along 199 Street NW	-Orientates east side further away from the passage. -Increases slope -Reduced permeability for small mammals associated with riparian habitats near the creek because the east entrance is located further from the creek.
OPTION 8 (same alignment as current design)	Arch	CSP	1.5m rise x 3.5m wide	51.4m	-Open bottom -Both entrances are above the design water level -Increased permeability for medium-sized mammals due to larger opening size	-Requires additional structures (footings, piles, etc.) that would conflict with aquatic culvert. If moved north to remove these conflicts would physically interfere with large terrestrial passage. If moved south, the required length of the structure would increase. -Effective area is not practical. Smallest span available is 3.5m with a 1.5m height. The only way to make this fit is by moving up the slope and increasing the length. -Greatly reduces permeability for small mammals due to larger opening size





Stantec

Stantec Consulting Ltd.
10160 - 112 Street
Edmonton AB Canada
Tel. 780.917.7000
www.stantec.com

Copyright Reserved

Legend

Notes

Development Engineer, Sustainable Development

Approvals YY.MM

Revision	By	Appd.	YY.MM
1. FIRST SUBMISSION	G.R.	T.C.	15.12
Issued	By	Appd.	YY.MM

PROJECT MANAGER: T. CHIARELLO DESIGN: G.R. DRAWN: G.R.

Permit-Seal

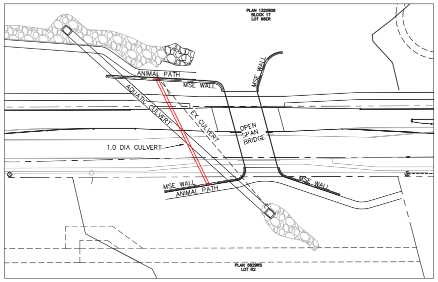
Client/Project
RIVERVIEW HEIGHTS ESTATES LTD.

199 STREET
23 AVENUE NW TO 35 AVENUE NW
Edmonton, AB

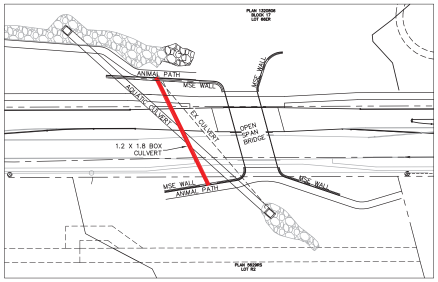
Title
WEDGEWOOD CREEK CROSSING
SITE CONSTRAINTS

Project No. 1161-103725
Scale NTS

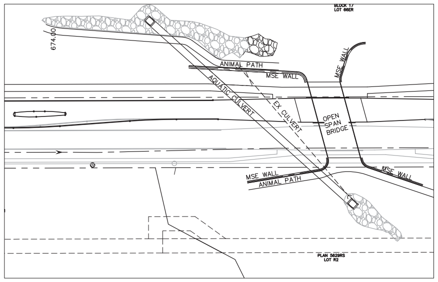
[illegible]
Drawing No. 1



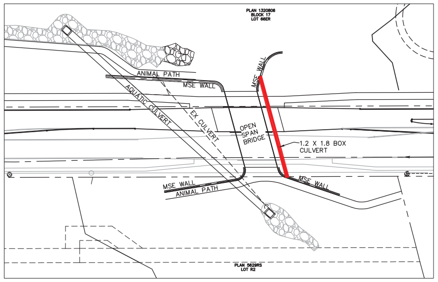
OPTION 1
1.0m DIA CSP ORIGINAL DESIGN



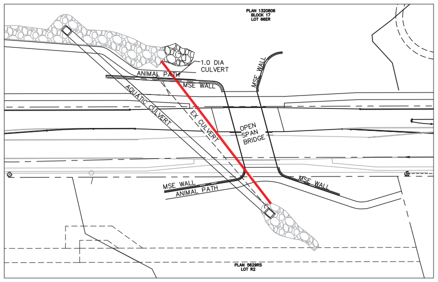
OPTION 2
1.2m RISE X 1.8m WIDE BOX CULVERT



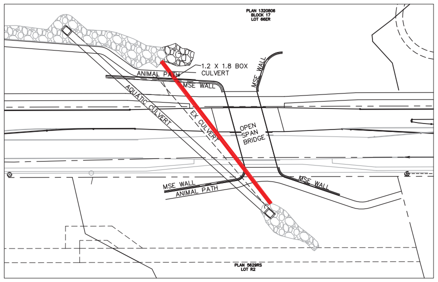
OPTION 3
NO MID SLOPE PASSAGE INCORPORATED



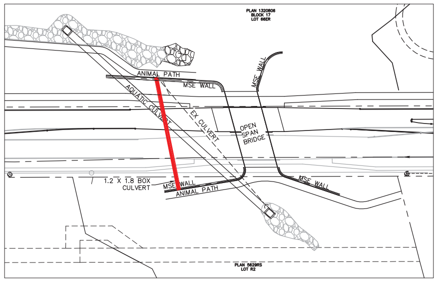
OPTION 4
1.2m RISE X 1.8m WIDE BOX CULVERT
NORTH OF OPEN SPAN PASSAGE



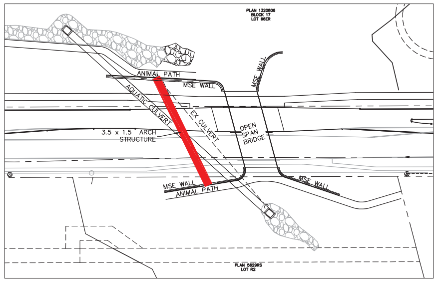
OPTION 5
1.0m DIA CSP
LOWERED TO CLOSER TO HWL



OPTION 6
1.2m RISE X 1.8m WIDE BOX CULVERT
LOWERED TO CLOSER TO HWL



OPTION 7
1.2m RISE X 1.8m WIDE BOX CULVERT
ALIGNMENT SHIFTED



OPTION 8
1.5m RISE X 3.5m WIDE ARCH STRUCTURE

N

Stantec

Stantec Consulting Ltd.
10160 - 112 Street
Edmonton AB Canada
Tel. 780.917.7000
www.stantec.com

Copyright Reserved

Legend

Notes

Development Engineer, Sustainable Development

Approvals			YY.MM
Revision	By	Appd.	YY.MM
1. FIRST SUBMISSION	G.R.	T.C.	15.12
Issued	By	Appd.	YY.MM

PROJECT MANAGER: T. CHIARELLO
DESIGN: G.R.
DRAWN: G.R.

Permit-Seal

Client/Project
RIVERVIEW HEIGHTS ESTATES LTD.

199 STREET
23 AVENUE NW TO 35 AVENUE NW
Edmonton, AB

Title
MID SLOPE WILDLIFE PASSAGE
ALIGNMENT & OPTIONS

Project No.
1161-103725

Scale
NTS

V:\1161\active\1161103725\project_resource\20160321_Med_wildlife passage options_TCedit.dwg
2016-4-03 1:28pm BY: TCHIARELLO

Drawing No.
2

Stantec
Stantec Consulting Ltd.
10160 – 112 Street, Edmonton AB T5K 2L6

January 13, 2016
File: 1102-19229

Urban Planning and Environment
12th Floor, HSBC Bank Place
10250 – 101 Street NW
Edmonton, AB, T5J 3P4

Attention: Catherine Shier, M.Sc., P.Biol.

Dear Ms. Shier:

Reference: 199th Street NW Road Upgrade – Wedgewood Passage Review

The comments received via email on December 20, 2015 from Catherine Shier, Ecology Unit, Parks + Biodiversity, Edmonton, AB, were reviewed by Stantec's design team consisting of Marc Obert, William Harper, Petrea Chamney, and Tony Chiarello. Responses to comments are as follows:

C1.	**… As discussed on the phone last week, with the information that has been provided to date, I think it is important for the consulting team to respond to the solution that is suggested below. Now, unknown to the City, it may be that this solution was considered, but was determined unfeasible due to other constraints. If such a discussion was held, then it should be outlined in the EIA so that both Administration and Council can understand that alternative solutions were explored in an attempt to avoid the large number of constraints that are associated with the current placement of the passage north of the stream.** **On the other hand, if this solution was not considered, it is worth responding – especially if it is only a land ownership issue (which may be resolved through some form of Conservation Easement). Ultimately, it is important for us to give Council an understanding that the solution that we will ultimately be presenting to them is the best one available to us (and is sure to produce the best results for the investment).**
R1**.**	Refer to R3 for our response regarding the proposed solution. As discussed, the Alternatives Section within the EIA will be updated to reflect the additional information now available.
C2a.	**Detailed review of the environmental impacts of this design will occur with review of the EIA which has not yet been submitted for this detailed design drawing set. At this time, however, it should be noted that a number of changes around the wildlife passage from that approved in the concept and first preliminary designs are presented in this package:** **1) Reduction in skew angle from 25 to 15 degrees (impact to line of sight through the passage needs to be further reviewed)**
R2a.	Per the Preliminary Plan 3rd Submission letter (18 September 2015): "Wildlife Crossing Structure Skew Change: The design of the wildlife crossing structure has been changed from the original 25° skew shown in the approved Concept Plan. The Preliminary Engineering plan now shows a 15° skew of the structure. It was brought to our attention that the prefabricated bridge girders

	used for construction of the wildlife passage structure only come in either 30° or 15° skew angles. As noted by William Harper (Stantec's Wildlife Biologist), the 15° skew angle is preferable to the 30° skew because it moves the western approach further from the rip-rap associated with the existing storm outfall and results in a shorter overall length of structure. Both of these advantages should result in a more effective wildlife crossing structure. Attached is a figure showing the line of sight of the 15° skew design." A review of sight lines through the structure indicates that there is no change between the 15° skew compared to the 25° skew. For animals at the east entrance looking west, they can see approximately 30 to 40 meters of habitat beyond the west exit of the structure (see attached). For animals at the west entrance looking east, they can see approximately 30 to 60 plus meters (m) of habitat beyond the east exit of the structure. These sight line estimates apply to both skew angles. Both designs provide sufficient clear view (sight lines) on the other side of the structure to encourage through passage of large animals (e.g., deer).
C2b.	**2) Introduction of an overflow channel to deal with road drainage (depending on the treatment of this recently introduced channel, there could be impacts wildlife passage functionality)**
R2b.	Per the Preliminary Plan 3rd Submission letter (18 September 2015): "With the construction of the wildlife crossing structure the low point will be pushed further south, off of the structure, where catch basins can be placed. An overflow point will be added down the embankment for times of major flow. A drainage report regarding this issue (accounting for the roadway drainage) had been submitted to the City's Drainage department for review and verbal acceptance of this report has been received from Drainage." The drainage swales included in the detailed engineering drawings are aligned from the low point in the roadway (i.e. the overflow point during major, rarer, storm events), along the top of the MSE wall structure, discharging away from the wildlife passage. The attached sketch further illustrates that the discharge locations from the swales are away from the openings of the wildlife passage to prohibit the interaction with wildlife.
C2c.	**3) The requirement to add a new small/medium mammal passage due to inability to address the needs of small/medium terrestrial wildlife due to required modifications.**
R2c.	The new small/medium mammal passage is an improvement over the conceptual design that called for a dry ledge to be installed within the aquatic passage. The modified aquatic culvert design was not viable because high level of beaver activity in the area (and debris associated with beaver activities) would likely interfere with any dry passage shelves installed within the culvert. The new structure provides a separate passage that is shorter and purpose-built for small/medium mammal passage. The original location and length of the small/medium terrestrial passage presented in the 3rd submission preliminary plan has been revised in the detailed design from 117.00m to 51.44m and shifted to the north to aid in the reduction in length. This overall configuration has been reviewed and supported by William Harper. The profile of the small/medium terrestrial passage is shown on drawing C105-012 attached.
C2d.	**4) The introduction of a 3m wildlife path likely due to the fact that there has been an**

	inability to resolve the problem of slope into/out of the passage and implications to line of sight (a requirement identified at concept level)
R2d.	The 3m wide wildlife paths have always been designed to provide north-south movement along the fill slopes on either side of the roadway (Stantec 2014) and are independent of the east-west slope grading into and out of the crossing structure. The slope in and out of the passage was revised to 5H:1V, based on our previous comment response, thus negating the initial concerns regarding the approach slope. Additional information to support the path has been provided per the Preliminary Plan 3rd Submission letter (18 September 2015): "Yes, animal pathways have been used successfully on the Trans-Canada Highway east of Golden, British Columbia to facilitate wildlife (deer, elk and bighorn sheep) approaches to crossing structures. The Washington State Department of Transport includes pathways in the "Passage Enhancement Toolbox" as a way to improve the permeability of crossing structures for terrestrial wildlife (see http://www.wsdot.wa.gov/NR/rdonlyres/AECC63E5-76FA-411B-9B28-15E1FB9388EF/0/PassageEnhanceToolbox.pdf)." "The pathway is designed to be effectively used by large, medium and small animals." "The animal path is located east of the rip rap associated with the outfall and therefore does not interact with it. The rip rap will not interfere with the effectiveness of the wildlife crossing structure, and therefore vegetating it will not be necessary."
C2e.	**5) A reduction (from 5 to 1 m) in the originally proposed meridian.**
R2e.	Per the Preliminary Plan 3rd Submission letter (18 September 2015): "The cross-section shown within the approved Concept Plan for 199 Street showed a 4.5m boulevard. This cross-section does not account for any motorist safety measures associated with a bridge crossing structure. Once the jersey barriers and shy distance between the edge of driving lane and the barriers were accounted for, the actual clear median distance will only be 1.0m as shown in our preliminary engineering plan. These safety measures are all shown within City of Edmonton roadway details as well as the Transportation Association of Canada (TAC) manuals." Given the geometry of the wildlife crossing presented in the approved Concept Plan, the median width is 1.0m as described above. Although an increase of the median will allow more light, Clevenger and Huisjer (2011) recommend that a shorter structure, with less daytime light and lower noise levels, will be more effective than crossing structures with large open medians. This recommendation is based primarily on structure length and traffic noise levels (Clevenger and Huisjer 2011). The current design with a $14m^2$ median is considered a good compromise for providing ample natural lighting in the structure, while keeping traffic noise to a minimum. In addition, any increase to the median width would also increase the length of the wildlife and aquatic passage. This would reduce their effectiveness and increase the overall disturbance footprint within the Wedgewood Creek ravine. Throughout the process we have always strived for the optimum strategy based on site constraints, engineering constraints, and the ecological needs of the target EDGs.
C2f.	**6) Increased length of aquatic passage**
R2f.	The aquatic passage has been designed to reproduce, as much as possible, the natural

	hydraulic conditions of Wedgewood Creek in order to provide flow velocities and minimum depths that permit upstream movement of aquatic species during low flow conditions. Although the length of the aquatic passage culvert has increased slightly from the preliminary design to the detailed design (117m to 122m), it is still shorter than the initial conceptual design (150m). Refer to drawing C105-012 (attached) for the length at detailed design. Any reorientations to shorten the structure will be a move away from the assessed optimum.
C3.	**Given the information provided, it appears that the first four design challenges may be solved by keeping the earlier design of the wildlife passage structure the same, but moving the terrestrial passage south of the creek. Such a relocation would also move the passage further away from the constraints resulting from the existing outfall/rip rap, manhole, and access road. By moving the structure away from these constraints, there would also be a reduction in the accessibility by people to use the wildlife passage – thereby further promoting its intended use for wildlife. Please be sure to respond to this potential solution through the Environmental Review process. Thank you."**
R3.	The project team examined the option of moving the large wildlife crossing structure and has come across other concerns outside private land issues to the east. As stated previously, we have always strived to present the optimum design solutions that take into consideration site constraints, engineering constraints, and the ecological needs of the target EDGs. However, we also need to account for downstream effects of the proposed project. The design of the aquatic passage has been optimized as described in Response R2f including the alignment set along the existing stream bed. We consider the location and alignment of the culvert a constant and the other structures were strategically located taking this in mind. In order to generate a new optimal large wildlife structure location we would need to reorient the drainage culvert. Any changes from this optimum could result in negative downstream effects (e.g., accelerated bank erosion). In accordance to federal and provincial legislation all water-related projects need to minimize any negative effects. If the large mammal passage is moved south of the creek, it will need to be aligned parallel to the aquatic passage to avoid conflict with wing walls, piles, etc. This orientation will increase the length of the passage substantially thereby increasing the cross-section required to maintain the openness ratio of 2.0. In this case, the angle of the wildlife passage would be approximately 50° hindering line of sight. In addition, the approach east of 199 Street would need to be located further up the slope, and given the steep conditions on the east side, sloping to existing at 5H: 1V would be difficult to achieve. To minimize the sloping required into the existing terrain, it is both practical and logical to locate the wildlife passage near to the lowest point (see attached) in the roadway as it is currently designed.

Reference: **199th Street NW Road Upgrade – Wedgewood Passage Review**

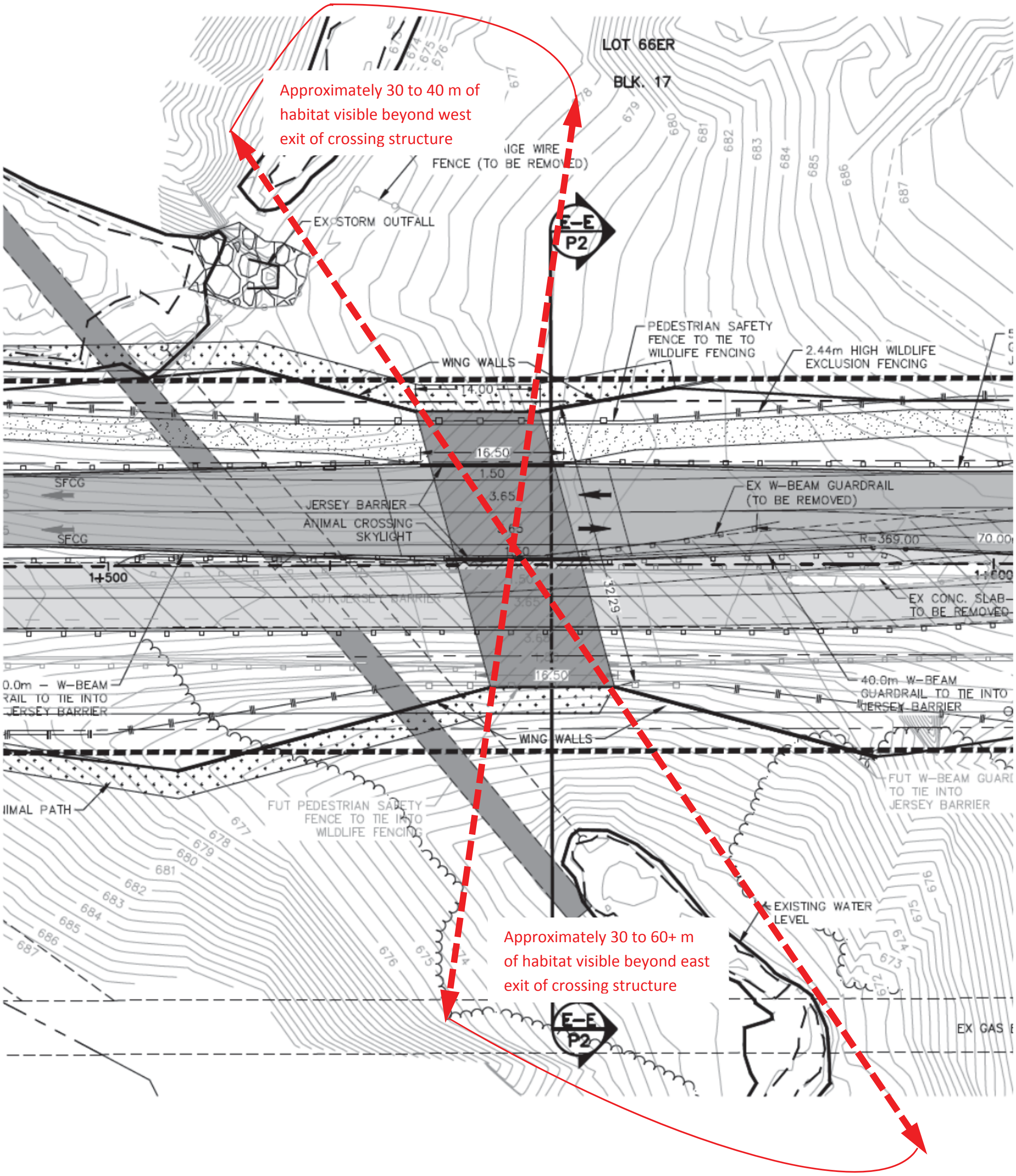
I hope this addressed all of your concerns. Please do not hesitate to contact the undersigned if you have any further questions.

Sincerely,

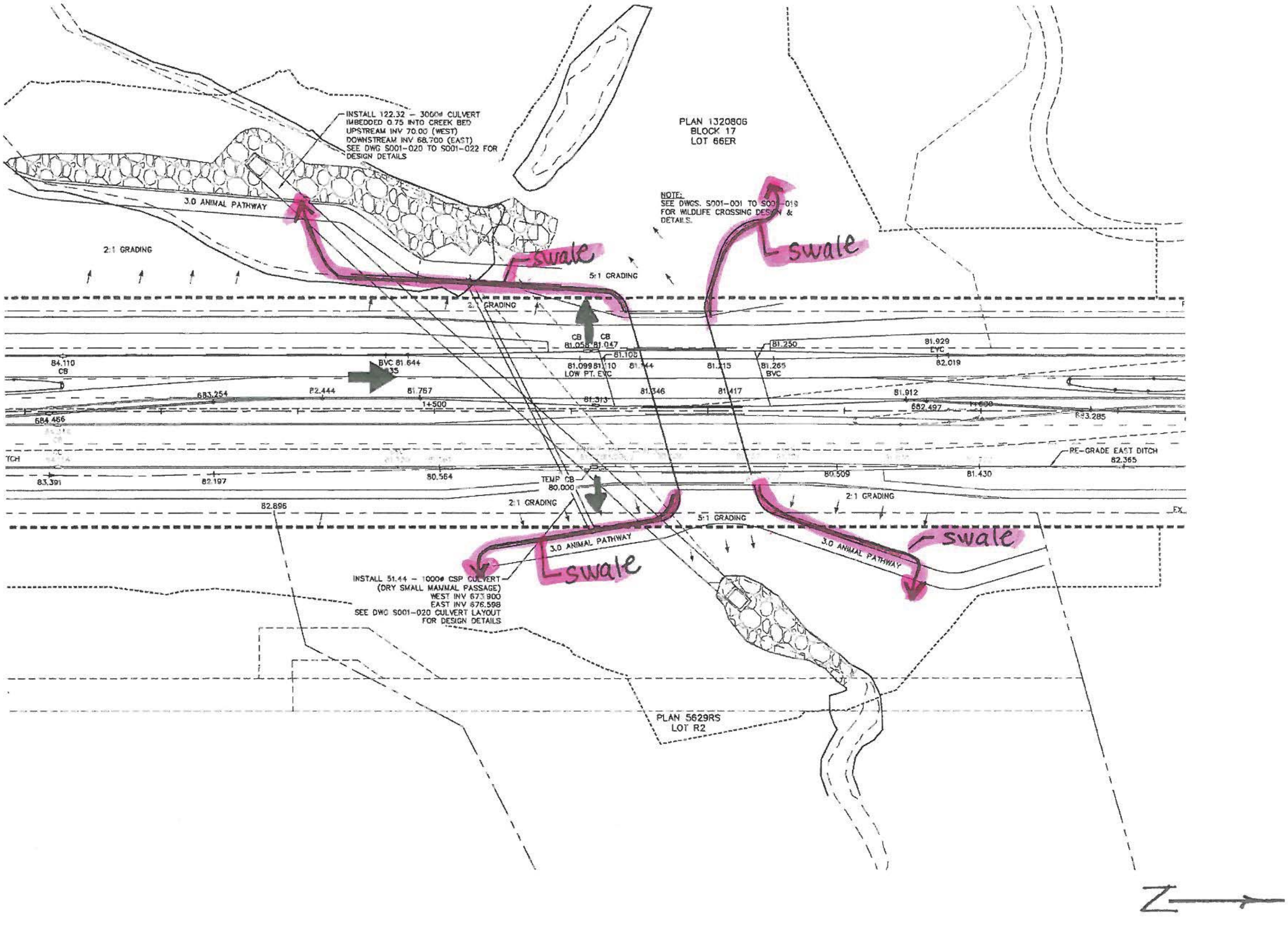
STANTEC CONSULTING LTD.

Marc Obert, B.Sc., P.Ag., P.Biol.
Environmental Scientist
Tel: (780) 969-2194
Fax: (780) 917-7249
marc.obert@stantec.com

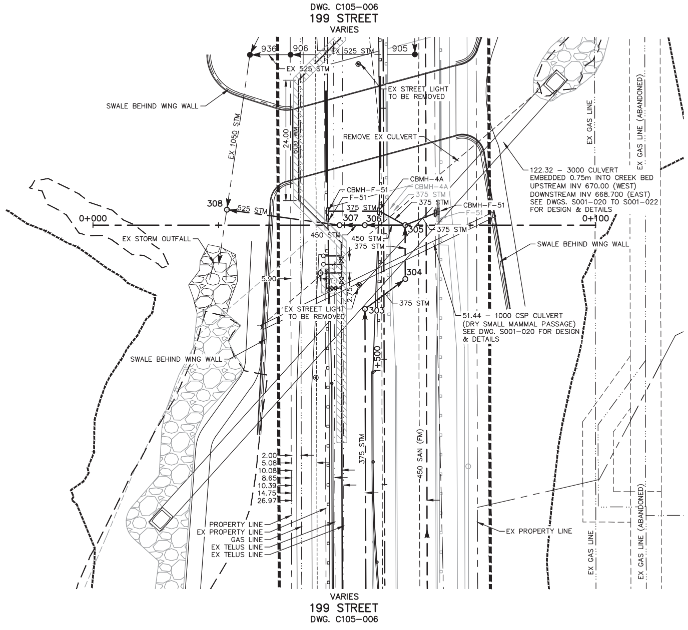
Attachment: Support Drawings

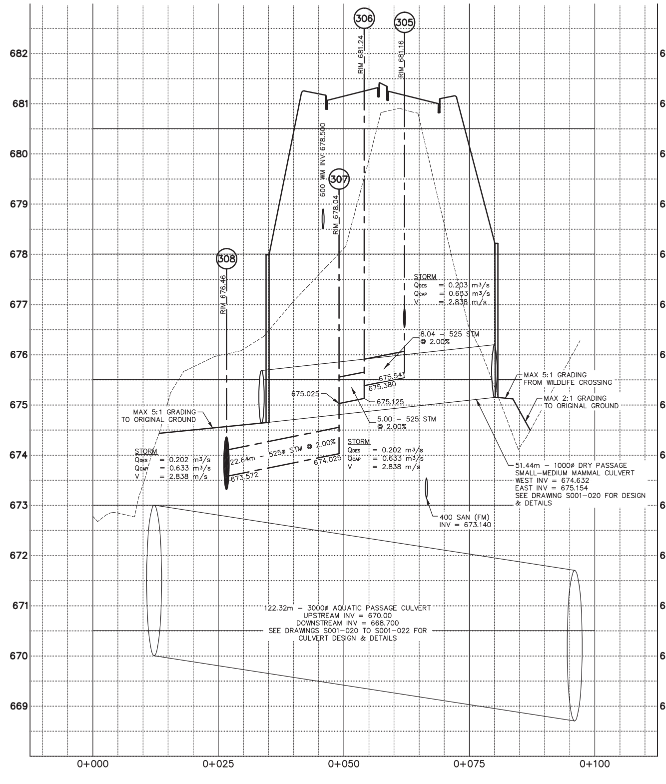


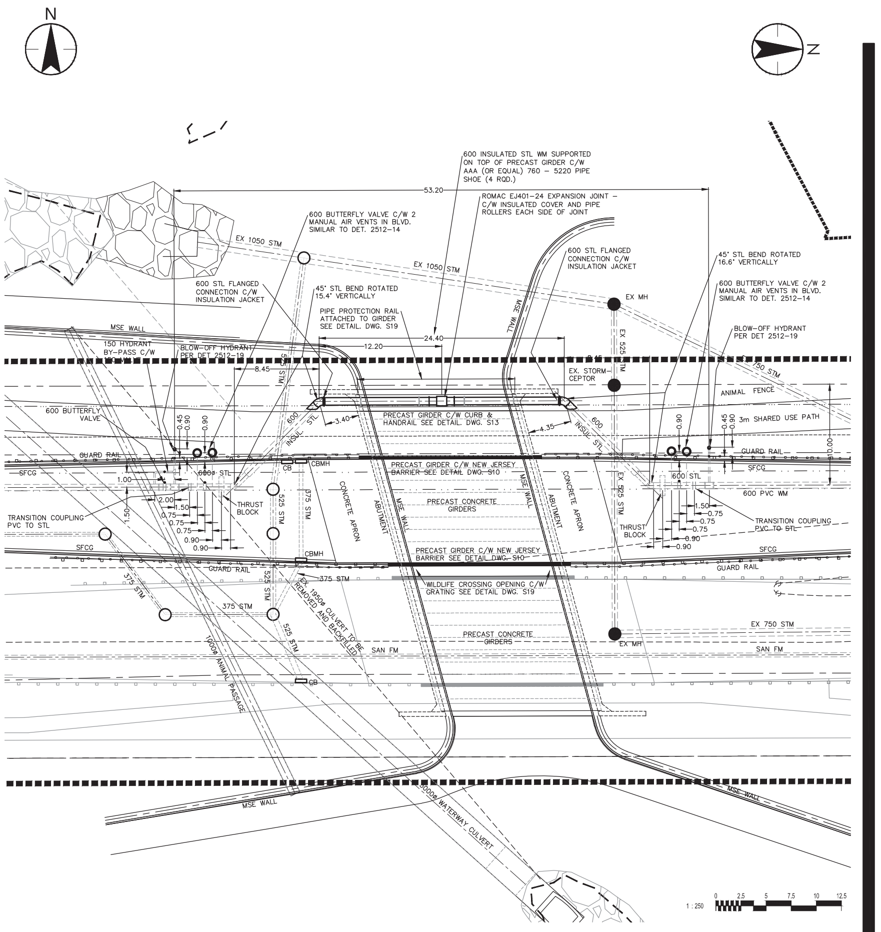
Sight-lines associated with the 15 skew (30 to 60+ m beyond exits)


INSTALL 122.32 – 3000Ø CULVERT
IMBEDDED 0.75 INTO CREEK BED
UPSTREAM INV 70.00 (WEST)
DOWNSTREAM INV 68.700 (EAST)
SEE DWG S001-020 TO S001-022 FOR
DESIGN DETAILS
PLAN 1320806
BLOCK 17
LOT 66ER
NOTE:
SEE DWGS. S001-001 TO S001-019
FOR WILDLIFE CROSSING DESIGN &
DETAILS.
3.0 ANIMAL PATHWAY
2:1 GRADING
5:1 GRADING
2:1 GRADING
swale
swale
swale
swale
CB
81.058
CB
81.047
81.105
81.099
81.110
LOW PT. EVC
81.144
81.215
81.250
81.265
BVC
81.929
EVC
82.019
84.110
CB
BVC 81.644
683.254
82.444
81.767
1+500
81.313
81.346
81.417
81.912
682.497
1+600
683.285
684.456
TCH
83.391
82.197
80.564
TEMP CB
80.000
80.509
81.430
RE-GRADE EAST DITCH
82.365
82.896
2:1 GRADING
5:1 GRADING
2:1 GRADING
FX
3.0 ANIMAL PATHWAY
3.0 ANIMAL PATHWAY
INSTALL 51.44 – 1000Ø CSP CULVERT
(DRY SMALL MAMMAL PASSAGE)
WEST INV 673.800
EAST INV 676.598
SEE DWG S001-020 CULVERT LAYOUT
FOR DESIGN DETAILS
PLAN 5629RS
LOT R2
N

1:1000







WATERMAIN BRIDGE CROSSING DETAIL
DWG. C105-006

MIN. DEPTH OF WATERMAIN INVERT FROM:			
PIPE DIA.	L.O.G. R.F.	L.O.G. S.F.	T.O.C.
50 mm	2.83	2.76	2.89
150 mm	2.83	2.76	2.89
200 mm	2.86	2.79	2.92
250 mm	2.88	2.81	2.94
300 mm	2.91	2.84	2.97
350 mm	2.94	2.87	3.00
400 mm	2.96	2.89	3.02
450 mm	2.99	2.92	3.05
600 mm	3.07	3.00	3.13
750 mm	3.15	3.08	3.21
900 mm	3.23	3.16	3.29

Stantec Consulting Ltd.
10160 - 112 Street
Edmonton AB Canada
Tel. 780.917.7000
www.stantec.com

Copyright Reserved

Legend

▬ ▬ ▬ LIMIT OF CONSTRUCTION
■ ■ ■ DEVELOPMENT BOUNDARY

Notes

1. WATERMAINS 150mm TO 300mm ARE AWWA C–900 DR–18 PVC PIPE WITH CLASS "B" BEDDING UNLESS OTHERWISE NOTED.
2. WATERMAINS 350mm AND LARGER ARE AWWA C–905 DR–25 PVC PIPE WITH CLASS "B" BEDDING UNLESS OTHERWISE NOTED.
3. APPLY APPROVED PVC WATER PIPE JOINT LUBRICANT FOR POTABLE WATER, ACCORDING TO MANUFACTURER'S RECOMMENDED APPLICATION PROCEDURES.
4. CB LEADS MUST BE A MINIMUM OF 3.0m FROM VALVE CASING.
5. ALL MATERIALS AND CONSTRUCTION TO BE AS PER THE CITY OF EDMONTON DESIGN AND CONSTRUCTION STANDARDS.
6. SANITARY SEWER PIPE SHALL BE TYPE PSM PVC SDR35 CONFORMING TO ASTM D3034, INSTALLED ACCORDING TO ASTM D2321. ALL EMBEDMENT MATERIALS SHALL BE TYPE 2, COMPACTED TO TOP OF EMBEDMENT TO 95% STANDARD PROCTOR DENSITY UNLESS OTHERWISE NOTED.
7. STORM SEWER PIPE ALTERNATIVES:
   A) CONCRETE PIPE:
   ALL CONCRETE PIPE TO BE SULPHATE RESISTANT WITH RUBBER GASKET CONFINED O–RING. BEDDING TO BE TYPE 2 UNLESS OTHERWISE NOTED.
   300mm – 525mm ASTM C14–CLIII, UNLESS OTHERWISE NOTED.
   600mm – 1800mm ASTM C76–CLIII, UNLESS OTHERWISE NOTED.
   B) PVC PIPE: (SEE NOTE 9. FOR BEDDING REQUIREMENTS)
   200mm – 525mm ASTM D3034 SDR35
   600mm – 1200mm ASTM F–679 SDR35
   PROFILE (RIBBED) CONFORMING TO CSA B182.4
8. ALL SEWER SERVICES ARE ASTM D3034 SDR35 PVC PIPE.
9. SANITARY SERVICES ARE 2.75m BELOW LOT GRADE UNLESS OTHERWISE NOTED.
10. ALL CB LEADS ARE 250mm SDR35 PVC WITH MIN. SLOPE OF 1.00% UNLESS OTHERWISE NOTED.
11. CATCHBASINS WITH A F–51, F–51G OR DK–7 FRAME AND COVER ARE 900ø. ALL OTHER CATCHBASINS ARE 600ø.
12. TIE INTO EXISTING TO THE SATISFACTION OF THE CITY OF EDMONTON.
13. OVERSIZED MANHOLE DETAIL ON DRAWING C200–XXX–DETAILS.

Development Engineer, Sustainable Development

Approvals YY.MM

Revision	By	Appd.	YY.MM
1. FIRST SUBMISSION	G.R.	T.C.	15.12
Issued	By	Appd.	YY.MM

PROJECT MANAGER: T. CHIARELLO
DESIGN: G.R.
DRAWN: G.R.

Permit-Seal

Client/Project
RIVERVIEW HEIGHTS ESTATES LTD.

199 STREET
23 AVENUE NW TO 35 AVENUE NW
Edmonton, AB

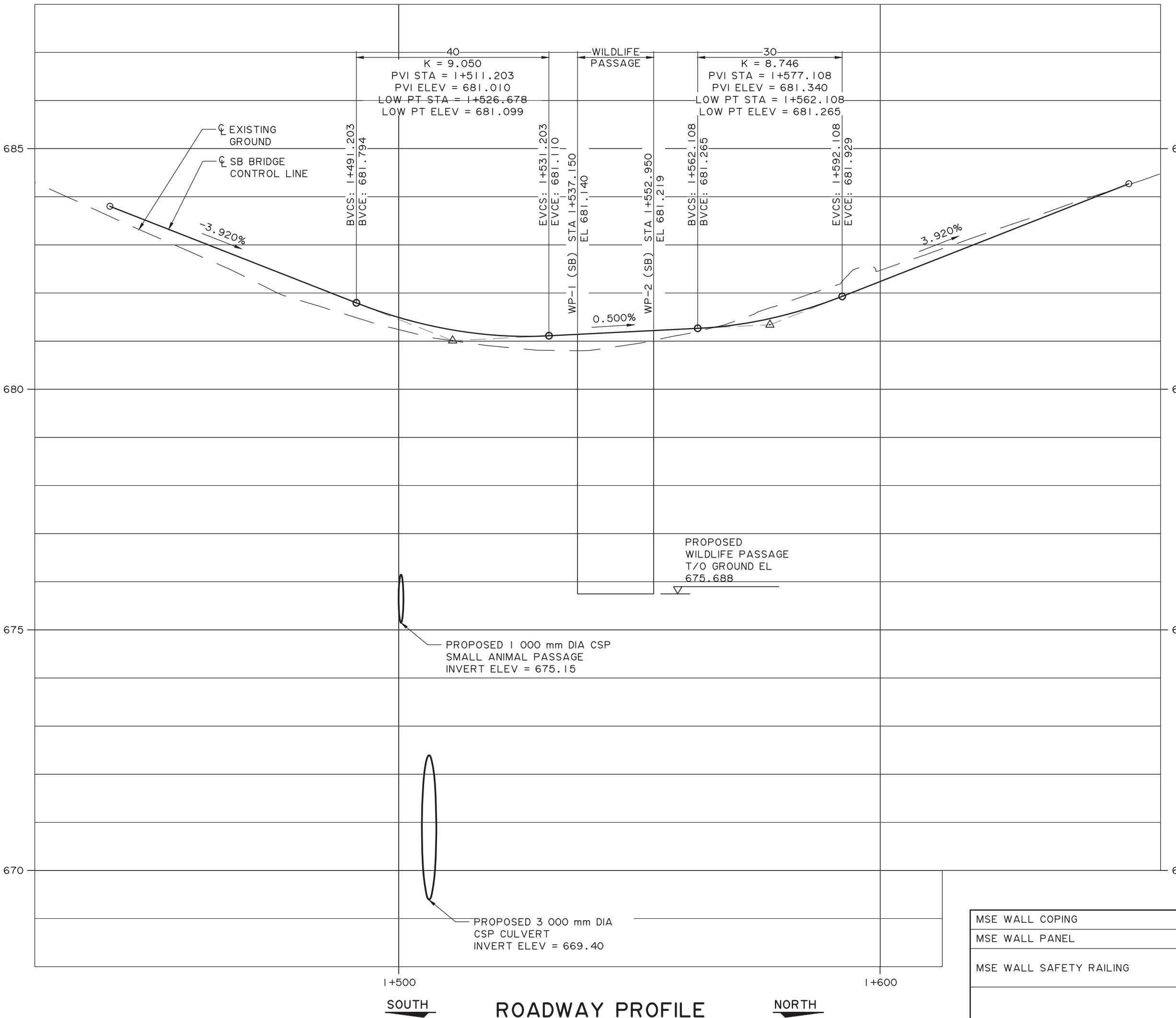
Title
WEDGEWOOD CREEK
UTILITIES CROSSING PLAN

Project No.
1161-103725

Scale
H 1:500 0 5 10 15 20 25
V 1:50 0 0.5 1.0 1.5 2.0 2.5

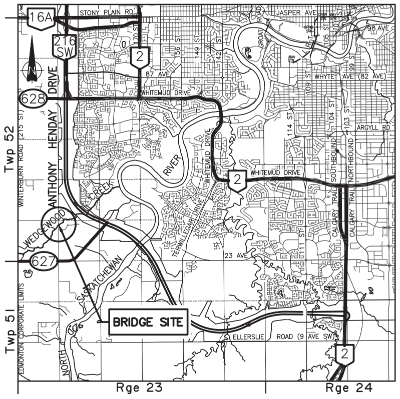
V:\1161\active\1161103725\drawing\sheet_files\1161103725 - C105 - UTILITIES CROSSINGS.dwg
2015-12-04 10:33am BY: GRUSSELL

Drawing No.
C105-012



ROADWAY PROFILE

HOR 1:500
VERT 1:50



SITE MAP

1:100 000

- PRIMARY HIGHWAY 1-216
- SECONDARY HIGHWAY 500-986
- LOCAL ROAD

SURVEY BY

- STANTEC CONSULTING LTD.

CONTROL POINTS

- ASCM 346890 N 5925450.255 E 22286.026 EL 689.519
- ASCM 479196 N 5926232.759 E 22296.323 EL 686.154

NOTES

- ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
- ROADWAY DESIGN STANDARD URBAN ARTERIAL ROAD DESIGN SPEED 70 km/h
- DESIGN SPECIFICATION: CAN/CSA S6-14 WITH CL800 DESIGN LIVE LOAD

ITEM		UNIT	SUBSTR	SUPERSTR	TOTAL EST	AS CONST
MSE WALL COPING		m	205		205	
MSE WALL PANEL		m²	880		880	
MSE WALL SAFETY RAILING	750 m HIGH	m	66		66	
	1070 m HIGH	m	140		140	
PRECAST SLC GIRDER	EXTERIOR (G1)	UNIT		1	1	
	INTERIOR	UNIT		18	18	
	BARRIER	UNIT		4	4	
	CURB	UNIT		2	2	
APPROACH RAIL TRANSITION		UNIT		4	4	
W-BEAM GUARDRAIL		m		232	232	
FLEAT 350 END TREATMENT		UNIT		4	4	
WATERPROOFING SYSTEM		m²		708	708	
WEARING SURFACE: TYPE H2 ACP (80 mm)		tonnes		116	116	
CONCRETE - CLASS HPC		m³				
CONCRETE - CLASS C		m³				
REINFORCING STEEL	STAINLESS	kg				
	PLAIN	kg				
PILING - HP310x79	SUPPLY & DRIVE	m	816		816	
	SET-UP	PILE	32		32	

QUANTITY ESTIMATE

Stantec Consulting Ltd.
10160 - 112 Street
Edmonton AB Canada
Tel. 780.917.7000
www.stantec.com

Copyright Reserved

Legend

Notes

Development Engineer, Sustainable Development

Approvals YY.MM

Revision By Appd. YY.MM

FIRST SUBMISSION R.C. T.C. 15.12.04

Issued By Appd. YY.MM

PROJECT MANAGER: D.H./T.C. RC DESIGN: R.C. DRAWN: J.D.C.

Permit-Seal

Client/Project

RIVERVIEW HEIGHTS ESTATES LTD.

199 STREET
N. OF 23 AVENUE TO S. OF 35 AVENUE
EDMONTON, AB

Title

WEDGEWOOD CREEK WILDLIFE CROSSING
INFORMATION SHEET 2

Project No. 1161-103725

Scale

W:\active\1161103725\detailed_design\drawings\S03 INFORMATION SHEET 2.dwg
2015-12-03 9:29am BY: ECLAY

Drawing No. S03

Riverview 199 Street Drainage System at Wedgewood Creek

Stantec

Prepared for:
Riverview Heights Estates Ltd.
Riverview Land Company Ltd.

March, 2015

Table of Contents

1.0 INTRODUCTION

As a component of the overall infrastructure for the proposed Riverview NH, The existing 199 Street arterial roadway will be extended south from the Edgemont NH through the Wedgewood Creek ravine to provide the required roadway level of service for Riverview.

Previously a new storm outfall to Wedgewood Creek was constructed on the north side of the Wedgewood ravine, which conveys pre-development regulated flows from a series of storm water management facilities (SWMF) and a segment of the 199 Street roadway drainage within Edgemont.

Upon review of the Riverview Neighbourhood Design Report (NDR), January 2015 prepared by MMM Group, and our review of the 199 Street roadway drainage through the Wedgewood ravine, it is not feasible to drain this area to any of the Riverview onsite SWMF.

This preliminary design brief outlines the methodology and design criteria used to assess and determine a practical conveyance system for the 199 Street drainage at Wedgewood Creek.

2.0 EXISTING AND ULTIMATE 199 STREET DRAINAGE

2.1 Existing 199 Street Drainage (Pre-Development)

The existing 199 Street drainage area to Wedgewood Creek was interpolated from the original ground contours provided within the Lidar topographical information for the Riverview NH. The drainage area shown on Figure 1 is 4.35 ha in size and is comprised of the existing rural roadway, drainage channels and some overland drainage from the adjacent properties.

The existing (original) 199 Street drainage currently drains off the roadway embankment into the ditches and /or Wedgewood Creek.

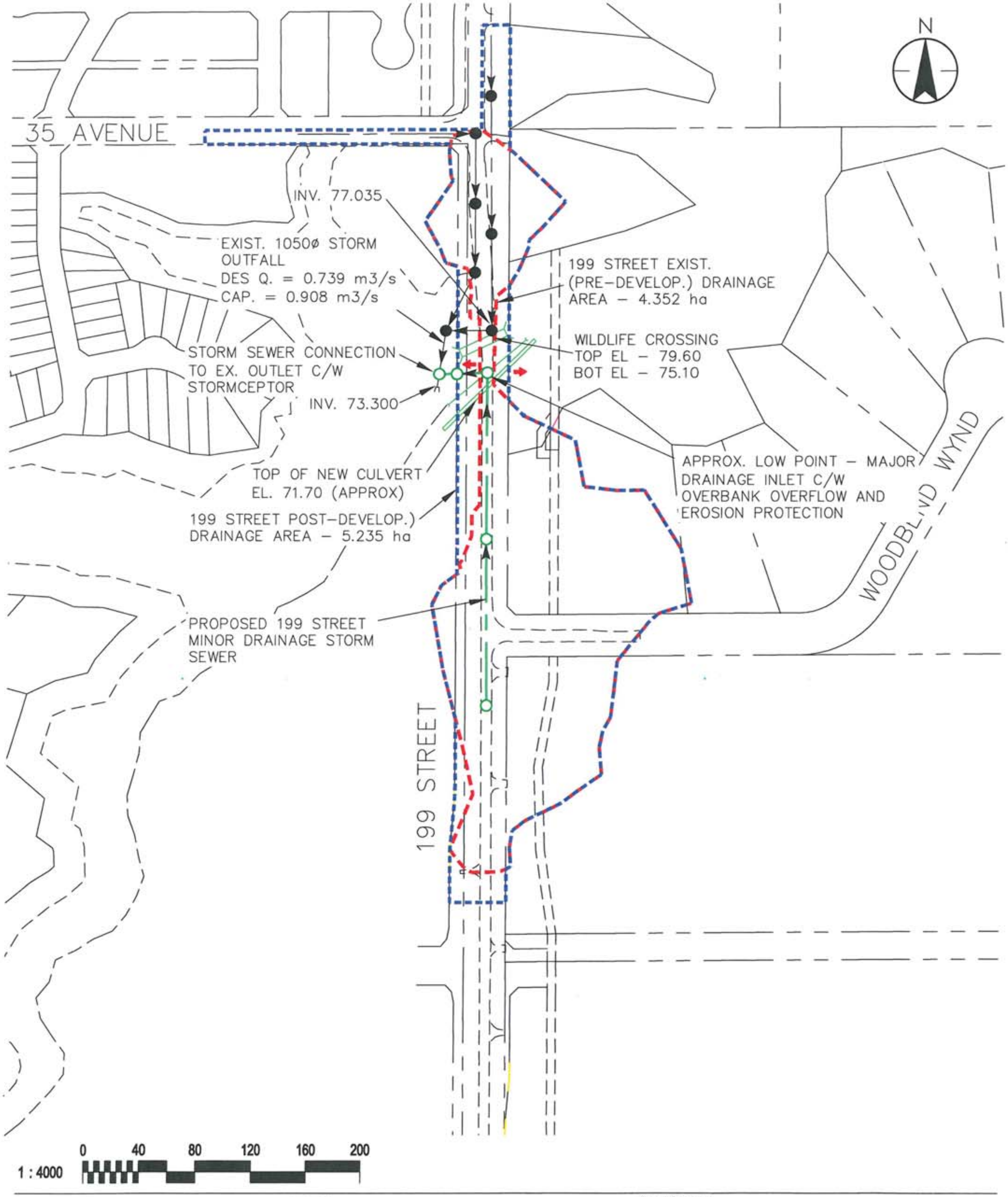
Table 2.1 depicts the drainage area components and resultant average runoff coefficient for this drainage basin. Kirpich's formula was used to determine the time of concentration for the overland drainage for this area. Utilizing the Rational Method, the peak runoff flow rates were determined for both the 5 year and 100 year storm events. These flow rates are 0.294 m^3/s and 0.655 m^3/s respectively.

2.2 Ultimate 199 Street Drainage (Post-Development)

Ultimately 199 Street will be improved to a four lane divided urban arterial roadway. The drainage area for the ultimate roadway was determined from the existing 199 Street design north in Edgemont, and the 199 Street preliminary plan for south of the Creek. The 5.235 ha drainage area is shown on Figure 1.

Table 2.2 depicts the drainage area components, time of concentration and peak runoff flow rates for the ultimate roadway drainage. The peak 5 Year flow rate is 0.488 m^3/s and the 100 year flow rate is 1.031 m^3/s. Kirpich's formula and the Rational Method were used to determine these flow rates.

It should be noted that the ultimate or post development flows exceed the existing pre-development flows. In order to maintain the pre-development flows some form of attenuation will be required.



Legend

Client/Project
RIVERVIEW OWNERS
RIVERVIEW 199 STREET
STORM DRAINAGE SYSTEM

Figure No.
1

Stantec

Title
EXISTING & PROPOSED DRAINAGE FACILITIES

Date : February 2015
Project Number : 1161 103725

Table 2.1

Existing 199 Street Drainage at Wedgewood Creek

PROJECT :	Riverview 199 Street Existing Runoff to Wedgewood Creek (Pre-development)
JOB No. :	1161 103725
DATE :	Feb. 2015

DRAINAGE AREA -	4.35	ha
OVERLAND FLOW LGTH -	350.00	m
AVERAGE SLOPE -	4.43	%
RUNOFF COEF -	0.269	5 Year
	0.314	100 Year

USING KIRPICH'S FORMULA TO DETERMINE TIME OF CONCENTRATION

TIME OF CONCEN. -	5.88	min
5 YEAR INTENSITY -	90.38	mm/hr
100 YEAR INTENSITY -	172.41	mm/hr
5 YEAR RUNOFF -	0.294	m3/s
100 YEAR RUNOFF -	0.655	m3/s

Surface	Area (ha)	Coef.	5 Year CA	100 year CA
Pavement	0.40	0.950	0.38	0.38
Field/ Ditch	3.95	0.200	0.79	0.99
Total	**4.35**		**1.17**	**1.37**
		Average CA	**0.269**	**0.314**

NOTE:

1. Drainage area, overland flow length and average slope determined from existing ground Lidar contours (Jan. 2014).
2. Anticedent runoff factor included in 100 year CA determination for pervious areas.

Table 2.2

Ultimate 199 Street Drainage at Wedgewood Creek

PROJECT :	Riverview 199 Street Post Development Runoff to Wedgewood Creek
JOB No. :	1161 103725
DATE :	Feb. 2015

DRAINAGE AREA -	5.240	ha
OVERLAND FLOW LGTH -	385.00	m
AVERAGE SLOPE -	2.23	%
RUNOFF COEF -	0.381	5 Year
	0.422	100 Year

USING KIRPICH'S FORMULA TO DETERMINE TIME OF CONCENTRATION

TIME OF CONCEN. -	6.18	min
5 YEAR INTENSITY -	88.06	mm/hr
100 YEAR INTENSITY -	167.92	mm/hr
5 YEAR RUNOFF -	0.488	m3/s
100 YEAR RUNOFF -	1.031	m3/s

Surface	Area (ha)	Coef.	5 Year CA	100 year CA
Pavement	1.19	0.950	1.131	1.131
Boulevards	1.08	0.250	0.270	0.338
Ex. Overland	2.97	0.200	0.594	0.743
Total	**5.24**		**1.995**	**2.211**
		Average CA	**0.381**	**0.422**

NOTE:

1. Drainage area, overland flow length and average slope determined from prelim plan with the existing overland boundary areas as determined in the pre devlopment runoff scenario.
2. Anticedent runoff factor included in 100 year CA determination for pervious areas.

3.0 ULTIMATE 199 STREET DRAINAGE SYSTEM OPTIONS

Several alternatives were reviewed in assessing the most practical and effective storm runoff conveyance system for the 199 Street roadway drainage. As noted previously the post development flows will exceed the pre-development flows indicating that some form of attenuation or storage will be required.

The proposed and existing grade lines for 199 Street eliminate the feasibility of constructing a gravity sewer to any of the proposed Riverview storm ponds.

The area near the intersection of Wedgewood Creek and 199 Street within the Wedgewood ravine also poses some topographical challenges when considering design options. As noted previously a new storm outfall has been constructed at the northwest quadrant of the creek crossing. This area has the flattest topography. The existing culvert under 199 Street is angled northeast (in the direction of flow) and the roadway embankment on the east side of the crossing is very steep and tree lined both north and south. The southwest area at the crossing is predominately the creek itself with a steep embankment and ponding caused by beaver damming at the existing culvert.

Some of the alternates reviewed include:

- Direct connection of the Riverview 199 Street drainage into the exiting Edgemont 199 Street storm sewer and Stormceptor.
- Review of the existing storm outfall for any excess capacity coupled with a major drainage overflow route over the roadway embankment for any rainfall event where the flows exceed the available outfall pipe capacity.
- Storage and controlled discharge from the Riverview 199 Street drainage into either the existing storm outfall pipe or a new separate outfall.

3.1 Direct Connection to Edgemont 199 Street Storm

A new water conveyance culvert and an animal wildlife crossing are to be completed as part of the Riverview 199 Street improvements. In reviewing the elevation grade lines for the wildlife crossing and the existing Edgemont sewer inverts, it was determined that a direct connection to this existing sewer main is not possible. It should also be noted that the existing Stormceptor and outfall pipe do not have sufficient capacity to adequately convey the major flows from the Riverview 199 Street drainage.

This alternative is not feasible due to the reasons noted above.

3.2 Connection to the Existing Outfall With Major Overflow Route

Currently the existing 199 Street drainage at Wedgewood Creek drtans off top of the roadway embankment and/or drainage ditches down the slopes into the creek.

The existing storm outfall pipe was reviewed to determine if any excess capacity was available through this conduit. As noted in Section 3.1 a direct connection to the existing 199 Street storm sewer is not possible. A new connection to the existing outfall is possible south of the proposed wildlife crossing. To facilitate water quality improvement a Stormceptor or equivalent would be provided prior to the outfall.

To facilitate a proper review of the proposed Riverview199 Street discharge and the intricate Edgemont drainage basin discharge into the existing outfall, computer modeling (SWMM 5.1) was utilized. The Edgemont drainage basin into the outfall includes the interconnection of three storm ponds and the existing storage pipe in 199 Street.

The existing storm outfall is a 1050 mmø pipe laid at a gradient of 0.10%. The design flow rate from the Edgemont basin is 0.739 m^3/s, with a full flow capacity of 0.908 m^3/s. The water flow velocity through the outlet pipe is 1.016 m/s. A rip rap energy dissipation apron has been built as part of the outfall structure.

The philosophy for this alternative is to discharge the first flush and the minor storm through a Stormceptor into the existing outfall pipe. Any major flows exceeding the capacity of the connection/outflow will then spill over the top of the roadway embankment via a rip rap or other suitably reinforced spillway into the adjoining creek.

Table 3.1 provides a summary of the modeling results for both the existing storm outfall pipe and the proposed overflow route to the creek. The peak flows for the various storm events for the Riverview 199 Street drainage generally occur much sooner than the peak controlled discharge from the Edgemont storm ponds and storage pipe. Based upon iterating an appropriate curb inlet/pipe size to connect to the existing storm outfall only two of the major storm events will require the overflow spillway to convey the major flows. These storm events are further described below.

3.2.1 100 Year 4 Hour Storm

With an equivalent curb inlet / pipe size combination to a 350 mmø pipe connection to the existing storm outfall the available capacity in the existing storm outfall pipe will not be exceeded. The net spillway flow (note that spillways will be located on both the east and west sides of the roadway) flow rate will be 0.21 m^3/s. This flow rate is less than the 5 year storm runoff rate for the existing (pre-development) 199 Street drainage (0.294 m^3/s – Section 2.1). The duration of flow via the bank spillway is approximately 30 minutes.

Table 3.1

199 Street Drainage at Wedgewood Creek Connection to Existing Storm Outfall Summary of SWWM Results

Rainfall Event	Peak Flow Time (Hour:Min.)	199 Street Peak Runoff Flow (m^3/s)	Connect Pipe to Ex. Outfall Peak Flow (m3/s)	Exist Outfall Pipe Peak		Overflow Channel Over Top of Bank	
				Flow (m3/s)	Velocity (m/s)	Flow (m3/s)	Velocity (m/s)
5 Year 4 Hour	1:30	0.39	0.39	0.50	1.01	0.00	0.00
25 Year 4 Hour	1:30	0.64	0.64	0.74	1.01	0.00	0.00
100 Year 4 Hour	**1:30**	**0.86**	**0.65**	**0.85**	**1.01**	**0.21**	**2.27**
100 Year 24 Hour	3:30	0.25	0.25	0.59	1.01	0.00	0.00
1937 Rainfall	9:00	0.23	0.23	0.63	1.01	0.00	0.00
1978 Rainfall	**15:40**	**0.76**	**0.64**	**1.17**	**1.35**	**0.12**	**1.82**
1988 Rainfall	12:05	0.22	0.22	0.49	1.01	0.00	0.00

The exit flow rate from the ultimate surface drainage will be controlled by the catch basin inlet sizes and/or the catch basin lead size. The maximum ponding depth of 150 mm on the arterial roadway will not be exceeded.

3.2.2 1978 Storm

Several curb inlet / pipe size combinations were iterated in reviewing this particular storm event. As the peak flow rate timing from the Riverview 199 Street drainage and the flow from the Edgemont basin occur fairly close together during this event, some limitations were noted. In attempting not to exceed the outfall pipe capacity the flow rate over the bank spillway increased dramatically.

Through iteration, it was determined that in order to minimize the overbank flow rates to approximately the 5 year pre-development rate, that an equivalent curb inlet / pipe size combination to a 350 mm^{ø} pipe connection to the existing storm outfall would achieve the best solution.

Utilizing the above iterative criteria, the spillway flow rate over the top of bank would be 0.12 m^3/s, however the flow rate through the existing pipe outfall would increase to 1.17 m^3/s. To sustain the slight increase in flow rate the outfall pipe will be surcharged. The resultant grade line to convey this flow rate is 0.17% with a pipe flow velocity of 1.31 m/s. The effective upstream surcharge depth required to convey this flow rate is 0.029m. The duration of time that this pipe will be surcharged is approximately one hour. The existing outflow pipe is the last pipe in the system and is significantly lower than any other upstream pipe. As such the surcharge height will not cause any detrimental effects. The increased outflow velocity can adequately be handled by the existing exit rip rap apron.

The exit flow rate from the ultimate surface drainage will best be controlled by the catch basin inlet sizes and/or the catch basin lead size. The maximum ponding depth of 150 mm on the arterial roadway will not be exceeded.

This alternate outlined in Section 3.2 is the recommended conveyance system for the Riverview 199 Street drainage. Please refer to Section 4 for additional recommendation comments. Figure 1 depicts this recommended alternative.

3.3 Storage and Controlled Discharge

Implementation of storage and releasing at a controlled outflow was reviewed. Due to the limited space and geometry available, and the proximity to the creek the only method of storage is an underground storage pipe. A controlled outlet flow rate of 35 l/s/ha was selected in conformance with the design parameters used for the existing Edgemont 199 Street storage.

Based upon the computer modeling implementing storage for the Riverview 199 Street drainage as well the upstream Edgemont drainage basins contributing into the existing storm outfall, the maximum storage volume required for the Riverview 199 Street retention would be 2150 cubic metres. Preliminary review indicates that approximately 375 lineal metres of 2.4 m by 2.4 m box section would be required to provide the necessary storage.

Providing the storage pipe and associated manholes significantly increase the infrastructure required. Ultimately this system will require maintenance and operation checks throughout its service life.

It should also be noted that the storage pipe on average is 5% full through various storm events. In considering the quantity, installation cost, and potential maintenance and operation requirements for a system that operates only for a limited period of time, we do not consider this a viable alternative.

4.0 SUMMARY AND RECOMMENDATIONS

The angled creek crossing of the 199 Street roadway and the existing topography at Wedgewood Creek present both design and construction challenges. In order to provide a sustainable design, and a practical and cost effective drainage solution, several alternatives noted above were reviewed.

The recommended alternative is to provide a connection for the Riverview 199 Street roadway drainage directly to the existing Edgemont storm outfall. During the more extreme major rainfall events an overland channel / spillway from the road surface over the embankment will be provided to convey the flows not conveyed through the outfall. These over top flows will be for a short duration only. The existing storm outfall and outflow apron have sufficient capacity to convey the flows. A new Stormceptor or equivalent will also be installed to improve the water quality of the 199 Street drainage flows.

It should be noted that this brief is a preliminary design and further refinements will be completed during the detailed design stage for the Riverview arterial roadway. Additional attention to the catch basin inlets, catch basin leads and outfall connection pipe size will be conducted during the detailed design. Further erosion and sediment control plans will also be developed during the detailed design process.

APPENDIX A

Appendix A

A review of the existing Edgemont outfall structure c/w rock rip rap apron was completed with a specific interest in the outflow velocities as the upstream storm discharge passes through the structure and over the rock rip rap apron.

The primary purpose of an outfall structure which includes rock rip rap is to provide energy dissipation over the length of the entire structure. The energy dissipation is achieved by dispersing and spreading the flow from an outlet pipe to a lower depth of flow over a broader width. The rock rip rap apron (approximately 8 metres in length) provides an area which disrupts the flow over a rough surface. As the flow stream exits the outlet pipe, gravity reacts and the flow stream drops and spreads out to conform to the downstream channel geometry.

As proposed, with the connection of the Riverview 199 Street arterial road drainage the flow rate to the outlet structure would increase to 1.17 m^3/s, with an outlet velocity at the headwall of the outlet structure of 1.35 m/s.

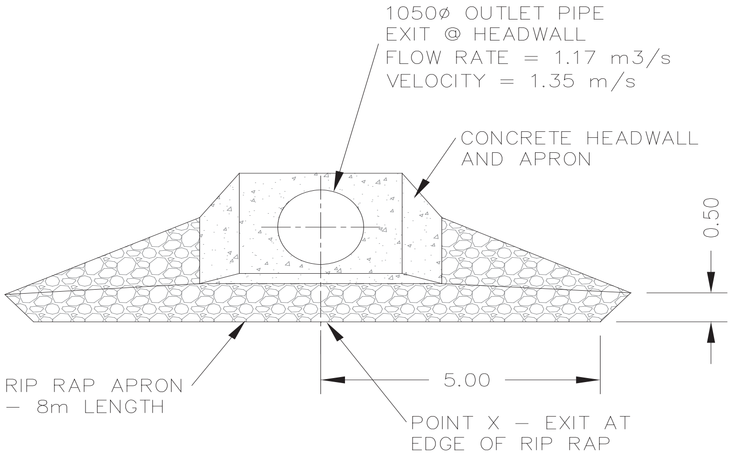
Table A1 identifies the outlet flows and velocities at the exit end of the rock rip. The resultant depth of flow at this exit location is 0.127 m with a velocity of 0.90 m/s. Manning's Equation was utilized in determining the flows and velocities over the rip rap apron. Various publications provide the roughness coefficient for rock rip ranging between 0.04 and 0.10. The n value used for this analysis is 0.07.

As identified in the City of Edmonton Design and Construction Standards, the final exit velocity where the flow passes from an apron or erosion control medium to the natural channel, shall not exceed 1.0 m/s.

As indicated in Table A1 the exit flow velocities exceed the required minimum standard.

Rock rip rap is typically used in energy dissipation situations as maintenance is easily achieved and easily modified should the need arise.

TABLE A1
OUTLET STRUCTURE RIP RAP VELOCITY EVALUATION



FRONT VIEW OF EDGEMONT OUTFALL STRUCTURE

Slope 0.063 Rock Rip Rap Apron Slope (8m length)
Mann n 0.070

Depth m	Width m	Area m2	Wet. Perm. m	R m	Flow - Q m3/s	Velocity - V m
0.10	10.0	1.020	10.224	0.100	0.783	0.768
0.11	10.0	1.124	10.246	0.110	0.920	0.818
0.12	10.0	1.229	10.268	0.120	1.065	0.867
0.127	10.0	1.302	10.284	0.127	1.172	0.900
0.130	10.0	1.334	10.291	0.130	1.219	0.914
0.14	10.0	1.439	10.313	0.140	1.382	0.960
0.15	10.0	1.545	10.335	0.149	1.553	1.005

Note:
Flow parameters evaluated using Manning's Equation

APPENDIX B

Appendix B

The proposed Riverview 199 Street storm drainage system is comprised of two components. The first component is a tradition storm sewer system which collects the storm runoff via catch basins and their connection into a conveyance storm sewer. This storm system collects at the lowest point in the 199 Street roadway near the proposed wildlife crossing. The storm sewer at this point is routed through a Stormceptor or equivalent into the existing Edgemont outlet structure. The storm sewer description and operation characteristics have been previously presented in the design brief prepared by Stantec in March 2015. Further information pertaining to the exit velocities from the outlet structure has been provided in Appendix A. The storm sewer will connect into the existing underground system below the ground surface and will not interact with the wildlife crossing or animal paths.

The second component also described in the Stantec March 2015 brief is the overflow channels over each side of the roadway embankment. These overflow channels only be active / used during two major storm events. As shown on Figure XXX, the overflow channels will be located such that the channels will not flow into the wildlife crossing. Where the overflow route crosses the animal paths a suitable type surface for the channel will be implemented to allow easy passage for the animals along the path.

Several alternatives (but not limited to) for the channel surface are available to convey the required overflow runoff are listed below:

- ½ Culvert channel or flume
- Concrete spillway channel
- Geotextile / geogrid / geo cell with earth and resultant grass growth overtop
- Turfstone and resultant grass growth through openings
- Rock rip rap channel

Due to physical and space constraints the proposed fill slope of the roadway embankment is 2:1 (2 horizontally to 1 vertically).

A more detailed description of the channel surfaces is provide below.

B.1 ½ CULVERT CHANNEL

Several potential issues pose constraints for using the ½ culvert option. Typically this type of channel uses a pre manufactured one half of a CSP culvert. The culvert half is anchored into the side slope. Over the long term and as result of many freeze /thaw cycles, the anchors and or the adjacent embedment materials may loosen causing continuity / stability problems. This option is not very aesthetic either. This option is not recommended.

B.2 CONCRETE SPILLWAY CHANNEL

A concrete channel could be used to convey the overflow runoff. It should be noted that with the steep roadway embankment this channel would be very difficult to construct. As well this is typically a smooth surface which does not provide for any further energy dissipation. Due to the construction difficulty and costs this option is not recommended.

B.3 GEOTEXTILE AND GRASS

This methodology is generally used for slope protection purposes and not necessarily a drainage conveyance channel. The construction includes the placement of a geotextile membrane followed by a suitable substrate and a grass surface. Although effective for slopes where rain falls only on the slope area the additional flow from the overflow and resultant flow velocity for this application's very steep slope may have an impact on the vegetation. Further review at the detailed design stage would be required to assess the permissible flow velocities for this slope structure. At this time it is unclear that this alternate would provide long term performance without any supplemental maintenance. Please refer to the additional notes at the end of this Section.

B.4 TURFSTONE AND GRASS

Turfstone is a pre-manufactured concrete block with a honeycomb pattern which provides opening for which grass or other vegetation can grow through. The Turfstone and grass surface is currently used in Edmonton applications for storm pond slopes where boat launch ramps are required. This is still considered to be a relatively smooth surface in relation to providing effective energy dissipation. This would be considered a suitable alternative from a slope protection aspect. Please refer to the additional notes at the end of this Section.

B.5 ROCK RIP RAP CHANNEL

From an energy dissipation aspect, rock rip is considered to be the best for this application. The size of stones and the rough surface would be the most effective at disrupting the flow in the channel. Erosion protection is easily achieved as the stones are hard and erosion resistant. The stones are installed over a geotextile membrane which retards the movement of moisture through the stone into the underlying embedment material. Rock rip rap is easily installed and maintained. Rock rip rap is considered to be the best alternate from both any energy dissipation and erosion protection perspective and is the recommended alternative.

B.6 OVERFLOW CHANNEL INTERFACE AT ANIMAL PATH

As noted above the rock rip rap is the recommended alternate for the overflow channel surface. If there is a concern for where the animals may need to cross the rock rip rap, the Tufstone and grass within the animal path area will provide a smoother surface for the animals to navigate.

B.7 ADDITIONAL NOTES

Slope protection and erosion and sediment control are two very important aspects in selecting an appropriate drainage control structure / surface. Any erosion and resultant sediment release would be considered a failure. In listing the above alternatives and recommendations, we have assessed that the rock rip rap protection would be the best alternative from a cost, durability and maintenance perspective.

The interface at the animal crossing / path has also been reviewed and the Turfstone and grass within the animal path would be considered acceptable. It should be noted that the rock rip rap may not pose an impediment for the various animals and is still considered to be the best alternate to provide energy dissipation and erosion protection.

Various pictures of the above noted alternatives are provided for reference.


WILDLIFE
3.0 PATH
EX TREES
WING WALL
EX. MANHOLE
WING WALL
LOT 66ER
BLK. 17
WILDLIFE PASSAGE
1:5
1:5
32.29
E-E
P4
E-E
P4
F-F
P4
WING WALL
EXISTING OUTFALL
WING WALL
MAJOR DRAINAGE ROUTE
MAJOR DRAINAGE ROUTE
WING WALL
3m WILDLIFE PATH
EXISTING TREES
3000 Ø CSP
F-F
P4
NE 1/4 6-52-25-W4
3m WILDLIFE PATH
3.0 PATH
WILDLIFE FENCE
NW 1/4 5-52-25-W4
WILDLIFE FENCE

Erosion and Sediment Control Report 199 Street Wedgewood Creek Crossing – North of Woodbend Wynd, South of 35 Avenue

Prepared for:
Riverview Heights Estates Ltd.

Prepared by:
Stantec Consulting Ltd.

December 2, 2015

Sign-off Sheet

This document entitled Erosion and Sediment Control Report 199 Street Wedgewood Creek Crossing – North of Woodbend Wynd, South of 35 Avenue was prepared by Community Development, Stantec Consulting Ltd. for the account of Riverview Heights Estates Ltd. Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Prepared by Tony Chiarello
(signature)

Eric Hagger, E.I.T. and Tony Chiarello, E.I.T.

Reviewed by Tony Chiarello
(signature)

Tony Chiarello, E.I.T.

Reviewed by ______________________
(signature)

Leanne Ure, P.Eng., CPESC

PERMIT TO PRACTICE
STANTEC CONSULTING LTD.
Signature ______________________
Date Dec 3/15
PERMIT NUMBER: P 0258
The Association of Professional Engineers and Geoscientists of Alberta

DAVID K. KINDERS
Dec 3/15

Table of Contents

LIST OF APPENDICES

1.0 EXISTING LAND

1.1 SITE LOCATION AND CHARACTERISTICS

Wedgewood Creek enters the City of Edmonton in the southwest corner near 23 Avenue and 215 Street, flowing northeast approximately 5 kilometers to the North Saskatchewan River. The existing Wedgewood Creek crossing along 199 Street consists of a rural two lane road constructed on an embankment, intersecting the Wedgewood Creek ravine. Drainage is maintained through the embankment with an existing 1900mm culvert underneath 199 Street.

The lands to be developed by Riverview Heights Estates Ltd. are within N.W. ¼ Sec. 5 TWP. 52 RGE. 25, W.4th Mer. and N.E. ¼ Sec. 6 TWP. 52 RGE. 25, W.4th Mer. covering approximately 1.5 ha.

The proposed construction for this area will consist of the first two lanes of 199 Street, constructed through the Wedgwood Creek ravine, to allow for access to the Riverview neighborhood. Roadway upgrades will include installation of a wildlife crossing (bridge), drainage measures, and culvert improvements. The wildlife crossing will be built to the ultimate four lane standard however only the first two lanes will be put into service at this time.

1.2 SOIL CHARACTERISTICS AND ERODIBILITY

The geotechnical report referenced in this report is entitled "Geotechnical Investigation – Proposed 199th Street Upgrades – Stage 1 – Underground Utilities, Deep Fill Culvert and Wildlife Crossing – 35 Avenue to Woodbend Wynd NW – Edmonton, Alberta," prepared by Hoggan Engineering and Testing (1980) Ltd. May 2015.

The general stratigraphy of the soil consists of existing asphalt and gravel, topsoil underlain by clay fill, sand, and high plastic clay underlain by clay till.

Using boreholes 2 and 3 of the attached geotechnical report, the clay strata is located approximately 10 meters below the existing surface. Over top of the in-situ clay layer, to a depth of 1 meter below the surface, is clay fill. Both the clay fill and in-situ clay are silty sandy with medium plasticity, stiff to very stiff consistency, moist, and brown to grey/brown in colour with sand and organic lenses occasionally encountered.

1.3 ADJACENT LANDS AND DOWNSTRWEAM RECEIVING AREAS

The Wedgewood Creek crossing transects the water body approximately 2 kilometers downstream along its journey to the North Saskatchewan River. At this location the ravine is well incised. The road right-of-way drains overland into Wedgewood Creek between immediately north of 35 Avenue and 450m south Wedgewood Creek.

1.4 SITE EROSION POTENTIAL AND RISK ASSESMENT

1.4.1 Site Erosion Potential

The overall site erosion potential is determined by the slope lengths, gradients and soil erodibility and is based on the City of Edmonton Erosion and Sedimentation Control (ESC) Guidelines (January 2005).

- At its most severe, the site has an overall slope of 9%, which is less than 10% and therefore classified as a "Gentle Slope."
- The existing slope length of approximately 200m is considered "long."
- The soil is silty sandy clay and therefore the soil erodibility rating is classified as "medium."

1.4.2 Risk Assessment

Using information and calculations gathered from the Site Erosion Potential Section 1.4.1 (above), a risk assessment of the site can be determined. The erosion potential for 199 Street at the Wedgewood Creek crossing is considered to be "Moderate." Therefore, ESC measures are required.

2.0 PROPOSED DEVELOPMENT

2.1 DEVELOPMENT PHASING

2.1.1 Construction Phase1: 199 Street – North of 23 Avenue to South of Woodbend Wynd

Phase 1 of construction will consist of 1,100 meters of 199 Street arterial roadway. This phase is set for completion by September 2016, and will be discussed in detail in the separate ESC report titled "199 Street North of 23 Avenue to Woodbend Wynd."

2.1.2 Construction Phase 2: 199 Street – North of Woodbend Wynd to South of 35 Avenue

Phase 2 of construction through the Wedgewood Creek crossing will consist of the following: removal of the existing 199 Street roadway embankment and existing culvert, culvert upgrade, embankment and wildlife crossing installation, underground deep and shallow utility installation, and final road construction.

2.2 CONSTRUCTION SCHEDULE

2.2.1 STEP 1: Creek Flow Management, Removal of Existing Infrastructure, and Proposed Culvert Installation.

2.2.1.1 Tentative Construction Procedure

Existing 199 Street ditches draining from 35 Avenue and Woodbend Wynd towards Wedgewood Creek will remain vegetated. In the event these ditches are disturbed by construction, SC150BN erosion and sediment control blanket will be installed along with "GeoRidge" triangle silt dikes. Erosion control through this area will be maintained throughout the life of the project.

Before removal of the existing 199 Street roadway through Wedgewood Creek begins, the creek will be temporarily dammed at the upstream construction limit using Aqua-Barrier or equivalent, a water-Inflated dam. The construction zone will be left to dry and a second Aqua-Barrier dam will be placed at the downstream construction limit isolating the construction zone to prevent sediment release. Once the construction zone is isolated removals can begin. In the event that the construction zone cannot dry naturally, excess water will be pumped mechanically through a silt bag to an adjacent vegetated area east of the crossing.

As the existing roadway embankment is removed, a water diversion channel will be established south of the proposed culvert to connect Wedgewood Creek without infiltration or leakage into the construction zone. This temporary channel will be lined with SmartDitch or an equivalent product capable of allowing Wedgewood Creek to flow along the channel without infiltration or leakage.

Following embankment removal the existing culvert will be removed at which time the purposed culvert will be installed. The creek bed and any disturbed areas will be restored per the landscape details. Once restoration is complete the temporary dams will be removed and the creek will be allowed to flow through the new culvert.

2.2.1.2 Proposed Timing

- **August 2016**
    - Upstream temporary dam to be installed and creek bed allowed to dry. In the event that the construction zone cannot dry naturally, excess water will be pumped mechanically through a silt bag to an adjacent vegetated area east of the crossing.
- **September 2016**
    - Installation of the downstream dam, as well as, the excavation of temporary diversion channel.
    - Removal of the existing 199 Street rural road structure, earth embankment, and existing culvert.
    - Installation of permanent 3000mm culvert.
    - Backfill above and around culvert with restoration of disturbed areas within the creek as per landscape details.
    - Backfill will occur to a level allowing for underground installation along 199 Street.
    - Once the creek bed has been rehabilitated, temporary dams will be removed and diversion channel backfilled, allowing Wedgewood Creek to flow through the proposed 3000mm culvert. Note that the temporary dams will remain in place until compost berms are installed (see section 2.2.2).

2.2.1.3 Summary of Interim ESC Measures

- For erosion and sediment control details please refer to Drawing C015-003, Step 1.
- Concentrated flow is expected along the existing slopes draining into the Wedgewood Creek Ravine to the north and south of the purposed crossing. Ditches will be left vegetated for erosion control. Disturbed areas will be bolstered with a combination of SC150BN erosion control blankets and "GeoRidge" triangle silt dikes in areas of erosion potential.
- SmartDitch or similar material will line the bottom of the temporary channel creating a smooth causeway to reduce erosion potential.

- Silt fence will be installed along the SmartDitch to restrict sediment entry from upstream sources that may enter the temporary diversion channel.
- Restoration measures will be followed as specified on the landscaping plans.
- In the event of rainfall during construction, standing water within the isolated construction zone will be pumped mechanically through a silt bag to an adjacent vegetated area east of the crossing.

2.2.2 STEP 2: Foundation, Embankment, Wildlife Crossing, Underground and Road construction

2.2.2.1 Tentative Construction Procedure

To re-isolate the construction zone from the restored and flowing Wedgewood Creek, compost berms will be installed along the side slopes before the creek. The compost berms will be placed perpendicular to the surface runoff and will act as a dike to retain overland drainage. Approximately 150 meters of compost berm will be installed at a height of 0.6 meters to protect the creek from sediment release. Depressions graded along the berms will assist in increasing the impoundment and sediment retention capabilities of the berms.

With Wedgewood Creek isolated, construction of the embankment, wildlife crossing, underground deep and shallow utilities, and road construction can occur without disturbance to the flowing creek.

2.2.2.2 Purposed Timing

- **October 2016**
    - Installation of the compost berms prior to removal of Aqua-Barrier.
    - Backfill temporary diversion channel through 199 Street.
    - Piles will be installed in the newly backfilled embankment to support the abutments of the proposed wildlife crossing.
    - Erosion control blankets placed as required on side slopes directed towards Wedgewood Creek.
- **Winter of 2016 and 2017**
    - Pile installation.
- **Spring and Summer 2017**
    - Underground installation.
    - Wildlife crossing construction.

  - Road construction.
  - Final grading of approaches and side slopes.

2.2.2.3 Summary of Interim ESC Measures

- For erosion and sediment control details please refer to Drawing C015-003, Step 2.
- Compost berms will be installed above the inlet and outlet of the culvert servicing Wedgewood Creek. Any surface runoff from within the construction zone will be directed to the berms by the natural topography.
- Impoundment areas will be excavated adjacent to the compost berms which will act as an impoundment and sedimentation retention area. Standing water will be pumped mechanically through a silt bag to an existing vegetated area east of the crossing to return to Wedgewood Creek.
- Disturbed areas will be graded rough and where possible SC150BN erosion control blankets will be placed to protect Wedgewood Creek from upstream slope erosion and sediment release.

2.2.3 STEP 3: Slope Protection following Road Construction

2.2.3.1 Tentative Construction Procedure

Following construction, side slopes and disturbed areas will be seeded and rehabilitated following the direction of the landscape details. These steps will be implemented in the fall of 2017 to prepare the site for landscaping in spring 2018.

2.2.3.2 Purposed Timing

- **September 2017**
  - Rehabilitation and restoration of disturbed slopes will occur with either a combination of seeding and erosion blanket or hydro-seeding. Installation method will be based on weather conditions. This will be followed by the removal of compost berms.

2.2.3.3 Summary of Interim ESC Measures

- For erosion and sediment control details please refer to Drawing C015-003, Step 3.
- Extensive seeding as detailed in the landscape plans will take place along the side slopes of the embankment following final grading. SC150BN erosion control blanket will be laid over the seeded areas to protect wash off and allow seeds to take hold. If weather permits, hydro-seeding will occur in lieu of placement of an erosion control blanket.

- Compost berms will be in place during construction and final grading to protect Wedgewood Creek. These will be removed once seeding and erosion control blankets have been installed.

3.0 EROSION AND SEDIMENT CONTROLS

3.1 EROSION CONTROL

3.1.1 Site Management

Grading contractors will leave the affected site rough instead of blading it smooth. By leaving the site rough resistance to water flow will be created and will limit water velocity, thereby helping to minimize soil erosion.

3.1.2 Grass seeding

Grass seeding will involve the application of a mixture of various grass seeds on the areas of bare ground that require vegetative covering. Grass seeding should commence as soon as it is feasible to do so following landscape details.

3.1.3 Erosion Control Blanket

Erosion control blankets are used to keep soil and seed in areas where erosion may occur. To minimize erosion and seed loss, SC150BN single net straw, or equivalent, will be installed to manufacturer's specifications. North American Green SC150BN is a short term double net straw fibre erosion control blanket that is biodegradable and is designed for use in medium-flow channels. It is designed to last up to 18 months, thus allowing for permanent vegetation establishment. These blankets will be placed in the areas where overland flow occurs (as per Drawing C015-003).

3.2 SEDIMENT CONTROL

3.2.1 Filtering

During site development and construction, containment of suspended soil particles will be managed on-site. Any potential spill location to Wedgewood Creek will be protected with silt fence and compost berms. If pumping water is absolutely required, water is to be pumped through a filter bag prior to the release and allowed to travel along vegetated paths east of the crossing before entering Wedgewood Creek.

3.2.2 Creek Isolation

The creek will be isolated from the construction using the temporary damming system Aqua-barrier, installed at upstream and downstream construction limits. To manage the rising water level upstream, a diversion channel will be excavated to maintain the downstream flow of Wedgewood Creek. As construction continues and the creek is allowed to flow through the proposed culvert, slopes adjacent to the construction site will be restricted from contributing to Wedgewood Creek with the use of compost berms.

3.2.3 Impoundment

Temporary impoundment areas will be excavated upstream of compost berms to trap the surface runoff inside the construction area so it can be pumped through a silt bag and along a vegetated area east of the crossing to return to Wedgewood Creek.

3.2.4 Temporary Water Diversion

During damming, Wedgewood Creek will be allowed to flow along a temporary diversion ditch. The ditch will be lined using SmartDitch or equivalent, a system that allows water to flow without leakage or infiltration.

3.2.5 Good Housekeeping Measures

Site maintenance will prevent excessive sediment from entering the Wedgewood Creek waterbody. The ESC system will be monitored for sediment removal after storm events and during weekly site inspections. A sample ESC inspection and maintenance report is attached. During construction, these weekly reports will be filed by the contractor with Drainage Services and with Stantec.

The developers and site contractors have a legal and contractual obligation to control the soils from sedimentation, road tracking, and wind erosion. Therefore any issues identified in the weekly inspection reports will be dealt with in a timely manner.

Appropriate signage restricting site access coupled with gravel pads at site entrances will minimize tracking of soil onto roadways. If required, gravel pads will be placed at entrance and exit locations of the construction site.

3.2.6 RUSLE 2.0

RUSLE2.0 is a tool used to calculate the average annual rate of soil loss based on subsoil conditions. RUSLE2.0 is a tool to support the use of best management practices. This tool, along with judgment and site conditions, should be understood when reviewing ESC. Calculations that factor into the BMPs have been included.

4.0 SUMMARY

With the information presented and the preventative measures outlined herein, the potential erosion from wind and water will be effectively controlled. The site will be monitored on a weekly basis during development and if required, further erosion and sediment control measures will be implemented to address potential problems. Riverview Heights Estates Ltd. and Stantec Consulting Ltd. are committed to uphold environmental legislation and municipal bylaws related to ESC and will work with the City of Edmonton to address any concerns.

This ESC plan and report will be confirmed with the Contractor and further verified with the Contractor's eco-plan and best management practices.

APPENDIX

Appendix A

GEOTECHNICAL INVESTIGATION – PROPOSED RIVERVIEW NEIGHBOURHOOD 1 – STAGE 1 AND FUTURE APPROXIMATELY 199 STREET & 23 AVENUE N.W. – EDMONTON, ALBERTA

HOGGAN ENGINEERING & TESTING (1980) LTD.

REPORT NO: 6004-38

GEOTECHNICAL INVESTIGATION
PROPOSED 199TH STREET UPGRADES – STAGE 1
UNDERGROUND UTILITIES, DEEP FILL CULVERT AND WILDLIFE CROSSING
35TH AVENUE TO WOODBEND WYND NW
EDMONTON, ALBERTA

May 2015

HOGGAN ENGINEERING & TESTING (1980) LTD.
17505 – 106th Avenue
Edmonton, Alberta
T5S 1E7

PHONE: **780-489-0880**
FAX: **780-489-0800**

REPORT NO: 6004-38

GEOTECHNICAL INVESTIGATION
PROPOSED 199TH STREET UPGRADES – STAGE 1
UNDERGROUND UTILITIES, DEEP FILL CULVERT AND WILDLIFE CROSSING
35TH AVENUE TO WOODBEND WYND NW
EDMONTON, ALBERTA

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION

PROJECT: Proposed 199th Street Upgrades – Stage 1
Underground Utilities, Deep Fill Culvert and Wildlife Crossing

LOCATION: 35th Avenue to Woodbend Wynd NW
Edmonton, Alberta

CLIENT: Qualico Communities
c/o Stantec Consulting Ltd.
10160 – 112th Street
Edmonton, Alberta
T5K 2L6

ATTENTION: Tony Chiarello, E.I.T.

1.0 INTRODUCTION

This report presents the results of the subsurface investigations made on the site of the proposed road upgrading in Edmonton, Alberta. The objective of the investigation is to determine the existing subsoil conditions along the proposed road alignment and to provide geotechnical recommendations for the roadway development, underground utility installation and wildlife crossing construction based on the soil data retrieved. Authorization to proceed with the investigation was received from Petrea Chamney of Stantec in February 2015. Field work for the project was completed in April 2015. Environmental and previous land use issues are beyond the scope of this report.

2.0 SITE DESCRIPTION

It is understood that the project consists of upgrading the existing rural 199th Street roadway to a four-lane urban arterial roadway, from 35th Avenue to 23rd Avenue. This project concentrates on Stage 1 of the upgrades between 35th Avenue to Woodbend Wynd NW as part of the overall Riverview Neighbourhood development. The new lanes will be constructed west of the existing 199th Street with minor widening to the east to accommodate walks and light standards. Water and storm services will be installed below the roadway as part of this project. The proposed depth of the utilities 6 to13 meters below existing ground surface. The deeper utilities are anticipated to be below the Wedgewood Creek (WWC).

In addition the proposed road upgrade will include replacement of the existing culvert from a 1900 millimeter diameter to a 2400 millimeter and constructing a separate wildlife passage at the WWC. It is understood that the wildlife passage will be a single-span bridge.

The existing 199th Street is a rural profile road which runs north south within the project limits. Stage 1 project limits are typically within the WWC crossing section. Power lines were noted on the east side of the road. Generally the road had a rolling terrain with a low area at the WWC location.

At the time of inspection, 199th Street was surfaced with hot mix asphalt. The road appeared in fair condition with no major rutting, cracking or failure noted.

Site reconnaissance was completed on the side slopes of the existing 199th Street at WWC on April 2, 2015. During the site inspection it was noted that the west slope was approximately 2.5H: 1V while the east slope was approximately 2H:1V. Both side slopes were covered with grass, light bush and small trees. The east slope featured areas where soil disturbance had occurred, likely due to the installation of underground utilities the previous year. Toe erosion was not noted on either side of the slope. A culvert, approximately 1900 millimeters in diameter was noted in the creek to allow for water flow under the road. The culvert appeared to be straight with no curvature. A protective metal cage was observed on the upstream portion of the culvert, on the west side of the road. The cage and culvert inlet was surrounded by a beaver dam. Further west of the culvert, a concrete storm outfall exists. The outfall was constructed in 2014. Beaver dam activity is quite evident upstream and downstream of the 199th Street. Beaver dams up to 1.5 to 2 meters high are noted. Evidence of side slope instability was not noted during our site visit. Site photos are provided in Appendix II.

Geotechnical Report Review

A search for geotechnical information was requested from the City of Edmonton Engineering Services Library. The following reports were reviewed:

- Slide Investigation, 199 Street and Wedgewood Creek, Edmonton, Alberta, Prepared by: Thurber Engineering, File No. 14-31-70, May 30, 1990.

- Slope Stability Assessment, Proposed Edgemont Neighborhood, North and West Bank of Wedgewood Creek, 215 Street and 35 Avenue, Edmonton, Alberta, prepared by Hoggan Engineering & Testing (1980) Ltd., File No. 6004-22, August 4, 2011.

Report 1 was completed on a failure of the west embankment fill of the existing 199th Street at WWC. The failure was noted to be shallow and not deep seated. It was determined that the failure occurred due to the buildup of water at the inlet of the culvert. The buildup of water occurred due to the beaver dam limiting the flow of water downstream. The observations indicated that the toe of the side slope became saturated leading to its failure. The report provided observations and recommendations for the repair of the side slope failure. No evidence of this past failure was noted during our site reconnaissance or the air photo review.

Report 2 completed by Hoggan was a slope stability analysis of the north and west banks of the Wedgewood Creek as part of the Edgemont Neighborhood Development. The slope assessment did not include the assessment of 199th Street side slopes at the WWC.

Aerial Photograph Review

Several sets of aerial photography taken between 1924 and 2014, covering the subject site and surrounding areas, were obtained from the City of Edmonton Mapping Department, the Alberta Sustainable Resource Development Library and Google Earth. The photos were reviewed to identify any signs of disturbances within the site.

The photo coverage obtained is summarized as follows:

Year	**Catalogue No.**	**Photo No.**	**Scale**
2004 – 2014	Google Earth	---------	Approximately 1:5000
2001	ED 2001-01	138 and 139	Approximately 1:20000
1993	AS 4383	208 and 209	Approximately 1:20000
1974	AS 1313	220	Approximately 1:12000
1962	AS 818	15	Approximately 1:31680
1949	AS 136	58 and 59	Approximately 1:40000
1924	C.ARS	35	Oblique

In1924, 199th Street did not cross the WWC at its existing location. It crossed the WWC to the west of its current crossing location. The road ends at 35th Avenue and then heads south roughly 160 meters west of the existing location and winds through the WWC. The road then follows its current alignment approximately 300 meters south of WWC. Little to no development with the exception of two farm houses was noted along the 199th Street to the south and north of the WWC. In 1949, road followed the same pattern and no observable changes to the road were noted. In the 1962 Air Photo, 199th Street appears to follow its current alignment and crosses WWC at its current crossing location. In the 1974 Air Photo, the road appears to be wider and appears to have been paved. Woodbend Wynd along with the subdivision development appears to the southeast of the WWC and 199th Street intersection. Several farm residences are noted to the north of WWC on the east and west sides of 199th Street. In the Air photos from 1993 Photos to the summer of 2014, no changes to the current road alignment from that of the 1974 Photo was noted. In the summer of 2014, 199th Street appears to have been removed due to the construction of underground utilities from 35th Avenue to the north edge of the WWC. Development of the Edgemont Subdivision is noted in the 2012 photos on Google Earth.

It should be noted that the failure noted in the 1990 Thurber Report could not be seen in any of the observed Air Photos. No slope stability concerns with the side slopes of 199th Street at the WWC were noted on the observed photos.

Geology

The geology of the site starts with the deposition of the bedrock soils in shallow seas present during the Cretaceous period. Clayey sandstone, shale, and bentonitic mudstone were formed at the bottom of these seas and are termed the Horseshoe Canyon Formation of the Edmonton Group. Long after the bedrock formation, a river flowed through the Edmonton area which also had several significant tributaries. Deep granular deposits termed Saskatchewan sands and gravels were formed in this river. This river was not the North Saskatchewan River as this flowed after the ice age came and went. However, it is noted that none of the deep testholes in this study encountered the bedrock or Saskatchewan sands and gravel formations.

The next major geologic event was the several advances of large ice sheets across most of North America. These large ice sheets plowed along the bedrock, then deposited a mixture of clay, silt and sand during their retreat, termed glacial clay till. A large lake formed over much of Edmonton near the end of the ice retreat. This lake deposited clay and silt soils, termed Lake Edmonton deposits.

On the west edge of the Lake Edmonton lacustrine deposits, aeolian (wind) deposits consisting of sand and silt were formed.

3.0 FIELD INVESTIGATION

The soils investigation for this project was undertaken on March 18 & 19, 2015 and April 8, 2015 utilizing a truck mounted drill rig owned and operated by SPT Drilling Ltd. of St. Albert, Alberta. Five testholes were drilled at locations shown on the attached site plan. The testholes were advanced to depths in the range of 14.9 and 26.7 meters below ground surface (BGS). The testhole layout was selected by Hoggan Engineering and Testing (1980) Ltd. (Hoggan) prior to drilling and the testholes were surveyed for location and elevation by Hoggan following drilling. The locations of the testholes were situated to avoid intersections and existing utilities. The testholes were drilled within the WWC crossing area. Drilling within the ditches was not possible due to the soft conditions, steep side slopes and power lines.

The testholes were advanced with 150 millimeter diameter solid stem augers in 1.5 meter increments in all of the testholes and probeholes. A continuous visual description, which included the soil types, depths, moisture, transitions, and other pertinent observations, was recorded on site. Disturbed samples were removed from the auger cuttings at 750 millimeter intervals for laboratory testing. Standard Penetration Tests c/w split spoon sampling was also taken at regular 1.5 meter intervals.

Following the drilling operation, slotted piezometric standpipes were inserted into all testholes for watertable level determination. The testholes were backfilled with cuttings, with bentonitic seals placed at the surface. Watertable readings were obtained between 12 to 13 days, 21 to 22 days and 27 to 28 after completion of drilling.

An additional probehole and standpipe was installed near Testhole 2015-02 in order to confirm the watertable readings in that testhole.

4.0 LABORATORY TESTING

All disturbed bag samples returned to the laboratory were tested for moisture content. In addition, the plastic and liquid Atterberg Limits and soluble soil sulphate concentrations were determined on selected samples. A grain size analysis was conducted on selected coarse grained samples. The Shelby Tube samples were tested for unconfined compressive strength and dry density. Lab results are included on the attached testhole logs located in Appendix I.

5.0 SOIL CONDITIONS

A detailed description of the soils encountered is found on the attached testhole logs in Appendix I. In general, the soil conditions at this site consisted of surficial asphalt and gravel underlain by clay fill, overlaying sand and/or lacustrine high plastic clay underlain by silt. The final soil encountered in the testholes was clay till.

Hot mix asphalt was noted at the surface of all testholes drilled at road surface. The asphalt was measured between 80 to 150 millimeters thick. Below the asphalt, moist, brown, well graded, dense, gravel was encountered to depths in the range of 450 and 700 millimeters BGS. The asphalt and gravel thicknesses are known at testhole locations only and may vary in between.

Fill was encountered below the asphalt in Testholes 2015-01 to 2015-04. The clay fill was typically, moist, very stiff, and medium to high plastic in nature and featured trace organics. The clay fill featured traces of coal, oxides, and pebbles throughout. In addition, the clay fill featured sandier areas within the deeper fill at the WWC crossing. The clay fill was encountered to depths in the range of 2.0 to 11.4 meters BGS in the testholes. In Testholes 2015-03 and 2015-04, an organic layer, approximately 0.1 to 0.8 meters thick was noted at the transition of the clay fill to the native clays. As mentioned previously, testhole drilling was not possible in the ditches; hence organic depths may vary away from the road.

Below the clay fill in Testhole 2015-02, silty sand was encountered. The sand was typically brown in colour and very moist to wet and compact in nature. The sand was encountered to a depth of approximately 8.8 meters BGS. Also, below the clay till in Testhole 2015-02 at a depth of approximately 19.0 meters BGS, a wet sand layer was encountered. This sand layer was generally wet, gravelly and featured traces of shale chips. The sand layer was encountered to testhole termination depth of approximately 21.0 meters BGS in Testhole 2015-02.

Below the clay fill in Testholes 2015-01, 2015-04 and 2015-05 and below the sand in Testhole 2015-02, lacustrine clay was encountered. The clay was typically moist and very stiff near the surface and became very moist to wet, medium plastic and firm to soft roughly 2 to 3 meters into the layer. The lacustrine clay transitioned into a very moist to wet clayey, sandy silt with increased depth. The silt was grey in colour, low to medium plastic in nature and was typically very soft, saturated and sensitive in nature. The clay and silt was encountered to depths in the range of 10.2 to 14.0 meters BGS.

Below the lacustrine clay and silts in Testholes 2015-01, 2015-02, 2015-04 and 2015-05 and the clay fill and organics in Testhole 2015-03, silty, sandy, glacial clay till was encountered. The clay till was typically moist with very moist areas and featured traces of coal, oxides, pebbles and the occasional sand lens or seam. The clay till was generally medium plastic in nature with a stiff to very stiff consistency. The clay till was encountered to testhole termination depths of 14.5 meters BGS in Testholes 2015-01, 2015-04 and 2015-05 and termination depth of 26.7 meters BGS in Testhole 2015-03 and to a depth of approximately 19.0 meters BGS in Testhole 2015-02.

During drilling, free water and slough were encountered in most of the testholes. See table in the next section for summary of free water and slough levels in each testhole at completion of drilling.

6.0 GROUNDWATER CONDITIONS

The groundwater table within the study area was generally moderate to low throughout the project area. The water table varied between 3.8 and 8.2 meters BGS. Three sets of watertable readings were taken, with the results shown in the table below.

Groundwater Table Readings
Proposed 199 Street Upgrades - Stage 1
(Metres Below Ground Surface)

Testhole	Elevation	Conditions At Testhole Completion	12 to 13 Day 30-Mar-15	21 to 22 Day 8-Apr-15	27 to 28 Day 14-Apr-15	Watertable Elevation
2015-01	683.04	5.2m water, 5.2m slough	5.73	5.60	5.65	677.39
2015-02	681.05	8.5m water, 4.3m slough	3.75	3.75	3.80	677.25
2015-03	680.88	16.3m water, 2.6m slough	9.21	7.82	7.79	673.09
2015-04	682.37	No water, No slough	5.85	5.75	5.85	676.52
2015-05	687.17	4.3m water, No slough	8.09	8.16	8.09	679.08

It should be noted that water table levels may fluctuate on a seasonal or yearly basis with the highest readings obtained in the spring or after periods of heavy rainfall. The above readings would be near the average seasonal levels.

The water level in Testhole 2015-02 indicated that the groundwater level is in the clay fill zone. This seemed peculiar. The standpipe was pumped from the water and the water level readings were further observed to be at the same level. Therefore, in order to confirm this reading, a second testhole was drilled next to Testhole 2015-02 in order to isolate the watertable within the sand. The watertable reading in the second testhole indicated a ground water level reading of approximately 5.6 meters BGS, within the native sand layer. Given that the higher groundwater level reading of 3.8 meters BGS has more of an adverse effect on the development, the higher reading was used in all of our analysis.

7.0 RECOMMENDATIONS

7.1 Underground Utilities

7.1.1 Open Excavation

1. The clay fill, upper clay, upper sand and clay till materials encountered in the testholes are considered fair to satisfactory while the lower clay, silt and sand material would be considered poor for the installation of underground utilities incorporating the City of Edmonton backfilling and compaction requirements. The clay fill, upper sands and clays, and clay till were near to slightly above optimum moisture content, while the lower silty clays, sands and silts were well in excess of optimum moisture content. Topsoil and other organic materials are not considered suitable for backfill material. The design sewer depths should minimize the cuts as much as possible due to the soft sensitive soils with depth.
2. Although the watertable was moderate to low between 3.8 and 8.2 meters BGS in the testholes, it would be considered high to moderate considering the proposed utility depths of approximately 6 to 12 meters BGS for this project. Saturated soil conditions, sloughing and ingressing groundwater will likely be encountered in most of the trenches at this site. The amount of ingressing water and sloughing conditions is dependent on the depth of utility design elevation compared to the water table. The amount of groundwater infiltration is expected to be slight to significant in the clay, clay till and silt areas and increased in the sand areas and will depend on the watertable versus trench depth at any given location. Temporary dewatering measures will likely be required during utility installation. Pumping from the trenches during installation should be sufficient to maintain trench working conditions in most areas. However, well points are a slight possibility in deeper trench locations. Delays in construction will likely occur in some locations. Weather conditions

will also have a significant bearing on site operations, with rain potentially causing significant problems in areas of open trenches due to the sand soils. Opening relatively long portions of utility trench is not recommended for this site.

3. Standard trenching cutback angles of approximately 45 degrees from the vertical are anticipated for the clay fill soil and some of the upper clay soils, and the lower clay till although some portions of the moister clays, silts and saturated sand seams will likely require increased cutback angles of more than 45 degrees or more in order to remain stable, due to their low strength and elevated moisture contents. Shoring of deeper trenches may be required (only for major sloughing). Actual cutback angles should be determined in the field during construction. Exact stable slope values cannot be pinpointed without detailed and extensive analysis. For this reason, this information should be used as a guideline only and that the optimum cutback angles for utility trenches be determined in the field during construction. The Occupational Health and Safety Code, Part 32 Excavating and Tunnelling should be strictly followed, except were superseded by this report.
4. Trench widths should be compatible with safe construction operations. The trench width must be wide enough to accommodate pipe bedding and compaction equipment.
5. Temporary surcharge loads, such as spill piles, should not be allowed to within 3.0 meters of an unsupported excavation face, while mobile vehicles should be kept back at least 1.0 meter. All excavations should be checked regularly for signs of sloughing or failures, especially after rainfall periods.
6. Pipe bedding and trench backfill procedures should adhere to the City of Edmonton specifications as outlined in The Servicing Standards manual. The backfill material beneath and above the pipe should be an approved bedding sand material where conditions allow. This material should be hand placed and hand tamped, with care taken to fill the underside of the pipe. The City of Edmonton trench bedding types are available in their specifications and are considered suitable. However, ingressing groundwater was encountered in many of the testholes around the site. To overcome the installation difficulties which may be encountered where ingressing groundwater and/or poor bearing conditions may be a problem, it is recommended that a washed rock and geotextile separator be utilized for pipe bedding in these areas. The washed rock and geotextile configuration should be determined

in the field during construction. The need for this configuration may be considerable at this site.

7. The moisture content of the clay fill was typically moist and near optimum moisture content. Minor moisture conditioning is anticipated for the existing clay fills encountered in the testholes. The moisture content of the silty clays in the testholes was variable, but was generally moist to very moist and wet with increased depth. The sand was typically dry to damp above the ground water table and very moist to wet below the ground water level. The clayey silts were typically wet and saturated with increased depth. The variable condition of the soils will cause a corresponding variability in the utility trench pipe bedding and backfill conditions. Some occasional wetting or drying will likely be required at this site to meet the moisture content criteria and adequately construct a platform for surface utility construction. The higher plastic clay materials should be moisture conditioned to a minimum of 1 percent over optimum moisture content (equal to approximately 3 percent above plastic limit) to help reduce swelling. Trenching operations may be slowed down due to the required moisture conditioning. Failure to adequately moisture condition the trench backfill may result in swelling or subgrade softening of the trench backfill. In occasional moister areas, drying or mixing of the backfill prior to placement in the trench will be required when adequate compaction cannot be achieved at the natural moisture content.

8. The majority of native inorganic soils and clay fill encountered in the testholes within the noted project area geotechnical investigation will meet the minimum 72 kPa allowable bearing capacity required by EPCOR for thrust block standard design. However, a portion of the native soils encountered will have an allowable soil bearing capacity that falls below the minimum 72 kPa. In the area of these testholes, thrust block designs should be modified to accommodate a design allowable bearing capacity of 50 kPa. The chart below depicts the testholes and their respective recommended bearing capacity at each individual testhole location. Engineered fill should have an allowable bearing capacity above 72 kPa for thrust block design.

   It is emphasized that soil conditions may vary away from the testhole locations. All thrust block excavation should be inspected to confirm the bearing capacity during construction prior to placement of concrete.

Watermain Thrust Blocks - Recommended Soil Bearing Values

Proposed 199 Street Upgrades - Stage 1

Testhole	Depth (m)	Allowable Bearing Capacity (kPa)	Testhole	Depth (m)	Allowable Bearing Capacity (kPa)
2015-01	0-13.1	50	2015-04	0-9.0	50
	13.1-14.9	Minimum 72		9.0-14.9	Minimum 72
2015-02	0-21.0	Minimum 72	2015-05	0-7.0	50
2015-03	0-26.7	Minimum 72		7.0-14.9	Minimum 72

9. Trench compaction requirements of the City of Edmonton are 100 percent of the One-Point Proctor Density above a depth of 1.5 meters, and 97 percent of the One-Point Proctor Density below this level. The maximum lift thickness is 300 millimeters. This degree of compaction should be achievable with occasional mixing or moisture conditioning of the trench backfill in portions of the trench as mentioned.

10. It should be noted that the ultimate performance of the trench backfill is directly related to the consistency and uniformity of the backfill compaction, as well as the underground contractors construction procedures. In order to achieve this uniformity, the lift thickness and compaction criteria should be strictly enforced.

Trench Backfill Maximum Moisture Content Criteria
Proposed 199 Street Upgrades - Stage 1

Testhole Number	Sample Depth	Liquid Limit	Plastic Limit	Field Moisture Content	Plasticity Index (PI)	Maximum Moisture Content Criteria							
						Uniform Backfill			Conventional Backfill			PL+10 Criteria	
						PI/2	PL+PI/2	+/- Criteria	PI/3	PL+PI/3	+/- Criteria	PL+10	+/- Criteria
2015-01	0.6 m	42.1	20.8	16.3	21.3	10.7	31.5	-15.2	7.1	27.9	-11.6	30.8	-14.5
2015-01	9.1 m	26.5	23.3	30.4	3.2	1.6	24.9	5.5	1.1	24.4	6.0	33.3	-2.9
2015-01	9.4 m	26.0	21.6	30.2	4.4	2.2	23.8	6.4	1.5	23.1	7.1	31.6	-1.4
2015-02	1.5 m	46.5	12.3	19.2	34.2	17.1	29.4	-10.2	11.4	23.7	-4.5	**22.3**	-3.1
2015-02	5.3 m	39.4	11.3	19.8	28.1	14.1	25.4	-5.6	9.4	20.7	-0.9	21.3	-1.5
2015-02	16.0 m	31.9	12.1	15.9	19.8	9.9	22.0	-6.1	6.6	18.7	-2.8	22.1	-6.2
2015-03	6.9 m	41.0	12.4	23.6	28.6	14.3	26.7	-3.1	9.5	21.9	1.7	22.4	1.2
2015-03	5.4 m	50.8	15.9	21.6	34.9	17.5	33.4	-11.8	11.6	27.5	-5.9	**25.9**	-4.3
2015-03	11.0 m	21.0	14.2	12.3	6.8	3.4	17.6	-5.3	2.3	16.5	-4.2	24.2	-11.9
2015-03	23.5 m	29.2	15.2	20.0	14.0	7.0	22.2	-2.2	4.7	19.9	0.1	25.2	-5.2
2015-04	3.7 m	58.9	16.1	24.8	42.8	21.4	37.5	-12.7	14.3	30.4	-5.6	**26.1**	-1.3
2015-04	8.4 m	28.5	21.0	30.4	7.5	3.8	24.8	5.7	2.5	23.5	6.9	31.0	-0.6
2015-05	1.5 m	49.9	15.1	30.0	34.8	17.4	32.5	-2.5	11.6	26.7	3.3	**25.1**	4.9

Notes:
- City specifications state that when the plasticity index criteria for maximum moisture content exceeds 10 percent over the plastic limit, the plastic limit plus 10 percent shall govern.
- All values are percentages.
- Bold values of PL+10 are governing criteria.
- Chart shows only the samples which were tested for Atterberg Limits. See testhole logs for all moisture content data.

7.1.2 Trenchless Installation

1. It is understood that trenchless installation may be utilized as the method of construction of the deep underground utilities, especially under WWC. The trenchless method to be used should be determined by the underground contractor.
2. Trenchless installation through the site clay fill, clay, and clay till soils will be considered fair to satisfactory while installation through the sand encountered in Testhole 2015-02 and the lower silt is considered fair. The sand and silt are susceptible to sloughing and squeezing, especially under the water table, as these soils are sensitive to disturbance. The mud composition may need alteration during installation to account for the variable soil conditions. Installation delays may occur due to the variable nature of the site soils.
3. Trenchless installation in the clay till soils encountered in the testholes may encounter some difficulties due to wet sand and gravel lenses and potential cobble and boulders, as the soil is a glacial deposit.
4. Exact potential for “frac-out” is difficult to determine, but it is generally considered low in the clay, silt and clay tills and moderate to high in the clay fill and sands soils. As a minimum, the contractor should review soil conditions on a continuous basis and take proper measures to prevent “frac-out” from occurring. An emergency “frac-out” response plan and contingency crossing plan that outline the protocol to monitor, contain and clean-up a potential “frac-out” should be in place prior to construction.
5. It is recommended that the drilling contractor follow standard horizontal directional drilling (HDD) practices. Such HDD practices can be found in “Horizontal Directional Drilling Good Practice Guidelines, Third Edition” as recommended by North American Society of Trenchless Technologies.

7.2 Surface Utilities

7.2.1 General Road Construction

1. The subsurface inorganic soil conditions encountered are considered generally fair to poor for the construction of roads, curbs, and sidewalks. Topsoil and all other deleterious materials along the road alignment should be removed prior to construction of the embankment across the ravine.
2. A main concern for surface utility construction at this site is the elevated moisture content of the lower silty clay, silt and sand materials. The near surface clay and clay fill is medium to

high plastic and was slightly above its optimum moisture content, but mixing and disturbance during underground utility installation will degrade the soil conditions. Extra subgrade work beyond standard scarification and re-compaction and cement stabilization may be required in order to construct an adequate working platform for the pavement structure placement and long term support. It is noted that the degree of trench backfill drying during underground utility installation affects the soil conditions for road and sidewalk construction, with increased drying improving the soil conditions.

3. The near surface site clays and clay fill are of low to moderate frost susceptibility, with the susceptibility becoming higher in the sands, silts and silty clay soils encountered at depth. A high watertable within approximately 3.0 meters of the road surface is required for significant frost heaving to occur. The closer the watertable is to the surface, the higher is the frost heave potential. The standpipes for this project have stabilized below this level, between 3.8 and 8.2 meters BGS, and as such, no frost heave concerns are foreseen, provided significant cuts are not made. For frost protection measure, the sand, silt and very silty low plastic clay backfill should be kept 1.5 meters or more below the subgrade.

4. Cement stabilization is the recommended minimum subgrade treatment for this site. For stiff clay subgrade, minimum 10 kilograms of cement per square meter of subgrade should be mixed to a depth of 150 millimeters, and re-compacted to 100 percent of Standard Proctor Density (SPD) near optimum moisture content. For soft to firm clay subgrade, 20 to 30 kilograms of cement per square meter of subgrade mixed to a depth of 300 millimeters would be required. The exact cement content and depths should be decided in the field based on a proof roll. Weather and time of year will also be factors.

   The subgrade should be inspected and proof rolled by qualified personnel after final compaction and any areas showing visible deflections should be repaired prior to paving.

5. If drying is not possible and cement stabilization fails to produce an adequate subgrade, replacing the subgrade with a gravel sub-base would be applicable. A pit-run gravel sub-base, 600 to 900 millimeters thick placed over a woven geotextile (Nilex 2006 or equivalent) is estimated for this purpose. The need for this sub-base should be low, but should be budgeted as a contingency for poor weather. The extent of subgrade

replacement should be decided on site during construction. The need for this measure is anticipated to be low at this site.

6. Surface water will often collect within the granular base, causing subgrade softening and pavement damage. Therefore, it is recommended that wic drains to be installed in the gravel road base at the curb bottom locations. The wic drains must be properly attached to the catch basins. Good drainage within the gravel base is imperative for lasting structural performance. The overall cross slope of the road subgrade should be as least 2 percent towards the wic drain connected to catch basins. Care must be taken not to allow any excess moisture into these soils.
7. It is recommended that all areas beyond the back of curb/sidewalk be landscaped as soon as possible to avoid water permeating into the subgrade from free standing puddles. The near surface clay soils encountered in some of the testholes throughout this area exhibit a moderate to high swelling potential. It is important that subgrade soils not be allowed to dry excessively when exposed, and moisture contents are kept slightly over optimum.
8. It is understood 199 Street will be a four lane divided arterial road. An estimated traffic volume of 35,750 vehicles per day in 2047 was found in the following report.

    - Riverview Neighbourhoods 1, 2 & 3, Neighbourhood Structure Plan, Transportation Impact Assessment, dated November 17, 2014, prepared by Bunt & Associates, file # 3366.03

    It was assumed that trucks account for 7 percent of the traffic, with an aggregate truck factor of 1.2, a growth rate of 3 percent per year, as well as a design life of 20 years. Based on the above assumptions, the total traffic loading was estimated to be approximately 2.9 x 10^6 ESALs. Based on an estimated California Bearing Ratio (CBR) of 3.0 percent, the following staged pavement design is recommended for this site.

Recommended Staged Roadway Structures
Proposed 199 Street Upgrades - Stage 1

	Traffic Loading	Arterial (2.9x10^6 ESALs)
Stage 1	Asphaltic Concrete (10mm-HT)	45 mm
	Asphaltic Concrete (20mm-B)	100 mm
	Crushed Gravel (3-20 or 3-63)	350 mm
Stage 2	Asphaltic Concrete (10mm-HT)	50 mm
Note:	10mm-HT = City of Edmonton Designation Asphaltic Concrete 10mm-Heavy Traffic 10mm-B = City of Edmonton Designation Asphaltic Concrete 20mm-Base 3-20 = City of Edmonton Designation 3 Class 20 aggregate All granular base material should be compacted to 100 percent of the Standard Proctor Density in maximum 150 mm lifts.	

Our firm should be advised if updated traffic loading information becomes available and the pavement design should be modified accordingly.

9. At the connections between the old and new pavements, the new subgrade should be tapered to match the existing subgrade to ensure even drainage within the gravel bases.

7.2.2 Embankment Construction

1. Grading plans were forwarded to our firm and they indicate that no significant cuts are planned for this area. The new road grades will match the existing grades.
2. It is understood the existing embankment across the WWC ravine will be widened. The recommended construction method for embankment widening is to remove the existing embankment side slopes in a step fashion. The side slopes should be benched in order to obtain bonding between the existing grade and the new embankment. Proper organic stripping is a must as well.
3. In order to widen the embankments slopes, the creek will require dewatering. This can be achieved by construction of a clay dam and pump system and/or diversion of the creek. Any organic soils encountered at creek bottom will have to be removed. Our firm should inspect the fill areas in order to ensure that all unsuitable materials are removed.
4. The excavation of the existing side slopes in preparation for the proposed widening may expose the soft lower clay near the bottom of the creek. Construction traffic may encounter difficulties travelling on this surface. A clay pad 600 millimeters thick may be required in soft soil areas to allow for grading construction equipment to operate.

Judgment should be used in the field at the time of construction to determine an initial lift thickness.

5. The embankment fill material must be cohesive and non-organic to ensure a positive bonding to the existing grade surface and provide erosion resistance. The source of the embankment fill is not known at this time. It is recommended that the clay fill consist of a medium to high plastic clay material as these soils will have a low susceptibility to erosion. The import clay should be approved by JRP prior to use. All grading fill within the embankment should be compacted to a minimum 98 percent of Standard Proctor Density (SPD). All fill should be placed and compacted in maximum thickness lifts of 150 millimeters.
6. The stability analysis included assessment of end of construction (short term) condition based on effective stress analyses with construction generated excess pore pressures as well as long term stability after pore pressure dissipation. Pore pressures generated in the embankment fill and in the underlying native clay till layer during fill placement and compaction have been estimated based on B-bar value of 0.3.

   The desired minimum side slope for the embankment at the proposed creek to minimize the environmental impact on the WWC is 2H:1V. Global stability analysis on the side slopes based on the proposed 2H:1V indicated a non-stable slope. Therefore, the slope will require slope stabilization measures. Global stability analysis on the reinforced side slopes was completed. Based on the results of the analysis, the stabilization measures should consist of placing a bi-axial geo-grid (Tensar BX1200 or similar) at the interface of the native in-situ clay tills and the first engineered fill layer, and then utilizing a uni-axial geo-grid (Tensar UX1100 or similar) every one meter of fill placement after that. The analysis indicated stable side slopes once the reinforcement is applied within the compacted clays. The reinforcement should extend transversely with the roadway underneath the entire approach ramp footprint at the top three meters below top of subgrade and a minimum 20 meters from the edge of the side slopes below three meters below the subgrade. This reinforcement configuration is shown on the slope stability graphics in Appendix II.
7. End of construction and long term settlement analysis was carried out using the computer program FoSSA to estimate ground settlement under the new approach fill loading over

the project. The approach fill geometry was based on the existing slope profiles at WWC provided to Hoggan by Stantec and the proposed 2H:1V side slopes. The underlying native lower clay, encountered mainly along the north facing slope of WWC, will consolidate as a result of the weight of the new fill. However, it is assumed that the ground was level with the surroundings before the ravine was created. The approximately 12 meter tall embankment would bring the grade back to the original level before the ravine was present. Therefore, the loading pressure from the embankment should be below the pre-consolidation pressure. The clay till is very stiff and moist in nature and is considered over consolidated and should not settle significantly with the additional fills. Settlement of the existing fill below the new fill is considered negligible.

No consolidation test was performed for this site. Based on our knowledge and experience of the lacustrine clay material, consolidation parameters (recompression index, C_r = 0.02, compression index C_c = 0.2, initial void ratio = 0.8) were assumed in the analysis. The analysis showed the maximum total consolidation of the underlying native clay soil would be approximately 0.2 meters, at the point of highest new fill.

It was also estimated the self-weight settlement of future embankment fill will be approximately 1 percent of the fill height (0.12 meters) and should take two to five years to occur.

It should also be noted that the settlement will not be even and will vary with fill height. This unevenness should be accounted for in the design, construction and future maintenance of the project, including the proposed underground utilities.

8. Runoff near the ravine crossing should be intercepted and directed to erosion protected channels or storm sewer. The finished embankment side slope should be covered with vegetation as soon as possible for erosion protection.

7.2.3 Culvert Installation

1. The soils encountered at the culvert elevation consisted of clay till and is considered suitable for a culvert installation. The design and installation of the culvert should be done in accordance with the City of Edmonton Specifications, except where superseded by this report.
2. Topsoil, clay fill, and organic soils should be completely removed from the culvert base area, including below the side backfill. The depth of the culvert subgrade will be below

the groundwater table; therefore a temporary dewatering system will be required. This system would likely consist of a perimeter ditch draining to a sump area away from the culvert base. Dewatering measures are best determined onsite during construction. The proper compaction of the culvert granular base may not be achievable without the dewatering.

3. The culvert excavations should be performed by a backhoe operating remote from the bearing surface, due to the watertable. The depth of the excavation should be sufficient for the pipe to lie in the native clay till material. The standard minimum subcut of 0.6 meters below the culvert inlet will be adequate for this site. The width of the excavation should be the greater of 2 pipe spans or 1 pipe span plus 3.0 meters. The excavation should extend longitudinally from the inlet to the outlet.
4. Backfill will be defined as either structural backfill, which is material placed in the critical zone around the pipe in accordance with the City of Edmonton specifications, or embankment fill, which is material placed beyond the structural backfill envelope.
5. Standard trenching cutback angles of approximately 45 degrees from the vertical are anticipated for the site, although some portions of the moister clays, lower very moist to wet clays may require increased cutback angles of more than 45 degrees in order to remain stable, due to their low strength and elevated moisture contents. Actual cutback angles should be determined in the field during construction. Exact stable slope values cannot be pinpointed without detailed and extensive analysis. For this reason, this information should be used as a guideline only and that the optimum cutback angles for utility trenches should be determined in the field during construction. The Occupational Health and Safety Act, Part 32 Excavations and Tunnelling should be strictly followed, except were superseded by this report.

6. All structural backfill material should be comprised of granular material. The placement of a non-woven geotextile separator between the subcut floor and the first lift of structural fill is recommended for this project. Placement of the fabric should be done in accordance with the supplier’s instructions. An initial lift of 450 millimeters of lightly compacted structural fill may need to be placed in order to achieve an adequate bridge above the clay subsoil. All subsequent lifts should be compacted to a minimum of 98 percent of Standard Proctor Density in maximum 150 millimeter lifts, at optimum

moisture content. Compaction in the haunch areas should be done manually in 100 millimeter lifts, and should still meet the above compaction requirement.

7. Exceptions to this compaction requirement are recommended for the 150 millimeter lift immediately below the bottom of the pipe (the bedding material), and the 300 millimeters of material immediately above the top of the pipe. These areas should have minimal compaction. The bedding material should be pre-shaped to the bottom of the pipe. The bedding shall be omitted in the clay seal areas. The 300 millimeters of material over the top of the pipe should be placed and compacted without vibration. Material above this level should meet the above compaction requirements.
8. In regards to settlement, the proposed culvert should be founded on native clay till soils. No significant consolidation settlement of the native clay till soil is expected. No significant heave should occur. If the gravel pad below the culvert is placed in thick lifts some settlement may occur once the base is reloaded, rough estimates of settlement are up to 25 millimeters.
9. Clay seepage cut-offs are recommended at both the inlet and outlet of the culvert structure. These may be eliminated from the culvert extensions where cutoffs already exist. The length of seal should be equal to 2 times the diameter of the culvert (as measured at the invert of the pipe). The length of the seal may be reduced when using large diameter culverts. The clay should be compacted to a minimum of 98 percent of Standard Proctor Density in maximum 150 millimeter lifts. This includes the area below the invert. The shape of the cutoffs will be as defined in the City of Edmonton specifications.
10. Compaction of both the structural fill and clay seal fill shall be by equipment moving parallel to the longitudinal axis of the pipe. Above 300 millimeters above the top of the pipe, the equipment should operate perpendicular to the longitudinal axis of the pipe. Backfill should progress simultaneously on both sides of the pipe. Backfill on one side of the pipe should not exceed the other by more than 300 millimeters. Care must be taken to ensure that no deflections in the pipe are caused by the backfill procedures. It is recommended that the rise and span of the pipe be measured at the center and 1/4 distances from each end during construction.

11. As requested, samples of soil and water were retained for resistivity and pH testing, as well as for the presence of sulphates, chlorides, and other salts. All samples were submitted to ALS Laboratories for testing. The results are as follows:

Soil Corrosion Testing Results

	Soil Samples		
Property	**Clay Fill 2015-02 @ 8.2 m**	**Clay Till 2015-02 @ 12.9 m**	**Water Sample WWC**
pH	7.94	8.21	8.09
Conductivity (paste)	0.356 dS/m	0.631 dS/m	586 uS/cm

7.3 Bridge Foundation

Part of the 199th Street Upgrades is the construction of a wildlife passage. It is understood from Stantec that the wildlife crossing will consist of a roughly 15 meter long single span bridge. The bridge construction will allow for roughly 4.5 meters of head space for the animals. In addition, MSE wing walls will be constructed on the side slopes. Testholes 2015-02 and 2015-03 were drilled at the proposed bridge location. A Sketch of the preliminary bridge design is available in Appendix III.

The following recommendations are provided to aid in the design and construction of the bridge.

7.3.1 Cast-in-Place Piles

1. The soils encountered at this site are suitable for a cast-in-place pile foundation. The structure may be founded on an adequately reinforced grade beam or pile cap supported by bored, cast-in-place, concrete piles. The design capacity can be calculated on the basis of factored skin friction or end bearing values. A combination of the two bearing modes may be utilized for individual piles.
2. The factored skin friction values that may be used are as follows:

Testhole 2015-02:

Soil Stratum	**Ultimate Skin Friction Resistance**	**Geotechnical Resistance Factor**	**Factored Skin Friction Resistance**
New Clay Fill (Top 1.5m)	0 kPa	0.4	0 kPa
New Clay Fill (Below 1.5m)	60 kPa	0.4	24 kPa
Existing Clay Fill	55 kPa	0.4	22 kPa
Sand	42 kPa	0.4	17 kPa
Clay Till*	90 kPa	0.4	36 kPa
Clay Till**	100 kPa	0.4	40 kPa

* (from Elevation 670.5 to 660.0 m)
** (below Elevation 660.0 m)

Testhole 2015-03:

Soil Stratum	**Ultimate Skin Friction Resistance**	**Geotechnical Resistance Factor**	**Factored Skin Friction Resistance**
New Clay Fill (Top 1.5m)	0 kPa	0.4	0 kPa
New Clay Fill (Below 1.5m)	60 kPa	0.4	24 kPa
Existing Clay Fill	55 kPa	0.4	22 kPa
Clay Till*	75 kPa	0.4	30 kPa
Clay Till**	100 kPa	0.4	40 kPa

* (from Elevation 669.0 to 662.0 m)
** (below Elevation 662.0 m)

Due to the close proximity of Testholes 2015-02 and 2015-03, the factored skin friction resistance below Elevation 660.0 for Testhole 2015-02 can be assumed to be the same as the provided factored skin friction resistance provided for Testhole 2015-03.

The above values include the total of all live and dead loads. Considering the effects of frost and seasonal moisture changes, the friction value for the first 1.5 meters of pile should not be considered in design.

3. To account for lateral load resistance, it is understood that batter cast-in-place piles may be required. Batter of cast-in-place skin friction piles are considered suitable at this site. A maximum batter angle of 1H:4V is recommended.

4. It should be noted that Serviceability Limit States (SLS) addresses the functional performance of a structure as opposed to Ultimate Limit States (ULS) which addresses failure. Therefore, the geotechnical issue for SLS loading on piles is settlement rather than

bearing capacity. While the predicted settlement of a pile is not readily calculated, the typical expectation of a structure placed on a pile foundation is essentially no settlement at all. In this case, the expected settlement for a skin friction pile loaded to the above factored bearing values would be less than 10 millimeters. Therefore, the design values provided in Item 7.3.2 are considered by the writer to be ULS and SLS values, if 10 millimeters of settlement is acceptable. It should be noted that piles in the new deep fill will have more involved settlement consideration due to the large negative skin friction/downdrag caused by the new fill. The existing fill is greater than 50 years old and is considered completely consolidated.

5. The preliminary bridge design drawing indicate that the piles along the south abutment will go through a maximum of 5.0 meters of new clay fill as part of the widening of the side slopes and the replacement of the existing culvert. Piles located in the new deep fill soils will be subjected to downdrag forces (negative skin friction) due to potential long term settlement of the new clay fill. Negative skin friction and downdrag forces generally do not affect the geotechnical/Ultimate Limit State capacity of the piles. Downdrag forces increase the pile settlement and should therefore be accounted for in the Serviceability Limit State assessment of the piles.

Downdrag forces do increase the axial load on the pile and the pile structural strength must account for this extra load.

The amount of settlement from the new fill is estimated at 1 percent of the fill height. The magnitude of the force is independent of the amount of settlement. Although, if a large amount of settlement occurs the fill could become stronger; this may increase the downdrag.

The downdrag load can be expressed as negative skin friction associated with the settling soil, and is given below.

Soil Stratum	**Negative Skin Friction Value**
New Clay Fill	-60 kPa

It should be noted that the negative skin friction is un-factored, as it essentially represents a load and not a skin friction resistance. Two loading scenarios should be considered when negative skin friction is involved. The first scenario is the normal design where the ULS factored live (transient) load and the factored dead (permanent) load are added and resisted by the geotechnical resistance of the pile. No drag load is considered in this scenario because drag load and transient load never combine. The second scenario is the factored dead (permanent) load combined with the drag load which must be resisted by the structural capacity of the pile.

Another significant design factor when addressing settling soils which cause negative skin friction is the settlement of the pile. The pile settlement will never be more than the soil settlement at the surface, and it is typically significantly less. It is a complex analysis to estimate the pile settlement when you can accurately predict the soil settlement. It is impossible when you cannot accurately estimate the soil settlement. The expected future settlement of the new side slope fill is estimated at 1 percent of the fill height. The maximum fill height for the side slopes is 12 meters, while the maximum fill height below the pile head is anticipated to be 5.0 meters.

The sensitivity of the structure to settlement is a large factor. If the structure is sensitive to movement, then the portion of the pile below the fill should be designed to withstand the drag load plus the permanent load utilizing factored resistances.

6. The recommended minimum pile depths at this site for frost uplift prevention in straight shaft piles is 6.0 meters in a non-continuously heated structure. The minimum pile diameter for all piles should be 400 millimeters, with a minimum skin friction pile spacing of 2.5 pile diameters on center. In addition, the minimum spacing between the edges of the bells at the bottom of the piles is 0.3 meters.
7. The clay till encountered in Testholes 2015-02 and 2015-03 were typically moist and very stiff in nature. The clay till encountered in the testholes is considered suitable for end-bearing below the proposed elevations as noted. The factored end-bearing values that may be used are as follows:

Testhole 2015-02

Soil Stratum	**Geotechnical Resistance Factor**	**Factored End-Bearing Resistance**
Clay Till (below Elevation 666.0m)	0.4	400 kPa

Testhole 2015-03

Soil Stratum	**Geotechnical Resistance Factor**	**Factored End-Bearing Resistance**
Clay Till (below Elevation 660.0 m)	0.4	750 kPa

The above values include the total of all live and dead loads. A combination of both skin friction and end-bearing resistance can be included in the design of end bearing piles. Shaft resistance should be neglected for the top 1.5 meters of the pile length, sides of the bell, and within one shaft diameter above the top of the bell.

End bearing piles should extend to a minimum of three bell diameters below the ground or excavation surface, and should have a minimum bell to shaft diameter ratio of 2:1 and maximum bell to shaft diameter ratio of 3:1. The bell should be fully formed in the clay till layer, with the bottom of the bell penetrating the stiff to very stiff areas below the specified elevations. The clay till encountered in the testholes may feature very sandy and gravelly zones and sand lenses, as it is in its nature. Forming a bell in the very sandy areas and in the sand lenses will be difficult. If very sandy layers or sand seams are encountered, it is recommended that the bell bottom be drilled deeper to a less sandy zone where the bell can be adequately formed.

8. All pile holes should be carefully inspected to ensure that no water or slough material is present prior to concrete placement. The ground water level stabilized at levels between 3.8 and 8.2 meters BGS. Also, significant free water and slough was encountered in the testholes. Casing of the piles will likely be required. The depth of casing is anticipated to below the depth of the creek, enough to form a seal. The pile concrete should be placed as soon as possible after the pile has been bored to minimize the volume of ingressing groundwater.
9. Some provision should be made for the possible swelling of the subsoil beneath the pile caps and the effects of frost action. This can be done by providing a void form or other provision for soil expansion beneath the grade beams and pile caps.
10. It is recommended that all piles be adequately reinforced. Concrete for all piles should be adequately vibrated.
11. All structural fill against foundation walls should be an inorganic material compacted in 150

millimeter lifts to at least 98 percent of the corresponding Standard Proctor Density at optimum moisture content.

7.3.2 Driven Piles

1. Driven piles are considered a suitable pile foundation at this site. The driven piles may be timber, pre-cast concrete, or steel H or pipe piles. All piles supporting the structure should be driven to refusal or to resistance as computed by a dynamic pile driving formula, such as the Hiley formula. The recommended maximum blow count in order to prevent pile damage for steel piles is 12 to 15 blows per 25 millimetres, although this should be confirmed after a review of the pile type, loads, and hammer data. It is recommended that all pile driving be conducted under the full-time supervision of geotechnical personnel.
2. With respect to driven piles, the preliminary design length can be calculated based on combined total/effective stress analysis. The theoretical capacity of driven steel H or pipe pile is as follows:

$$Q = r_s A_s D + r_t A_t \qquad \text{where:}$$

Q = Load on the piles (kN)
r_s = Average factored skin friction between piles and soil over applicable length (kPa)
A_s = Minimum perimeter of the pile section (m) [H piles: $A_s = 2(L+W)$; Pipe Pile $A_s=2\pi r$]
D = Effective depth of the pile embedment (m)
r_t = Factored end-bearing (kPa)
A_t = Cross-sectional area of the pile tip (m^2) [plug may be assumed to form for steel piles at this site provided pile depth is a minimum 20 pile diameters]

3. The factored skin friction and end-bearing values (ULS) are given as follows. For driven piles, the end bearing and skin friction bearing modes may be combined.

Testhole 2015-02:

Soil Stratum	Geotechnical Resistance Factor	Factored Skin Friction Resistance	Factored End-Bearing Resistance
New Clay Fill (Top 1.5m)	0.4	0 kPa	N/A
New Clay Fill (Below 1.5m)	0.4	24 kPa	N/A
Existing Clay Fill	0.4	22 kPa	N/A
Sand	0.4	17 kPa	N/A
Clay Till*	0.4	36 kPa	400 kPa
Clay Till**	0.4	40 kPa	750 kPa

* (from Elevation 661.0 to 660.0 m)

** (below Elevation 660.0 m)

Testhole 2015-03:

Soil Stratum	**Geotechnical Resistance Factor**	**Factored Skin Friction Resistance**	**Factored End-Bearing Resistance**
New Clay Fill (Top 1.5m)	0.4	0 kPa	N/A
New Clay Fill (Below 1.5 m)	0.4	24 kPa	N/A
Existing Clay Fill	0.4	22 kPa	N/A
Clay Till*	0.4	30 kPa	N/A
Clay Till**	0.4	40 kPa	750 kPa

* (from Elevation 669.0 to 662.0 m)

** (below Elevation 662.0 m)

Due to the close proximity of Testholes 2015-02 and 2015-03, the factored skin friction resistance and end-bearing resistance capacities below Elevation 660.0 for Testhole 2015-02 can be assumed to be the same as the provided factored skin friction and end-bearing resistance capacities provided for Testhole 2015-03.

4. The driven piles will be subjected to downdrag forces (negative skin friction) due to the placement of the new fill as well as the settlement of the native clay soils. Item 7.3.1.2 should be reviewed for downdrag considerations of driven piles.
5. The actual capacity of a driven pile can only be determined accurately by a pile load test. Hoggan recommends that a wave equation formulae with a factor of safety of 2.5 be utilized for determining pile capacity at the subject site during installation. Alternatively, a pile driving analyser (PDA) may be utilized. Our firm does not have such equipment and would need to sub-consult this work. With PDA analysis, a higher resistance factor of 0.5 (FOS = 2), may be utilized.
6. The recommended minimum pile depths at this site to prevent frost uplift is 6.0 meters in a non-continuously heated structure. In the event that hard driving is encountered, guidelines for refusal criteria can be provided once the pile design and driving equipment have been finalized. Refusal criteria are directly dependent on such factors as pile size, length and wall thickness as well as the specified design load and driving energy.
7. To account for lateral load resistance, it is understood that batter piles will likely be required. A maximum batter angle of 1H:4V is recommended for driven piles.
8. Driven piles at this site may encounter low driving resistance due to strength loss as a result

of quickening of the saturated silt and sand materials. If such low resistance is encountered, the pile should be driven to within 1 meter of its anticipated design elevation and left undisturbed for a minimum of 96 hours. The pile should then be re-driven and the blow counts obtained utilized for load capacity calculation. A longer waiting period may be required for the soils to re-stabilize. This pile set-up should be accounted for in the pile installation plan.

9. The piles must be designed to withstand the bending moments caused by handling, and the design structural loads.
10. The top 1.5 meters of the pile should be neglected due to frost and seasonal moisture changes.
11. It is recommended that driving records be maintained for each pile and all adjacent pile elevations should be monitored during driving. Piles that have heaved due to the driving of adjacent piles should be re-driven. To avoid heaving problems, the spacing and driving pattern used during construction must be planned carefully.
12. The recommended minimum hammer weight for drop and single acting machines is twice the weight of the pile. The driving energy utilized for this project should be maximum $6x10^6$ Newton meters times the cross sectional area (in m^2) of the steel piles. It is recommended that our firm perform a WEAP analysis on the proposed driven steel piles to recommend pile hammer sizes and assess drivability.
13. The head of the pile should be protected by an adequate helmet. The pile head protection should be checked regularly during pile installation to ensure adequate protection is maintained.
14. The pile driving contractor should have adequate experience in driven pile installation.

7.3.3 Shallow Foundations – Wing Walls

1. Four mechanically stabilized earth (MSE) wing walls and abutment retaining walls are planned as part of the construction of the bridge. A footing foundation system is considered geotechnically satisfactory for the MSE as well as abutment retaining walls. Given the nature of the site conditions, the MSE and abutment retaining wall foundations will likely be founded on either undisturbed, native non-organic soil or the side slope clay fill. The factored bearing capacities (Ultimate Limit States) that may be used are as follows:

Soil Stratum	Geotechnical Resistance Factor	Factored Bearing Resistance (Strip Footing)	Factored Bearing Resistance (Spread Footing)
TOPSOIL	0.4	0 kPa	0 kPa
CLAY FILL*	0.4	100 kPa	120 kPa
SAND	0.4	150 kPa	180 kPa

*Engineered fill of the 199th Street side slopes.

These figures include the total of all live and dead loads. All footings within a continuously heated structure should have a minimum 1.5 meters frost cover, with a minimum cover of 2.5 meters for a non-continuously heated structure or exterior isolated footings. Alternatively, the MSE walls may be designed to allow for frost movement or rigid insulation.

2. It is not recommended that footings be constructed below the watertable, as this will require dewatering efforts. It is anticipated that the MSE walls will be constructed above the watertable. Therefore, it does not appear that the watertable will affect footing foundation construction, and no construction difficulties or delays are foreseen.
3. Settlement will be the main concern for the MSE and abutment retaining walls. The south walls will likely experience differential settlement due to the consolidation of the clay fill. It is estimated the self-weight settlement of future embankment fill will be approximately 1 percent of the fill height below wall and should take two to five years to occur. The depth of fill across the wall is difficult to determine but may be in the range of 5 to 7 meters or greater. The north walls should be founded in the native sands and should not experience any long term settlement, as the settlement is considered immediate. The MSE and abutment walls should be designed to account for differential settlement.
4. Care should be taken during construction and the life of the structure to prevent excessive changes in moisture content of the material. Footing excavations should be protected from drying, rain, snow, freezing, and the ingress of groundwater.
5. No loose, disturbed, remoulded or slough material should be allowed to remain in the open footing excavations. Hand cleaning is advised if an acceptable surface cannot be prepared by mechanical equipment. Excavations should be dug with equipment operating remote from the bearing surface.

7.4 Lateral Loads

1. Due to the nature of this project, lateral load information may be required. A coefficient of horizontal subgrade reaction may be applied to the analysis of soil resistance for laterally loaded piles according to the following:

Soil Stratum	Coefficient of Lateral Subgrade Reaction (kN/m^3)
Clay Fill (Top 1.5m)	0
Clay Fill (Below 1.5m)	7,000/d
Sand	7,350/d
Clay Till	11,000/d

(where d = diameter of the pile in metres)

2. For design purposes, the top 1.5 meters of pile length should be disregarded. Additional lateral load information can be provided once pile dimensions have been chosen and the pile stiffness becomes known.

3. The horizontal modulus of subgrade reaction applies to an individual pile or a pile in a group where the pile spacing is greater than about 7 diameters (or flange widths) center to center spacing. For closely spaced piles in groups, there will be interaction between piles and the lateral support to each pile will be reduced accordingly. Pile group interaction may be modelled by applying group reduction factors to the modulus of horizontal subgrade reaction. The group reduction factor will depend on the location of the pile within the group, the least reduction being applied to lead (front) row piles. Group reduction factors are presented in the table below as a function of pile row and the pile spacing to diameter ratio.

Group Reduction Factors for Modulus of Horizontal Subgrade Reaction

(Rollins et al, 2006)

Ratio of Pile Spacing to Pile Diameter (or Width)	**Group Reduction Factors for Modulus of Horizontal Subgrade Reaction**		
	Leading Row Piles	**Second Row Piles**	**Third and Higher Row Piles**
2.5	0.74	0.48	0.30
3.0	0.79	0.57	0.41
4.0	0.86	0.72	0.58
5.0	0.92	0.84	0.72
6.0	0.97	0.93	0.83

Pile loads are assumed to be aligned at right angles to the direction of the load.

4. The estimated internal friction angles and associated lateral load design factors for typical fill soils are listed below. Once proposed fill soils are evaluated, more accurate values can be supplied.

Fill Soil	**Effective Friction Angle**	**K_o**	**K_a**	**K_p**	**γ_T**
CLAY FILL	25°	0.6	0.4	2.5	20 kN/m^3
GRAVEL	36°	0.4	0.3	3.8	21 kN/m^3

The K_o condition would be applicable in a situation where no movement of the structure is allowed, such as the proposed bridge. The K_a condition would be applicable where some movement of the structure is allowed for, such as the wing walls of the proposed structure.

The amount of movement required to produce active (or K_a) earth pressure is a function of the height of the structure, 0.02H, where H is the height of the structure in meters.

7.5 Earthquake Design

1. Based on the soils encountered in the testholes, the upper 30 metres of soil at this site is comprised generally of stiff to very stiff clay soils. As such, for structural design

purposes, this site can be classified as Seismic Site Response Site Class D as per Table 4.1.8.4.A in the Alberta Building Code 2006.

7.6 Cement

Tests on selected soil samples indicated negligible concentrations of water soluble soil sulphates in the near surface clay deposits. The following alternatives are advised to address the sulphate content in the soil:

1. Underground Concrete Pipe

   Concrete used for all underground pipes must be constructed of C.S.A. Type HS (high sulphate resistant hydraulic cement).

2. Curbs and Sidewalks

   All concrete for surface improvements such as sidewalks and curbs may be constructed using C.S.A. Type GU (general use hydraulic cement).

3. Foundation Construction

   All concrete used for residential construction and coming into direct contact with the soil may be constructed with CSA Type GU (general use hydraulic cement). In addition, all concrete subject to freezing must be air entrained with 5 to 7 percent air. Individual locations may show lower concentrations of soluble soil sulphates, and thus additional soil testing on particular sites may prove valuable.

8.0 CLOSURE

This report has been prepared for the exclusive and confidential use of Qualico Communities, Stantec Consulting Ltd., City of Edmonton and authorized agents. Use of this report is limited to the subject proposed roadway upgrade and subject bridge only. The recommendations given are based on the subsurface soil conditions encountered during test boring, current construction techniques and generally accepted engineering practices. No other warranty, expressed or implied, is made. Due to geological randomness of many soils formations, no interpolation of soil conditions between or away from the testholes has been made or implied. Soil conditions are known only at the test boring location. Should other soils be encountered during construction or other information pertinent becomes available, the undersigned should be contacted as the recommendations may be altered or modified.

We trust this information is satisfactory. If you should have any further questions, please contact our office.

Respectfully Submitted:
HOGGAN ENGINEERING & TESTING (1980) LTD.

IBRAHIM RAHIME

Abe Rahime
May 29/15.

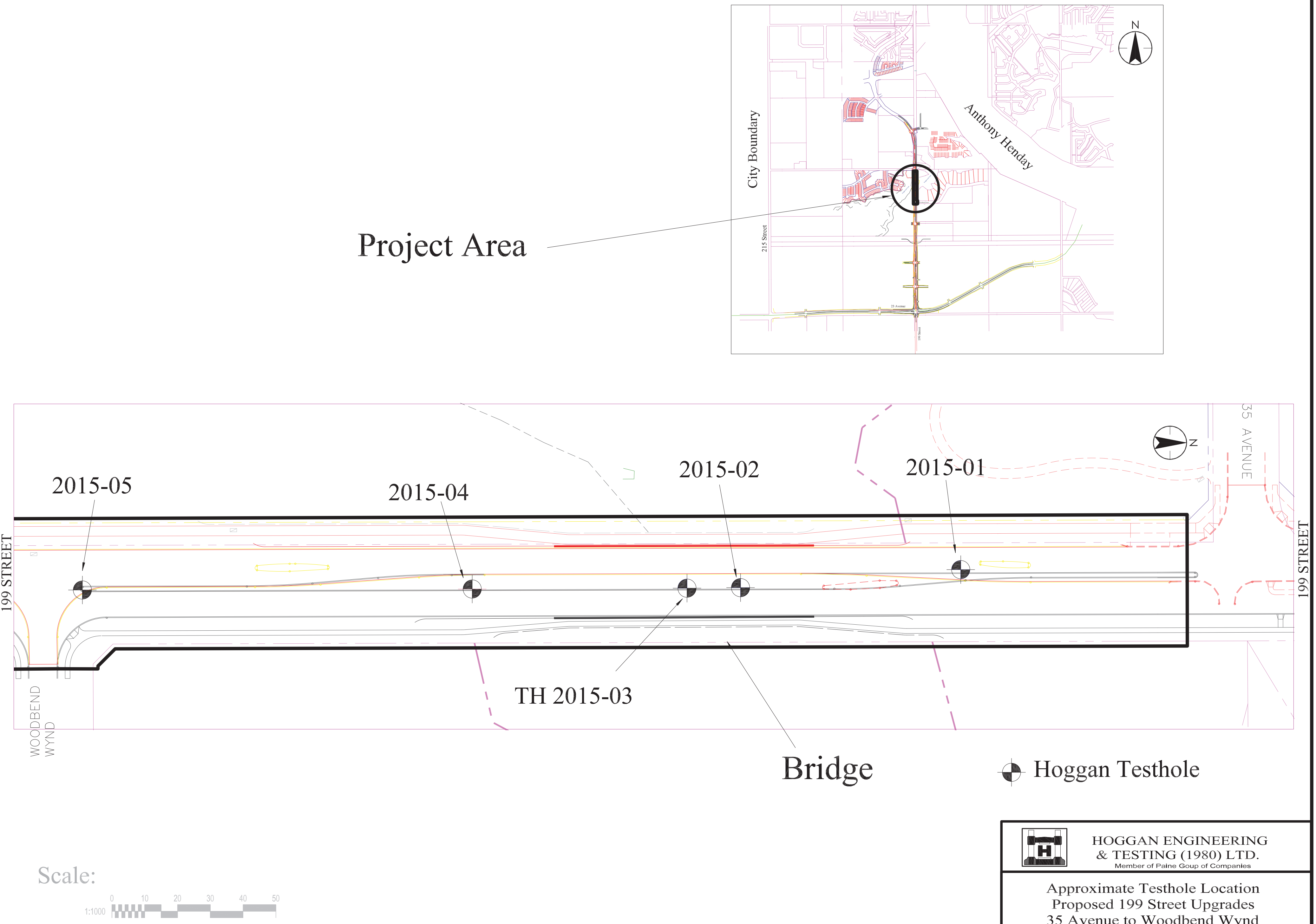
PERMIT TO PRACTICE
HOGGAN ENGINEERING & TESTING (1980) LTD.
Signature R. Evans
Date May 29, 2015
PERMIT NUMBER: P 3691
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

Abe Rahime, P. Eng.

Reviewed By: Rick Evans, P. Eng.

H:\DATA 2015\6004 Stantec Consulting Ltd\6004-38 Proposed 199 Street Upgrade\hr1261sta.doc

A P P E N D I X I – Site Plan and Testhole Logs



HOGGAN ENGINEERING
& TESTING (1980) LTD.
Member of Paine Goup of Companies

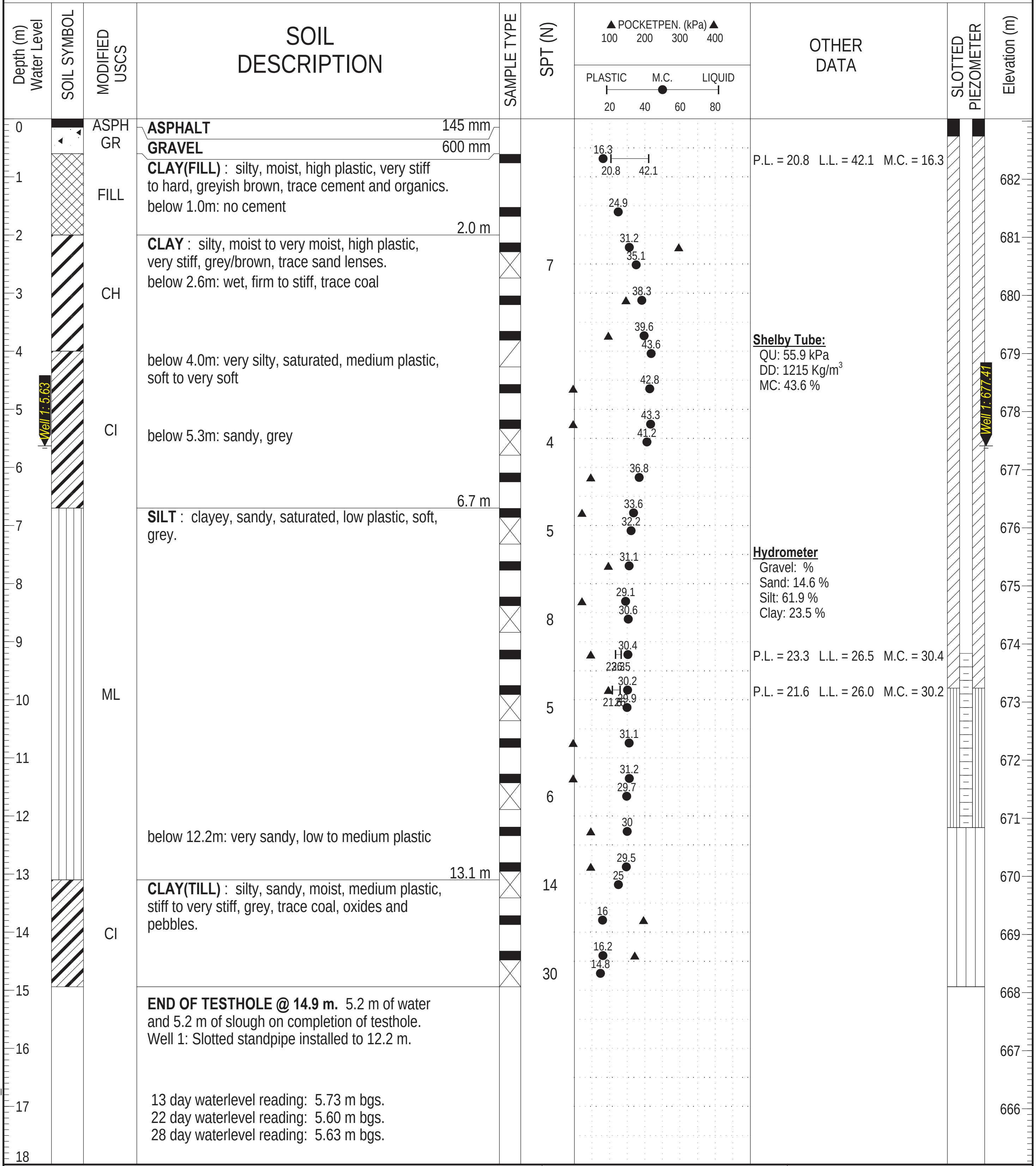
Approximate Testhole Location
Proposed 199 Street Upgrades
35 Avenue to Woodbend Wynd
Edmonton, Alberta

SCALE: 1:1000	DATE: April 2015
FILE #: 6004-38	FIGURE #: 1

PROJECT: Proposed 199 Street Upgrades	PROJECT NO: 6004-38	BOREHOLE NO: **2015-01**
CLIENT: Stantec Consulting Ltd	DRILL METHOD: Solid Stem Auger	ELEVATION: 683.04 m
OWNER: Qualico Communities	LOCATION: As per site plan	

SAMPLE TYPE: ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁚ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



SOIL DESCRIPTION

ASPHALT 145 mm

GRAVEL 600 mm

CLAY(FILL) : silty, moist, high plastic, very stiff to hard, greyish brown, trace cement and organics.
below 1.0m: no cement
2.0 m

CLAY : silty, moist to very moist, high plastic, very stiff, grey/brown, trace sand lenses.
below 2.6m: wet, firm to stiff, trace coal

below 4.0m: very silty, saturated, medium plastic, soft to very soft

below 5.3m: sandy, grey
6.7 m

SILT : clayey, sandy, saturated, low plastic, soft, grey.

below 12.2m: very sandy, low to medium plastic
13.1 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

END OF TESTHOLE @ 14.9 m. 5.2 m of water and 5.2 m of slough on completion of testhole.
Well 1: Slotted standpipe installed to 12.2 m.

13 day waterlevel reading: 5.73 m bgs.
22 day waterlevel reading: 5.60 m bgs.
28 day waterlevel reading: 5.63 m bgs.

OTHER DATA

P.L. = 20.8 L.L. = 42.1 M.C. = 16.3

Shelby Tube:
QU: 55.9 kPa
DD: 1215 Kg/m^3
MC: 43.6 %

Hydrometer
Gravel: %
Sand: 14.6 %
Silt: 61.9 %
Clay: 23.5 %

P.L. = 23.3 L.L. = 26.5 M.C. = 30.4

P.L. = 21.6 L.L. = 26.0 M.C. = 30.2

HOGGAN ENGINEERING & TESTING (1980) LTD. 17505 - 106 Avenue Edmonton, AB T5S 1E7 Phone: (780) 489-0700 Fax: (780) 489-0800	LOGGED BY: A Rahime	COMPLETION DEPTH: 14.94 m
	REVIEWED BY: R Evans	COMPLETION DATE: 18/03/15
	Fig. No: 2	Page 1 of 1

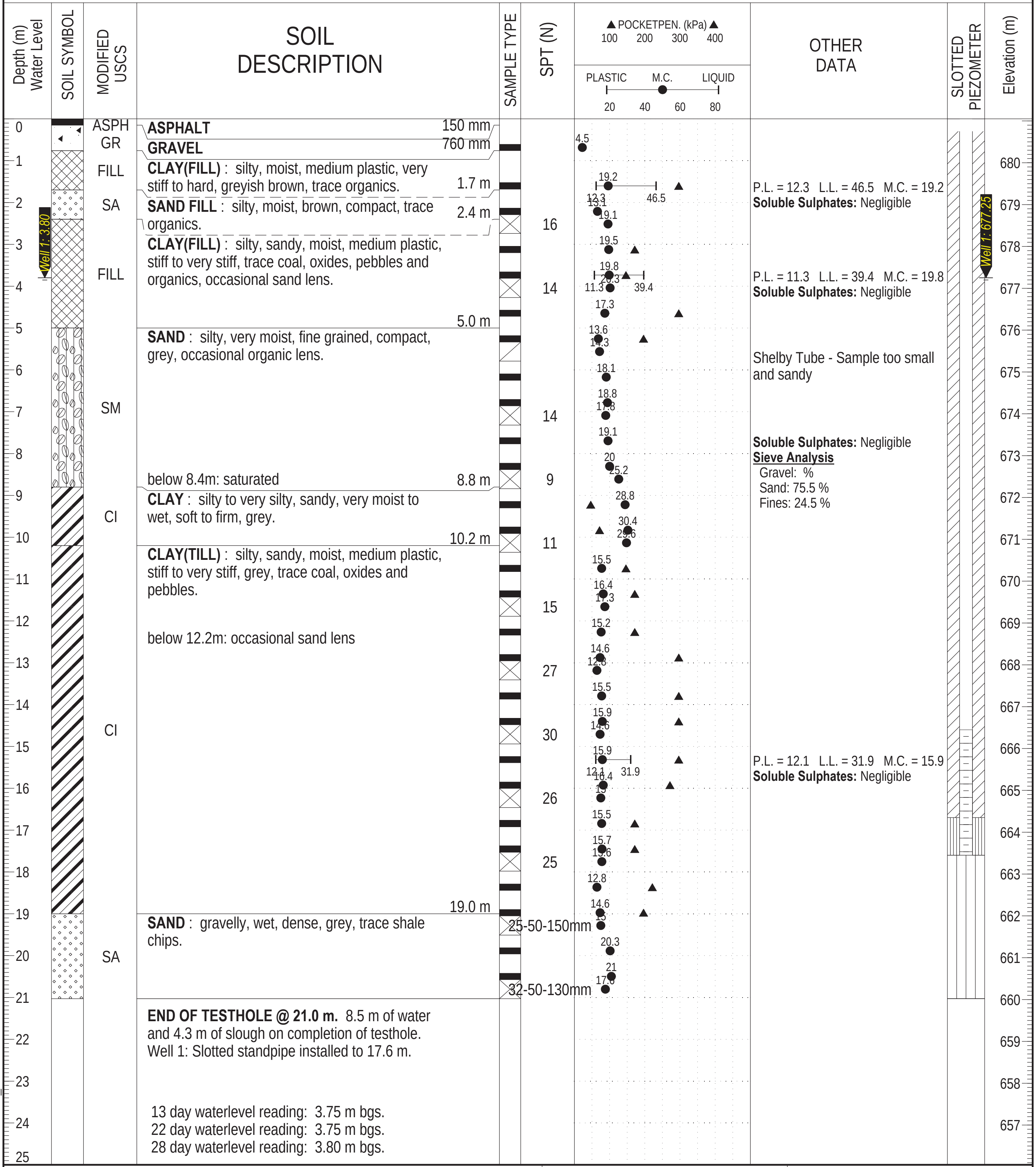
PROJECT: Proposed 199 Street Upgrades | PROJECT NO: 6004-38 | BOREHOLE NO: **2015-02**

CLIENT: Stantec Consulting Ltd | DRILL METHOD: Solid Stem Auger | ELEVATION: 681.05 m

OWNER: Qualico Communities | LOCATION: As per site plan

SAMPLE TYPE: ▨ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



ASPHALT 150 mm

GRAVEL 760 mm

CLAY(FILL) : silty, moist, medium plastic, very stiff to hard, greyish brown, trace organics. 1.7 m

SAND FILL : silty, moist, brown, compact, trace organics. 2.4 m

CLAY(FILL) : silty, sandy, moist, medium plastic, stiff to very stiff, trace coal, oxides, pebbles and organics, occasional sand lens. 5.0 m

SAND : silty, very moist, fine grained, compact, grey, occasional organic lens.

below 8.4m: saturated 8.8 m

CLAY : silty to very silty, sandy, very moist to wet, soft to firm, grey. 10.2 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

below 12.2m: occasional sand lens

19.0 m

SAND : gravelly, wet, dense, grey, trace shale chips.

END OF TESTHOLE @ 21.0 m. 8.5 m of water and 4.3 m of slough on completion of testhole. Well 1: Slotted standpipe installed to 17.6 m.

13 day waterlevel reading: 3.75 m bgs.
22 day waterlevel reading: 3.75 m bgs.
28 day waterlevel reading: 3.80 m bgs.

OTHER DATA

P.L. = 12.3 L.L. = 46.5 M.C. = 19.2
Soluble Sulphates: Negligible

P.L. = 11.3 L.L. = 39.4 M.C. = 19.8
Soluble Sulphates: Negligible

Shelby Tube - Sample too small and sandy

Soluble Sulphates: Negligible
Sieve Analysis
Gravel: %
Sand: 75.5 %
Fines: 24.5 %

P.L. = 12.1 L.L. = 31.9 M.C. = 15.9
Soluble Sulphates: Negligible

HOGGAN ENGINEERING & TESTING (1980) LTD.

17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

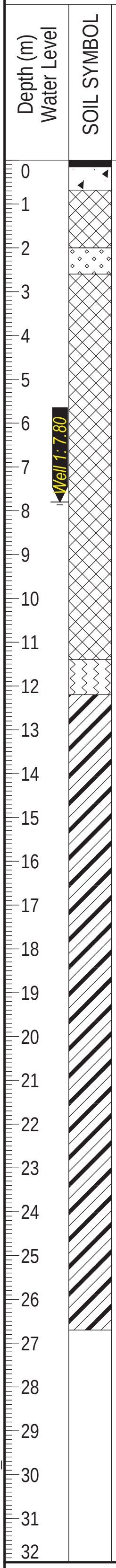
LOGGED BY: A Rahime | COMPLETION DEPTH: 21.03 m

REVIEWED BY: R Evans | COMPLETION DATE: 18/03/15

Fig. No: 3

PROJECT: Proposed 199 Street Upgrades
PROJECT NO: 6004-38
BOREHOLE NO: **2015-03**
CLIENT: Stantec Consulting Ltd
DRILL METHOD: Solid Stem Auger
ELEVATION: 680.88 m
OWNER: Qualico Communities
LOCATION: As per site plan

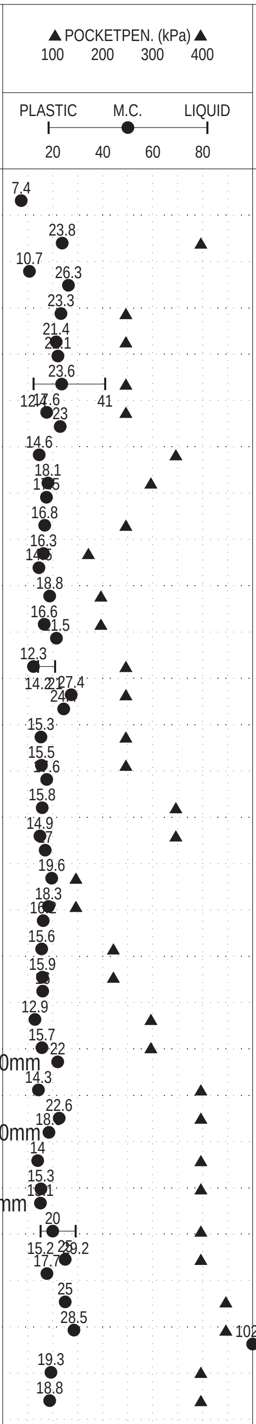
SAMPLE TYPE: ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY
BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND

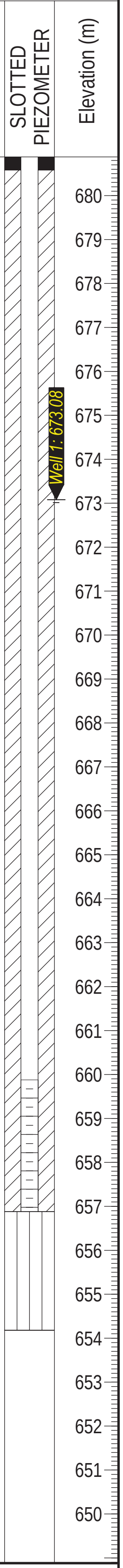


MODIFIED USCS	SOIL DESCRIPTION		SAMPLE TYPE / SPT (N)	OTHER DATA
ASPH	**ASPHALT**	150 mm		
GR	**GRAVEL**	680 mm		
FILL	**CLAY(FILL)** : silty, moist, high plastic, very stiff to hard, greyish brown, trace oxides and organics. below 1.2m: sandy	2.0 m		
SA	**SAND FILL** : silty, moist, brown, fine to medium grained, compact, trace organics.	2.6 m	15	**Soluble Sulphates:** Negligible **Sieve Analysis** Gravel: % Sand: 75.4 % Fines: 24.6 % **Shelby Tube:** QU: 201.1 kPa DD: 1654 Kg/m^3 MC: 22.1 %
FILL	**CLAY FILL** : silty, sandy, medium plastic, stiff to very stiff, grey, trace coal, oxides, pebbles and organics. below 3.4m:silty, sandy, medium plastic, grey, trace coal, oxides and pebbles. below 5.8m: very sandy, low to non plastic, very stiff, slight black staining.		13 16 25	P.L. = 12.4 L.L. = 41.0 M.C. = 23.6 **Soluble Sulphates:** Negligible
	below 9.9m: trace coal, oxides and pebbles,	11.4 m	20	P.L. = 14.2 L.L. = 21.0 M.C. = 12.3 **Soluble Sulphates:** Negligible
OR	**ORGANICS** : topsoil, peat, granular material and wood chip mixture, wet, black.	12.2 m	16	
CI	**CLAY(TILL)** : silty, sandy, moist, medium plastic, very stiff, grey, trace coal, oxides and pebbles.		20	at 13.0m: free water noted on PEN
	below 14.9m: very sandy, gravelly, wet		27	
	below 15.9m: back to clay till		10 15	
	below 19.0m: occaional wet sand lens		32-30-50-130mm 22-36-50-130mm 41-50-100mm	P.L. = 15.2 L.L. = 29.2 M.C. = 20.0 **Soluble Sulphates:** Negligible
	at 23.5m: wet coal lens		56	
	below 24.4m: trace shale chips		87	

END OF TESTHOLE @ 26.7 m. 16.3 m of water and 2.6 m of slough on completion of testhole.
Well 1: Slotted standpipe installed to 24 m.

12 day waterlevel reading: 9.21 m bgs.
21 day waterlevel reading: 7.82 m bgs.
27 day waterlevel reading: 7.80 m bgs.





JRP 6004-38.GPJ JRPV3_0.GDT 28/05/15

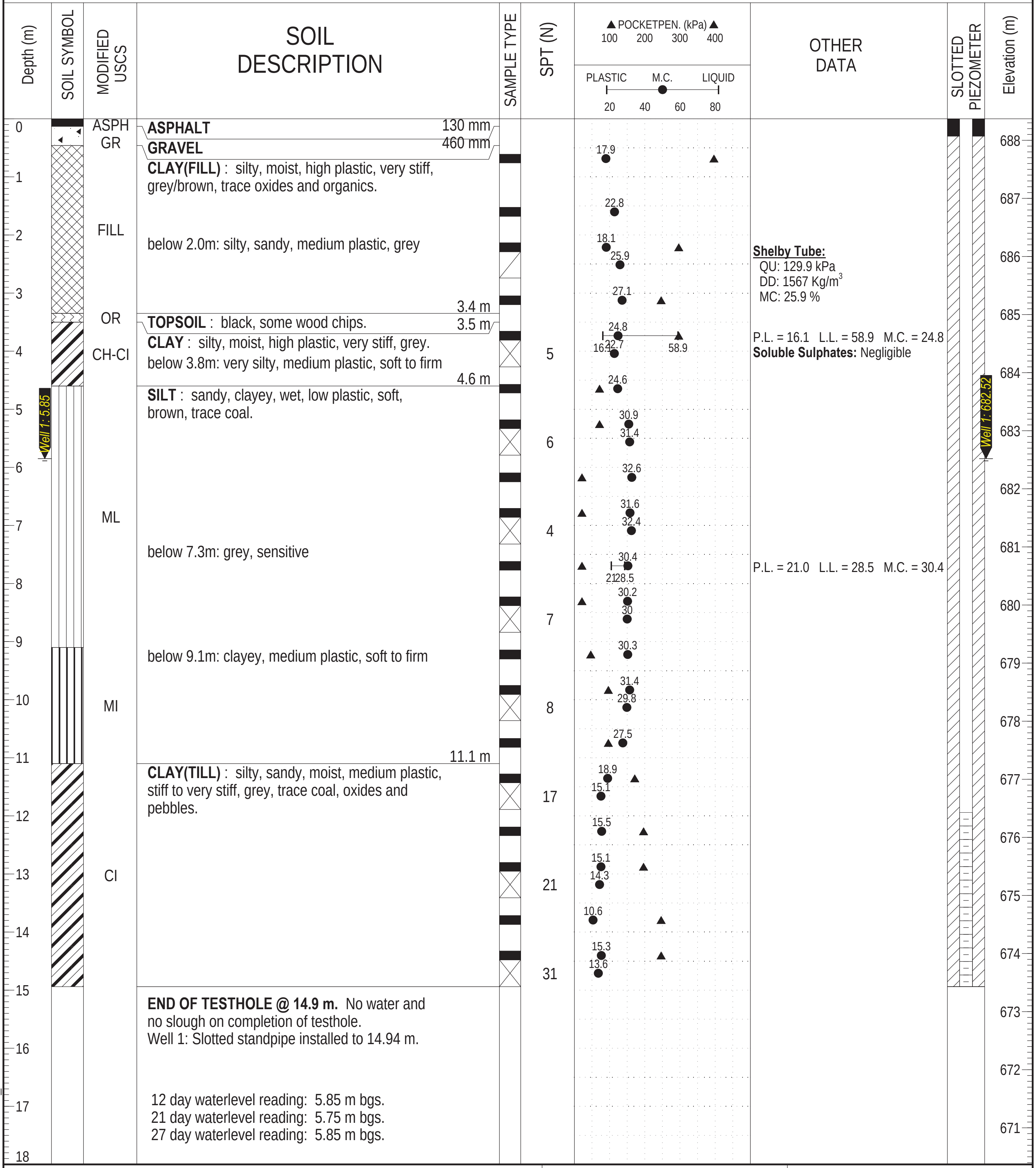
HOGGAN ENGINEERING & TESTING (1980) LTD.
17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime
REVIEWED BY: R Evans
Fig. No: 4
COMPLETION DEPTH: 26.70 m
COMPLETION DATE: 19/03/15

PROJECT: Proposed 199 Street Upgrades	PROJECT NO: 6004-38	BOREHOLE NO: **2015-04**
CLIENT: Stantec Consulting Ltd	DRILL METHOD: Solid Stem Auger	ELEVATION: 688.37 m
OWNER: Qualico Communities	LOCATION: As per site plan	

SAMPLE TYPE: ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



SOIL DESCRIPTION

ASPHALT 130 mm

GRAVEL 460 mm

CLAY(FILL) : silty, moist, high plastic, very stiff, grey/brown, trace oxides and organics.

below 2.0m: silty, sandy, medium plastic, grey

3.4 m

TOPSOIL : black, some wood chips. 3.5 m

CLAY : silty, moist, high plastic, very stiff, grey.

below 3.8m: very silty, medium plastic, soft to firm

4.6 m

SILT : sandy, clayey, wet, low plastic, soft, brown, trace coal.

below 7.3m: grey, sensitive

below 9.1m: clayey, medium plastic, soft to firm

11.1 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

END OF TESTHOLE @ 14.9 m. No water and no slough on completion of testhole.
Well 1: Slotted standpipe installed to 14.94 m.

12 day waterlevel reading: 5.85 m bgs.
21 day waterlevel reading: 5.75 m bgs.
27 day waterlevel reading: 5.85 m bgs.

OTHER DATA

Shelby Tube:
QU: 129.9 kPa
DD: 1567 Kg/m^3
MC: 25.9 %

P.L. = 16.1 L.L. = 58.9 M.C. = 24.8
Soluble Sulphates: Negligible

P.L. = 21.0 L.L. = 28.5 M.C. = 30.4

HOGGAN ENGINEERING & TESTING (1980) LTD.

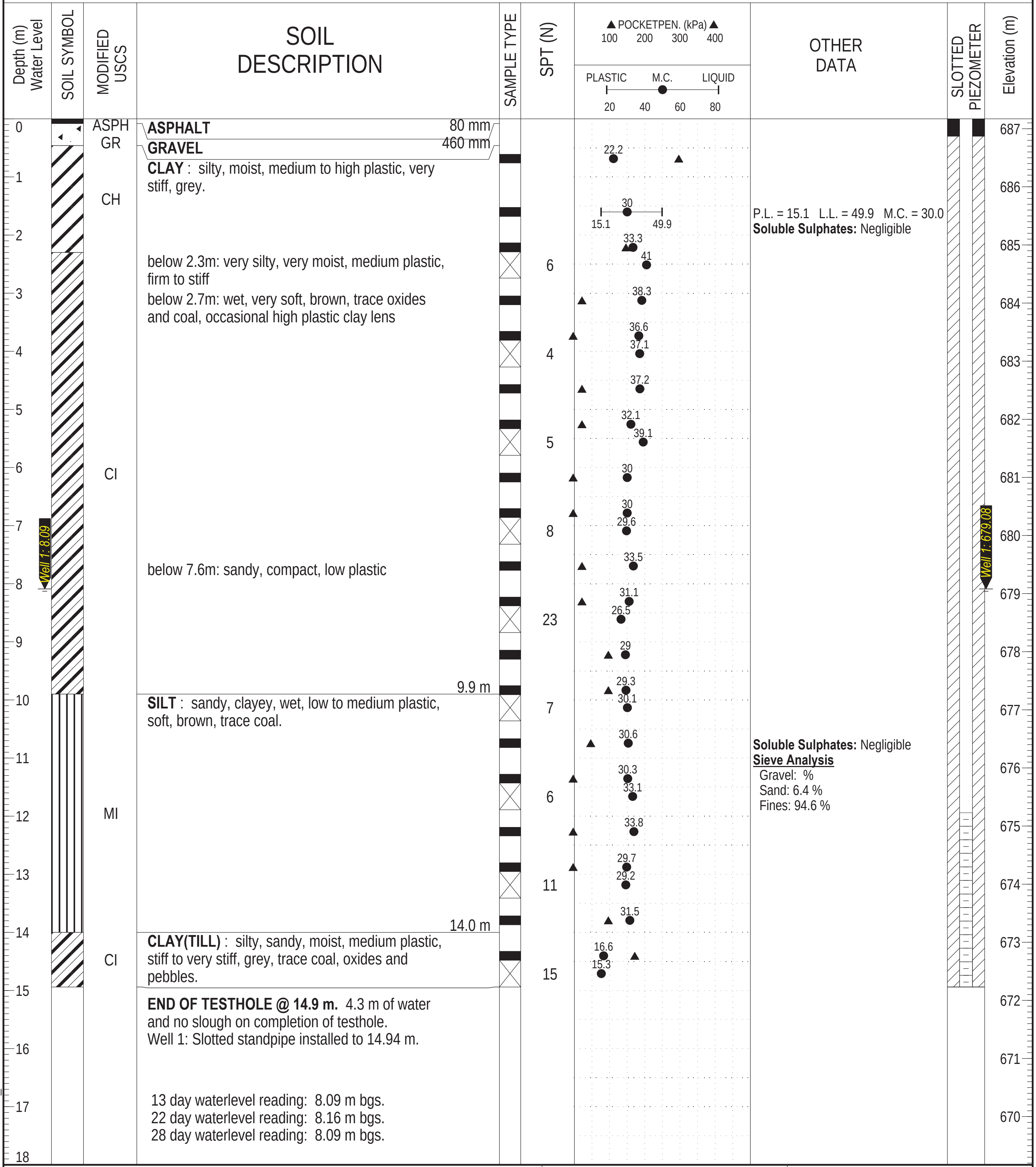
17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime	COMPLETION DEPTH: 14.94 m
REVIEWED BY: R Evans	COMPLETION DATE: 19/03/15
Fig. No: 5	Page 1 of 1

PROJECT: Proposed 199 Street Upgrades
PROJECT NO: 6004-38
BOREHOLE NO: **2015-05**
CLIENT: Stantec Consulting Ltd
DRILL METHOD: Solid Stem Auger
ELEVATION: 687.17 m
OWNER: Qualico Communities
LOCATION: As per site plan

SAMPLE TYPE ▨ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ■ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND



ASPHALT 80 mm

GRAVEL 460 mm

CLAY : silty, moist, medium to high plastic, very stiff, grey.

below 2.3m: very silty, very moist, medium plastic, firm to stiff

below 2.7m: wet, very soft, brown, trace oxides and coal, occasional high plastic clay lens

below 7.6m: sandy, compact, low plastic

9.9 m

SILT : sandy, clayey, wet, low to medium plastic, soft, brown, trace coal.

14.0 m

CLAY(TILL) : silty, sandy, moist, medium plastic, stiff to very stiff, grey, trace coal, oxides and pebbles.

END OF TESTHOLE @ 14.9 m. 4.3 m of water and no slough on completion of testhole.
Well 1: Slotted standpipe installed to 14.94 m.

13 day waterlevel reading: 8.09 m bgs.
22 day waterlevel reading: 8.16 m bgs.
28 day waterlevel reading: 8.09 m bgs.

P.L. = 15.1 L.L. = 49.9 M.C. = 30.0
Soluble Sulphates: Negligible

Soluble Sulphates: Negligible
Sieve Analysis
Gravel: %
Sand: 6.4 %
Fines: 94.6 %

HOGGAN ENGINEERING & TESTING (1980) LTD.

17505 - 106 Avenue
Edmonton, AB T5S 1E7
Phone: (780) 489-0700
Fax: (780) 489-0800

LOGGED BY: A Rahime
REVIEWED BY: R Evans
Fig. No: 6
COMPLETION DEPTH: 14.94 m
COMPLETION DATE: 18/03/15

A P P E N D I X II – Site Photos and G-Slope

Photo 1: 199th Street looking northbound from Woodbend Wynd.

Photo 2: 199th Street looking southbound from 35th Avenue/Edgemont Boulevard.

Photo 3: East side slope and Wedgewood Creek looking down from 199th Street.

Photo 4: East side slope of 199th Street.

Photo 5: Existing culvert output end. Downstream on the east side of 199th Street.

Photo 6: West sideslope of 199th Street looking southwest of Wedgewood Creek.

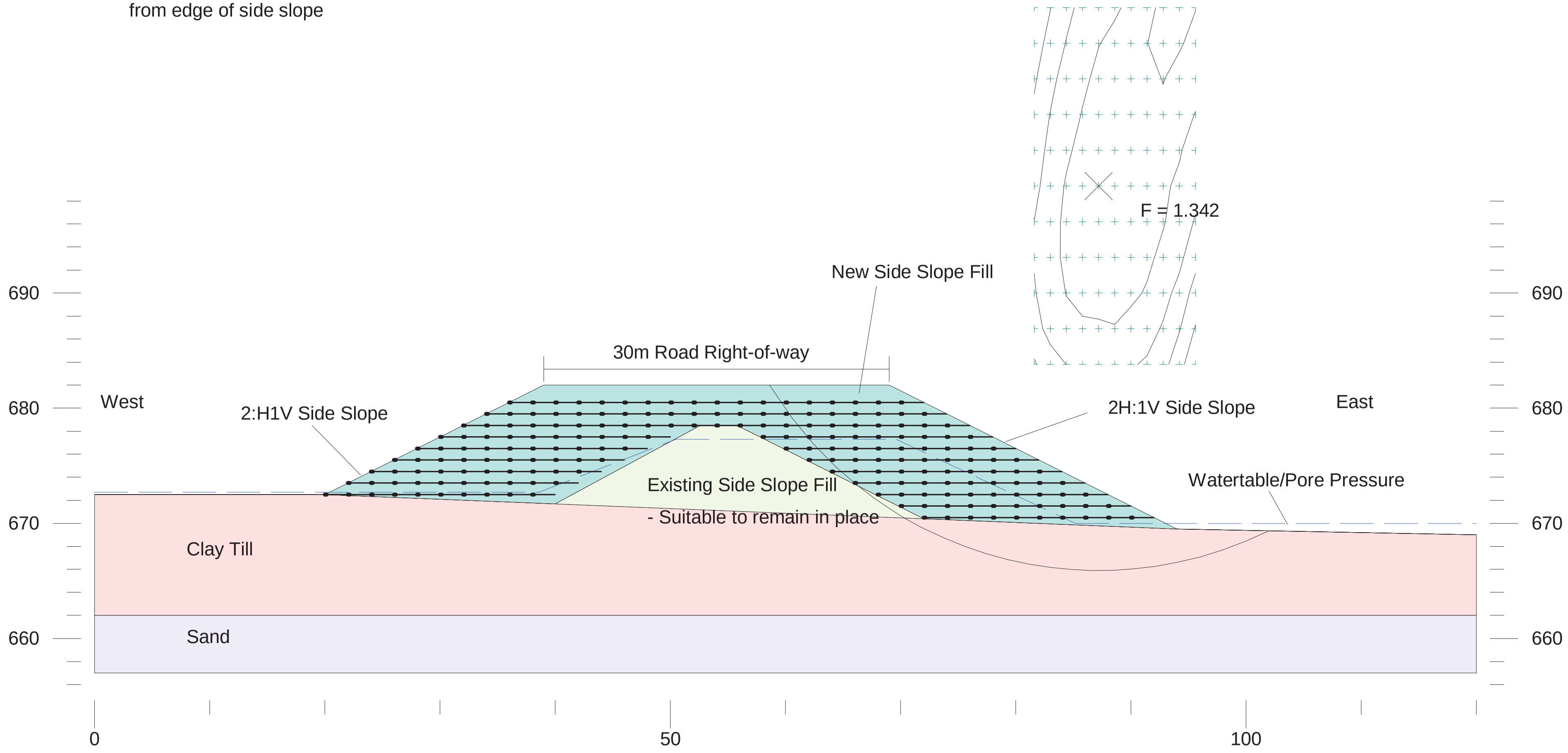
Photo 7: Storm outfall and beaver dam on west sideslope of 199th Street.

Photo 8: Close up of dam west side of 199th Street and input area of culvert.

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
🟦 New Clay Fill	19	0	25	0
🟩 Existing Clay Fill	19	2	25	1
🟥 Clay Till	19	5	28	1
🟪 Sand	20	0	30	1

Hoggan Engineering & Testing (1980) Ltd.
6004-38
Proposed 199 Street Upgrades
April 5, 2015
Wedgewood Creek Sideslopes
Reinforcement 1m spacing

NOTES:
- Top 3 meters of fill below subgrade, reinforcement shall be placed across extent of fill
- Below 3 meters of fill from subgrade, reinforcement shall be a minimum 20 meters long from edge of side slope

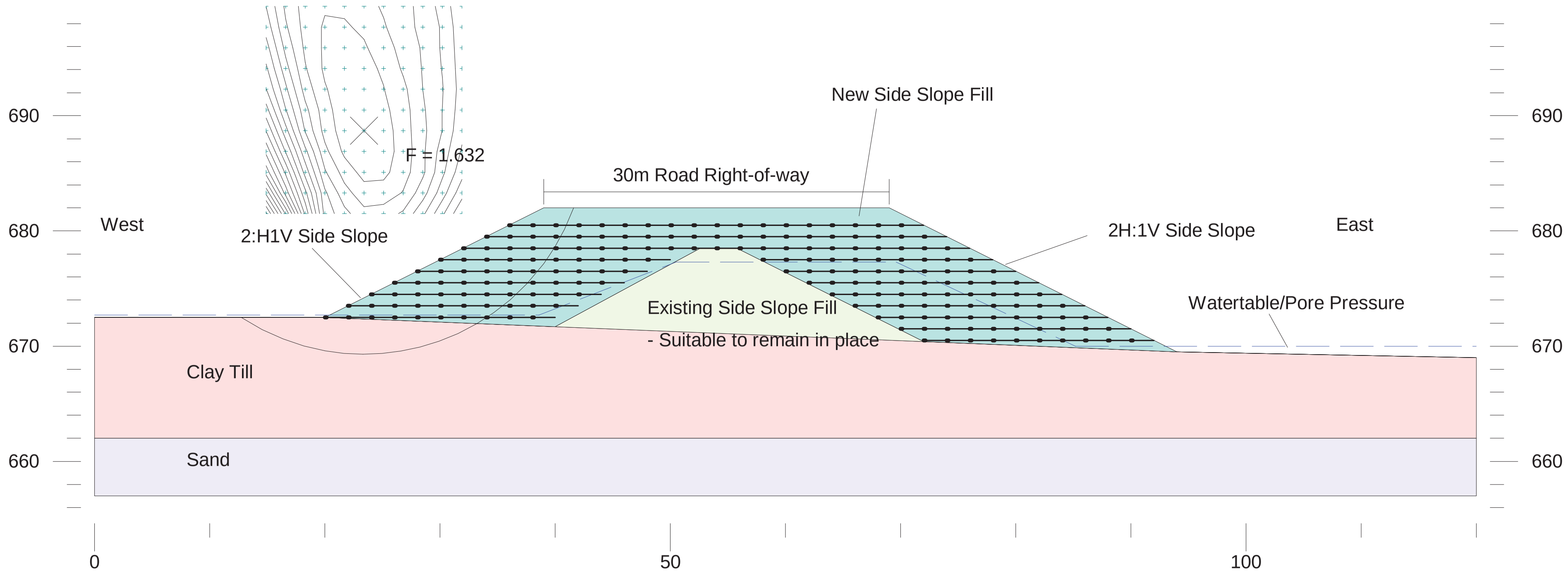


4/22/2015 1:51:57 PM H:\DATA 2015\6004 Stantec Consulting Ltd\6004-38 Proposed 199 Street Upgrade\Gslope\Both Fills Final(East).gsl Hoggan Engineering & Testing (1980) Ltd. F = 1.342

	Gamma kN/m3	C kPa	Phi deg	Piezo Surf.
New Clay Fill	19	0	25	0
Existing Clay Fill	19	2	25	1
Clay Till	19	5	28	1
Sand	20	0	30	1

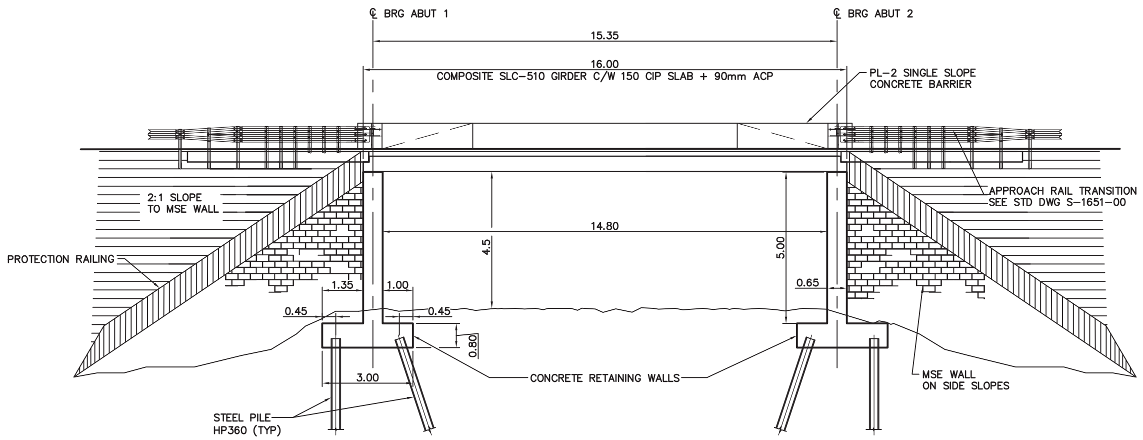
Hoggan Engineering & Testing (1980) Ltd.
6004-38
Proposed 199 Street Upgrades
April 5, 2015
Wedgewood Creek Sideslopes
Reinforcement 1m spacing

NOTES:
- Top 3 meters of fill below subgrade, reinforcement shall be placed across extent of fill
- Below 3 meters of fill from subgrade, reinforcement shall be a minimum 20 meters long from edge of side slope

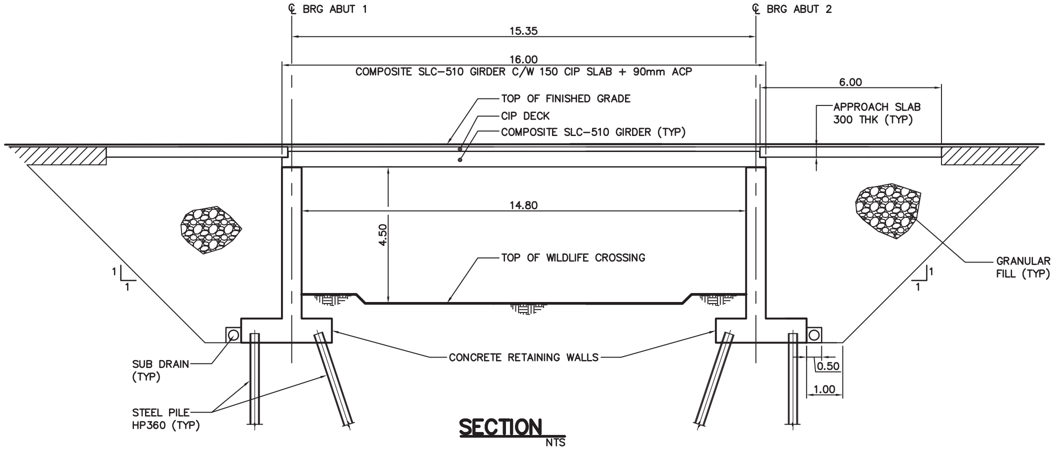


4/22/2015 1:50:00 PM H:\DATA 2015\6004 Stantec Consulting Ltd\6004-38 Proposed 199 Street Upgrade\Gslope\Both Fills Final(west).gsl Hoggan Engineering & Testing (1980) Ltd. F = 1.632

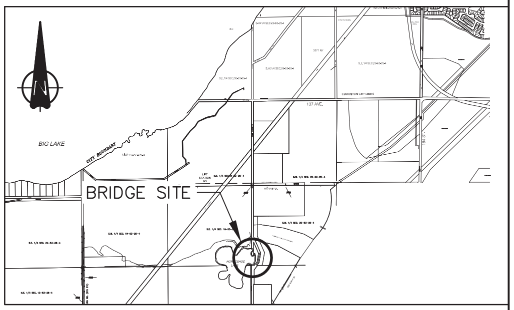
APPENDIX III – Preliminary Bridge Design



SIDE ELEVATION
NTS



SECTION
NTS



SITE PLAN
NTS

ORIGINAL SHEET - ANSI A

Month, Year
04,2015

W:\temporary\199 Street Wildlife Crossing\199 Street.dwg
2015/04/01 2:28 PM By: Pokhrel, Ashish

Stantec

10160 - 112 Street
Edmonton AB Canada
www.stantec.com

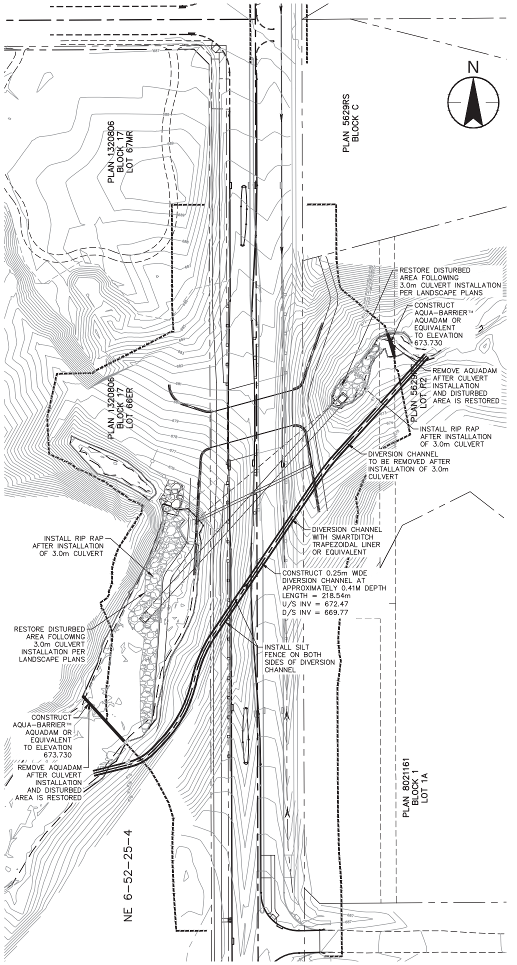
Client/Project
RIVERVIEW OWNERS GROUP LTD.
199 STREET
WEDGEWOOD RAVINE

Figure No.
1.0

Title
WILDLIFE CROSSING
CROSS SECTIONS

Appendix B

DRAWINGS



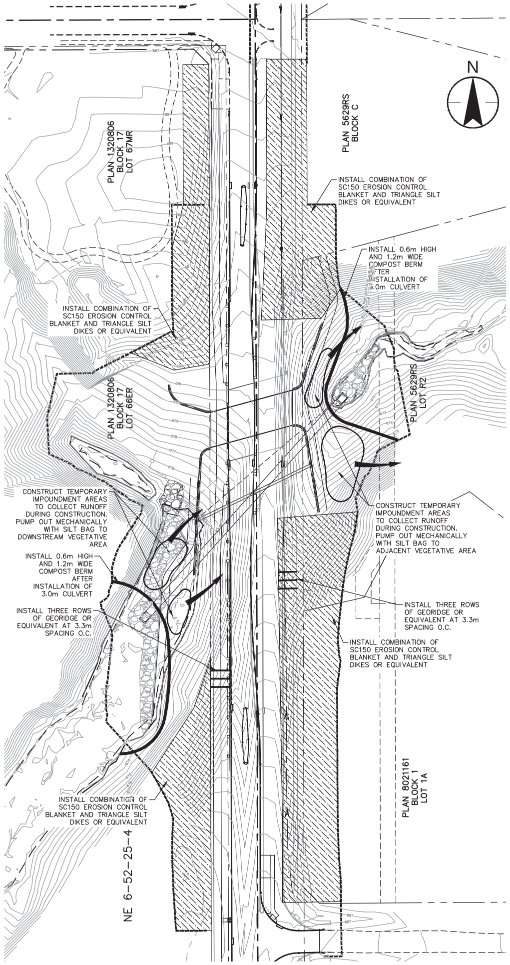
STEP 1

TENTATIVE CONSTRUCTION PROCEDURE

- INSTALL AQUA-BARRIER DAM UPSTREAM AND DOWNSTREAM TO DRY CONSTRUCTION AREA AND PREVENT SEDIMENT RELEASE.
- START REMOVING ROAD STRUCTURE AND EMBANKMENT UNTIL A CREEK DIVERSION CHANNEL CAN BE CONSTRUCTED.
- EXCAVATE WATER DIVERSION CHANNEL TO CONNECT EXISTING UPSTREAM AND DOWNSTREAM OF WEDGEWOOD CREEK USING SMARTDITCH OR EQUIVALENT PRODUCT. INSTALL SILT FENCE ON BOTH SIDES OF DIVERSION CHANNEL.
- REMOVE REMAINDER OF EXISTING ROAD STRUCTURE, EMBANKMENT, AND EXISTING CULVERT.
- INSTALL PROPOSED 3000mm DIAMETER CULVERT.
- RESTORE DISTURBED AREAS AS PER LANDSCAPE DETAILS.
- REMOVE AQUA-BARRIER DAM AND ALLOW WEDGEWOOD CREEK FLOW THROUGH THE PROPOSED 3000mm DIAMETER CULVERT.

SUMMARY OF INTERIM ESC MEASURES

- CONCENTRATED FLOW IS EXPECTED ALONG THE EXISTING SLOPES DRAINING INTO THE WEDGEWOOD CREEK RAVINE TO THE NORTH AND SOUTH OF THE PURPOSED CROSSING. DITCHES WILL BE LEFT VEGETATED FOR EROSION CONTROL. DISTURBED AREAS WILL BE BOLSTERED WITH A COMBINATION OF SC150BN EROSION CONTROL BLANKETS AND "GEORIDGE" TRIANGLE SILT DIKES IN AREAS OF EROSION POTENTIAL.
- SMARTDITCH OR SIMILAR MATERIAL WILL LINE THE BOTTOM OF THE TEMPORARY CHANNEL CREATING A SMOOTH CAUSEWAY TO REDUCE EROSION POTENTIAL.
- SILT FENCE WILL BE INSTALLED ALONG THE SMARTDITCH TO RESTRICT SEDIMENT ENTRY FROM UPSTREAM SOURCE THAT MAY ENTER THE TEMPORARY DIVERSION CHANNEL
- RESTORATION MEASURES WILL BE FOLLOWED AS SPECIFIED ON THE LANDSCAPING PLANS.
- IN THE EVENT OF RAINFALL DURING CONSTRUCTION, STANDING WATER WITHIN THE ISOLATED CONSTRUCTION ZONE WILL BE PUMPED MECHANICALLY THROUGH A SILT BAG TO ADJACENT VEGETATED AREA EAST OF THE CROSSING.



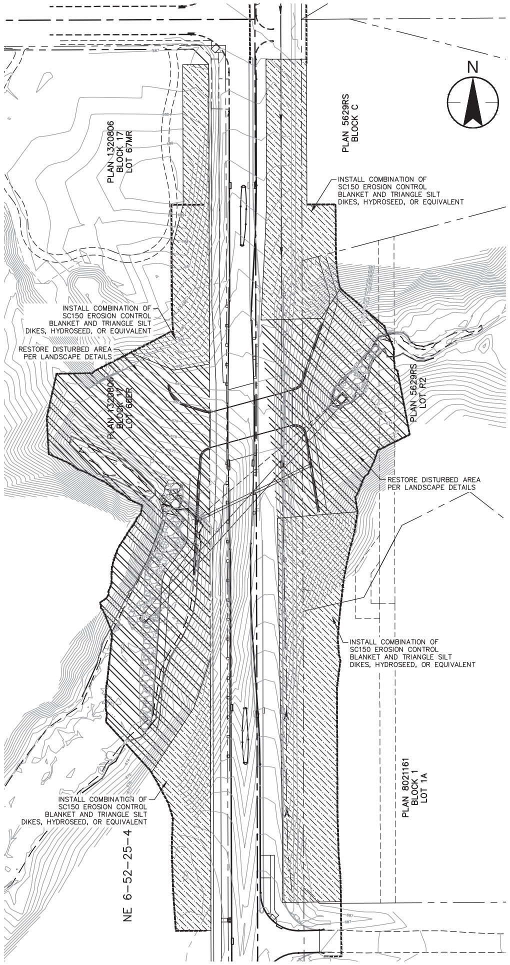
STEP 2

TENTATIVE CONSTRUCTION PROCEDURE

- CONSTRUCT 0.6m HIGH COMPOST BERMS NEAR PROPOSED 3000mm CULVERT INLET AND OUTLET TO RETAIN SEDIMENT AND ISOLATE CONSTRUCTION ZONE FROM WEDGEWOOD CREEK.
- CONSTRUCT TEMPORARY IMPOUNDMENT AREAS ADJACENT TO THE COMPOST BERM FOR INCREASED SEDIMENT RETENTION.
- BEGIN ROADWAY AND EMBANKMENT GRADING IN ADDITION TO PILE CONSTRUCTION FOR THE WILDLIFE CROSSING STRUCTURE.
- BEGIN UNDERGROUND DEEP AND SHALLOW UTILITY INSTALLATION.

SUMMARY OF INTERIM ESC MEASURES

- COMPOST BERMS WILL BE INSTALLED ABOVE THE INLET AND OUTLET OF THE CULVERT SERVICING WEDGEWOOD CREEK. ANY SURFACE RUNOFF FROM WITHIN THE CONSTRUCTION ZONE WILL BE DIRECTED TO THE BERMS BY THE NATURAL TOPOGRAPHY.
- IMPOUNDMENT AREAS WILL EXCAVATED ADJACENT TO THE COMPOST BERMS WHICH WILL ACT AS AN IMPOUNDMENT AND SEDIMENTATION RETENTION AREA. WATER WILL BE PUMPED MECHANICALLY THROUGH A SILT BAG TO AN EXISTING VEGETATED AREA EAST OF THE CROSSING TO RETURN TO WEDGEWOOD CREEK.
- DISTURBED AREAS WILL BE GRADED ROUGH AND WHERE POSSIBLE SC150BN EROSION CONTROL BLANKETS WILL BE PLACED TO PROTECT WEDGEWOOD CREEK FROM UPSTREAM SLOPE EROSION AND SEDIMENT RELEASE.



STEP 3

TENTATIVE CONSTRUCTION PROCEDURE

- REHABILITATE DISTURBED AREAS AS PER LANDSCAPE DETAILS.
- REHABILITATE AND SEED ROADWAY EMBANKMENT SIDESLOPES AND OTHER AREAS IF REQUIRED.

SUMMARY OF INTERIM ESC MEASURES

- EXTENSIVE SEEDING AS DETAILED IN THE LANDSCAPE PLANS FOLLOWING FINAL GRADING WILL TAKE PLACE ALONG THE SIDE SLOPES OF THE EMBANKMENT. SC150BN EROSION CONTROL BLANKET WILL BE LAID OVER THE SEEDED AREAS TO PROTECT WASH OFF AND ALLOW SEEDS TO TAKE HOLD.
- COMPOST BERM WILL BE IN PLACE DURING CONSTRUCTION AND FINAL GRADING TO PROTECT WEDGEWOOD CREEK. THESE WILL BE REMOVED ONCE SEEDING AND EROSION CONTROL BLANKETS HAVE BEEN INSTALLED.

Stantec Consulting Ltd.
10160 - 112 Street
Edmonton AB Canada
Tel. 780.917.7000
www.stantec.com

Copyright Reserved

The Contractor shall verify and be responsible for all dimensions. DO NOT scale the drawing - any errors or omissions shall be reported to Stantec without delay. The Copyrights to all designs and drawings are the property of Stantec. Reproduction or use for any purpose other than that authorized by Stantec is forbidden.

Legend

➞ SPILL DIRECTION
← OVERLAND FLOW
⬅ DIRECTION OF MAJOR DRAINAGE
CATCHBASIN WITH SEDIMENT BARRIER
✱ INTERIM STORM WATER DISCHARGE LOCATION
SILT FENCE
STRAW WATTLE
▨ EROSION BLANKET
LIMIT OF CONSTRUCTION
DEVELOPMENT BOUNDARY

Notes

1. EROSION AND SEDIMENT CONTROL MEASURES SHALL BE ADOPTED DURING CONSTRUCTION IN ACCORDANCE WITH THE CITY OF EDMONTON EROSION AND SEDIMENTATION CONTROL GUIDELINE AND FIELD MANUAL.

Development Engineer, Sustainable Development

Approvals			YY.MM
Revision	By	Appd.	YY.MM
1. FIRST SUBMISSION	G.R.	T.C.	15.12
Issued	By	Appd.	YY.MM

PROJECT MANAGER: T. CHIARELLO
DESIGN: G.R.
DRAWN: G.R.

Permit-Seal

Client/Project
RIVERVIEW HEIGHTS ESTATES LTD.

199 STREET
23 AVENUE NW TO 35 AVENUE NW
Edmonton, AB

Title
EROSION AND SEDIMENT CONTROL

Project No.
1161-103725

Scale
1:1000
0 10 20 30 40 50

V:\1161\active\1161103725\drawing\sheet_files\1161103725 - C015 - ESC.dwg
2015-12-04 8:34am BY: BIWONG

Drawing No.
C015-003

Appendix C

UNIVERSAL SOIL LOSS EQUATION

Summary of Sediment Delivery Using RUSLE 2.0

Project: 199 STREET - Wedgewood Creek Crossing

Overland flow path*		**Segment Length (m)**	**Cover:** none **Segment Slope (%)**	**Soil:** Silty clay (no OM)	**Practise:**	**Sediment Delivery: tonnes/ha/yr**
Slope 1		120.0	4.5		Erosion Control Blanket with Triangle Dikes and Silt Fence	0.42
Slope 2		195.0	9.0		Erosion Control blanket	1.2
Slope 3 (Ditch)		170.0	4.5		Erosion Control Blanket with Triangle Dikes and Silt Fence	0.58
Slope 4 (Ditch)		110.0	8.0		Erosion Control Blanket	1.15
Total Annual sediment delivery average loss per ha/yr:						0.8
*Slopes have been determined based on the steps throughout construction						

Appendix D

INSPECTION SCHEDULE

ESC INSPECTION/ MAINTENANCE REPORT

Project Name:	**File #:**
Inspection Date/Time:	Date of Last Inspection:
Inspected By:	
Verbal/Written Notification given to:	Date:
Current Weather:	Weather Forecast
Amount of Rain last week (mm):	Amount of Rain in last 24hrs (mm):
Stage of Construction:	
Contractors on Site:	Heavy Equipment on Site:
Construction Activities on Site:	

INSPECTION CHECKLIST	YES	NO	COMMENTS	ACTION REQUIRED
Has stripping and grading been phased where possible?				
Have stripped areas/exposed soils/ steep slopes been **protected**?				
Have stripped areas/exposed soils/ steep slopes and **stabilized**?				
Have waterways and drainage ways been **protected**?				
Have waterways and drainage ways been **stabilized**?				
Are perimeter controls in place and functioning adequately?				
Are offsite/ downstream properties/ waterways protected?				
Are Sedimentation control BMP's in place and functioning adequately?				
Are transport control BMP's in place and functioning adequately?				
Are Erosion control BMP's in place and functioning adequately?				

Wedgewood Creek Road Crossing 199 Street: Fish Habitat Assessment 05-52-25-W4M

Stantec

Prepared for:
Riverview Owners Group

Prepared by:
Stantec Consulting Ltd

December 1, 2014

Stantec Quality Management Program

This document entitled Wedgewood Creek Road Crossing 199 Street: Fish Habitat Assessment 05-52-25-W4M was prepared by Stantec Consulting Ltd. (“Stantec”) for the account of Riverview Owners Group (the “Client”).

Prepared by ____
(signature)

Shona Derlukewich, Dipl. EPt
Fisheries Biologist

K Cronk

Reviewed by ____
(signature)

Kelsey Cronk, B.Sc.
Fisheries Biologist

Reviewed by ____
(signature)

Paul Harper, B.Sc.
Senior Fisheries Biologist

Table of Contents

LIST OF TABLES

LIST OF APPENDICES

Abbreviations

CAPP	Canadian Association of Petroleum Producers
COP	Codes of Practice (ESRD)
CRA	Commercial, Recreational, or Aboriginal fisheries
DFO	Fisheries and Oceans Canada
ESRD	Alberta Environment and Sustainable Resource Development
FWMIS	Fisheries & Wildlife Management Information System
QAES	Qualified Aquatic Environmental Specialist
RAP	Restricted Activity Period

1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The Riverview Owners Group (the Client) has proposed development for Riverview Neighbourhoods 1, 2 and 3. This Project includes the widening of a section on 199 Street and installation of a wildlife passage at the Wedgewood Creek crossing location (Appendix A, Figure 1). The culvert to be replaced is a 1.8 m structural plate culvert with a 62.8 m length; it was built in 1952 and modified in 1968 (Terrace 2014).

1.2 REGULATORY CONTEXT

1.2.1 Fisheries Act

The *Fisheries Act* applies to all projects that have the potential to cause serious harm to commercial, recreational, or aboriginal (CRA) fisheries. Fisheries and Oceans Canada (DFO) has developed a self-assessment process (DFO 2014a) that sets out exclusion criteria a project can meet and Measures to Avoid Causing Harm to Fish and Fish Habitat (DFO 2014b). If the Project does not meet the established criteria, a review by DFO will be required to determine if the project has the potential to cause serious harm to CRA fisheries. If DFO determines that serious harm is expected, an Authorization under paragraph 35(2)(b) of the *Fisheries Act* will be required.

1.2.2 Alberta *Water Act*

The Alberta *Water Act* applies to all work undertaken in and around Alberta water bodies that have the potential to affect the aquatic environment. The *Code of Practice for Watercourse Crossings* (COP) establishes the objectives, standards, and conditions to be met when undertaking the activity of constructing or removing watercourse crossing.

If the Project can meet the requirements outlined in the COP, the proponent can proceed with the Project without the requirement of obtaining separate approval under the Alberta *Water Act* (ESRD 2013).

1.3 OBJECTIVES

The purpose of the fish and fish habitat assessment is to characterize fish species presence and available fish habitat near the Project.

2.0 METHOD

Baseline data on general fish presence and fish near the Project were collected through a desktop review of existing information and through a field survey.

2.1 DESKTOP ASSESSMENT

2.1.1 Restricted Activity Period

A review of the Alberta Environment and Sustainable Development's (ESRD) Code of Practice for Watercourse Crossings (COP) - St. Paul Management Area Map (ESRD 2012a) was conducted.

2.1.2 Fish Presence

A review of existing fish and fish habitat information for Wedgewood Creek was conducted. The review included a search of published and unpublished reports, maps, file data, and aerial photographs available from government files and consultant libraries. A review of ESRD's Fisheries & Wildlife Management Information System (FWMIS) Internet Mapping Framework (ESRD 2014) provided previously recorded fish presence in Wedgewood Creek within the Project area.

2.1.3 Species of Management Concern

For the Project, species of management concern (SOMC) includes species protected by federal and provincial legislation, including the *Species at Risk Act* (SARA) Schedule 1 listed species (GOC 2014) and the Alberta *Wildlife Act* (1997). Species with designations and status reports were also considered, including General Status of Alberta Wild Species (ESRD 2011) and the Committee on the Status of Endangered Wildlife in Canada (COSEWIC) (GOC 2014).

2.2 FISH HABITAT ASSESSMENT

The fish and fish habitat assessment was conducted near the Project extending from approximately 100 m upstream to 300 m downstream as per the guidelines in the Alberta Transportation Fish Habitat Manual (AT 2009). Field information and observations were recorded and included the following, where applicable:

- Channel characteristics (e.g. wetted and channel widths);
- In-situ water chemistry (i.e., pH, temperature (°C), conductivity (µs/cm), dissolved oxygen (mg/L), and turbidity (NTU));
- Barriers, obstructions, and debris (e.g., log jams, beaver dams, man-made barriers, etc.);
- Habitat type (e.g., pool, riffle, and run) (Appendix A);
- Bed material (% substrate size distribution);

- Vegetation (instream and riparian);
- Degree of stream channel confinement;
- High water mark, flood signs;
- Stage of the river (low, moderate, high); and
- Digital photographs.

2.3 FISH PRESENCE

Prior to the fish inventory being conducted, a Fish Research Licence (FRL) was obtained from Alberta Environment and Sustainable Resource Development (ESRD). FRL 14-3852 was issued on October 3, 2014. All data collected under the authority of FRL 14-3852 was submitted to ESRD for entry into the Alberta Fish and Wildlife Management Information System (FWMIS) database.

Backpack electrofishing was utilized as the only fish sampling method at this site. A Smith-Root LR-24 electrofisher powered by a 24 V battery with duty cycle (%), frequency (Hz), voltage (v), and time(s) recorded sampling event information. Electrofishing is a non-lethal and non-exclusive method for capturing fish as per the sampling protocol "Electrofishing Policy Respecting Injuries to Fish" (ESRD 2012b).

Captured fish were placed in an aerated holding tank until they were processed. Fish were measured for length, weight, and qualitatively evaluated for health and spawning condition. Fish that avoided capture, but were identified with confidence were enumerated and recorded as observed (Appendix B).

3.0 RESULTS

3.1 DESKTOP ASSESSMENT

3.1.1 Restricted Activity Period

Wedgewood Creek originates southwest of Edmonton and flows through agriculture lands discharging into the North Saskatchewan River. This creek is impacted extensively by beaver activity creating water impoundments throughout the Project area.

RAP's are set to protect sensitive life stages of fish that may be present in the watercourses (ESRD 2012a). RAPs are important for protecting a fishery where there is uncertainty about the conditions at the site, the fish that might be present at the work location, or the potential impacts of the work. Under the advice of a QAES, works may occur within the RAP if potential impacts to the aquatic environment are mitigated.

Wedgewood Creek is an unmapped water body that enters into the North Saskatchewan River, which is a mapped Class C watercourse. Wedgewood Creek assumes the Class C designation of the North Saskatchewan River and therefore has a RAP from September 16 to July 31 (ESRD 2012a).

3.1.2 Fish Presence

A FWMIS search was conducted on October 2, 2014. Historical records show that two forage fish species have been recorded within the Project area (ESRD 2014) (Table 3-1). The FWMIS search included 5 km search area of Wedgewood Creek at the proposed project location.

Table 3-1 Fish species present in Wedgewood Creek 1 km upstream of the Project area and 4 km downstream to the North Saskatchewan River.

Species		**Conservation Status**			
Scientific Name	**Common Name**	**Alberta Wild Species Rank**[1]	**Alberta *Wildlife Act***[2]	**COSEWIC**[3]	**SARA**[3]
Forage Fish					
Culaea inconstans	brook stickleback	*Secure*	N/A	N/A	N/A
Pimpephales promelus	fathead minnow	*Secure*	N/A	N/A	N/A

Notes:

[1] ESRD (2011)
[2] Wildlife Act Wildlife Regulation (1997)
[3] Government of Canada (2014)

3.1.3 Species of Management Concern

No fish species known to occur in the Project area are provincially or federally listed (ESRD 2011, Wildlife Act [1997]; GOC 2014).

3.2 FISH HABITAT ASSESSMENT

The fish habitat assessment on Wedgewood Creek was conducted by qualified aquatic environmental specialists (QAES) on October 8, 2014. At the time of the visits, low flow conditions were present and weather conditions were favourable (i.e., clear skies, no precipitation).

Overall, the fish habitat in Wedgewood Creek was rated as "moderate" based on habitat characteristics and the fish species known to occur in the area (Appendix A, Figure 2).

The existing crossing is a 1.8 m diameter culvert and 62.8 m in length (Terrace 2014). Water depth in the culvert was shallow and the water velocity was slow. It appears that small-bodied fish could swim through the culvert with minimal woody debris, aquatic vegetation and fines observed (Appendix B, Photo 4). The culvert does not appear to be a fish barrier at the outlet; however, beaver dam may impede passage 2 m upstream of the inlet. High flows in the spring could be constricted in the culvert and the velocity could impede upstream migration. Low flows in the late fall could reduce water depths in the culvert, making it impassable.

Forage fish habitat in assessed reaches of the creek is rated as "good" with suitable areas of aquatic vegetation providing spawning habitats for the fish species known to occur in the area. Woody debris, water depth, and aquatic vegetation provide good cover and habitat for rearing and overwintering. The availability of overwintering habitat is present in beaver impounded water typically > 1.2 m deep.

Coarse and sport fish habitat is rated as "poor" with limited areas of spawning substrate such as gravels, cobbles, and aquatic vegetation. Lower oxygen levels downstream of the crossing in beaver impoundments do not provide suitable rearing or overwintering habitat.

Barriers to fish passage were observed along the creek and migration potential was considered "poor" for all fish species. Beaver dams were present throughout the Project area (Appendix A, Figure 2) including the existing crossing (Appendix B, Photo 6) immediately upstream of the culvert. The beaver dams provide temporary barriers and create deep pool habitats compared to a free flowing creek. This allows habitat for selected forage fish species but will not be suitable for coarse or sportfish species.

3.3 FISH PRESENCE

During the survey, backpack electrofishing was conducted for a total of 304 seconds, within a 60 m long section between Transects 1 and 3. Brook stickleback, and finescale dace (Appendix B, Photos 3) were the only species captured during this event (Table 3-2).

Table 3-2 Fish Species Captured in Wedgewood Creek, October 8, 2014

Gear	Fish Sampling Data		
	Species	Count	Size Range (mm)
Backpack Electofisher	brook stickleback	5	45 - 57
	finescale dace	1	76

NOTE:

See Appendix B for detailed fish capture results.

4.0 SUMMARY

Fish habitat in Wedgewood Creek is rated as moderate for forage fish species. The Project area has sections of beaver impounded pools that support rearing, spawning and overwintering of forage fish that tolerant of low oxygen levels. The large and extensive beaver dams create habitat, but also limit fish migration in the creek. The habitat consisted of fines substrate, deep pools, aquatic vegetation, and overhanging vegetation.

5.0 CONCLUSION

This report documents the fish and fish habitat near the 199 Street crossing on Wedgewood Creek to support regulatory requirements. Recommendations outlined in this report have been provided at the design stage of the project. Additional QAES recommendations may be required once the construction schedule and instream work requirements have been confirmed.

6.0 REFERENCES

Alberta Environment and Sustainable Resource Development (ESRD). 2001. *Guide to the code of practice for watercourse crossings, including guidelines for complying with the code.* Accessed November 2, 2014. Available at: <http://esrd.alberta.ca/water/legislation-guidelines/documents/WatercourseGuide.pdf >

Alberta Environment Sustainable Resource Development (ESRD). 2011. *Alberta Wild Species General Status Listing – 2010.* Fish and Wildlife Division, Sustainable Resource Development.

Alberta Environment and Sustainable Resource Development (ESRD). 2012a. *Code of Practice – St. Paul Management Area Map. November 2012*. Prepared by Resource Information Management Branch, Corporate Services Division. Government of Alberta.

Alberta Environment and Sustainable Resource Development (ESRD). 2012b. *Electrofishing Policy Respecting Injuries to Fish*. Available at: <http://esrd.alberta.ca/fish-wildlife/fish-research-licence/documents/ElectrofishingPolicy-InjuriestoFish-Nov2012A.pdf>

Alberta Environment and Sustainable Resource Development (ESRD). 2013. *Code of Practice for Watercourse Crossings made under the Water Act and the Water (Ministerial) Regulation. Consolidated to include amendments in force as of June 24, 2013.* Accessed November 2, 2014. Available at: <http://www.qp.alberta.ca/documents/codes/CROSSING.pdf>

Alberta Environment and Sustainable Resource Development (ESRD). 2014. *Fish and Wildlife Management Information System (FWMIS) Internet Mapping Guide*. Accessed on October 2, 2014 available at: <https://maps.srd.alberta.ca/FWIMT_Pub/default.aspx?Viewer=FWIMT_Pub>

Alberta Transportation (AT) 2009. *Fish Habitat Manual: Guidelines & Procedures for Watercourse Crossings in Alberta*. Government of Alberta. Edmonton, Alberta. Revised August 2009. Available at: <http://www.transportation.alberta.ca/Content/docType245/Production/Complete_Fish_Habitiat_Manual.pdf>

Fisheries and Oceans Canada (DFO). 2014a. *Projects near Water. Self-Assessment: Does DFO need to review my project?* Available at: <http://www.dfo-mpo.gc.ca/pnw-ppe/index-eng.html>

Fisheries and Oceans Canada (DFO). 2014b. *Measures to Avoid Causing Harm to Fish and Fish Habitat*. Available at: <http://www.dfo-mpo.gc.ca/pnw-ppe/measures-mesures/index-eng.html>

Government of Canada (GOC). 2014. *The Species at Risk Public Registry*. Accessed on November 3, 2014 and available at: <http://www.sararegistry.gc.ca/sar/index/default_e.cfm?stype=species&lng=e&index=1&cosid=&common=&scientific=&population=&taxid=3&locid=0&desid=0&schid=0&desid2=0&>

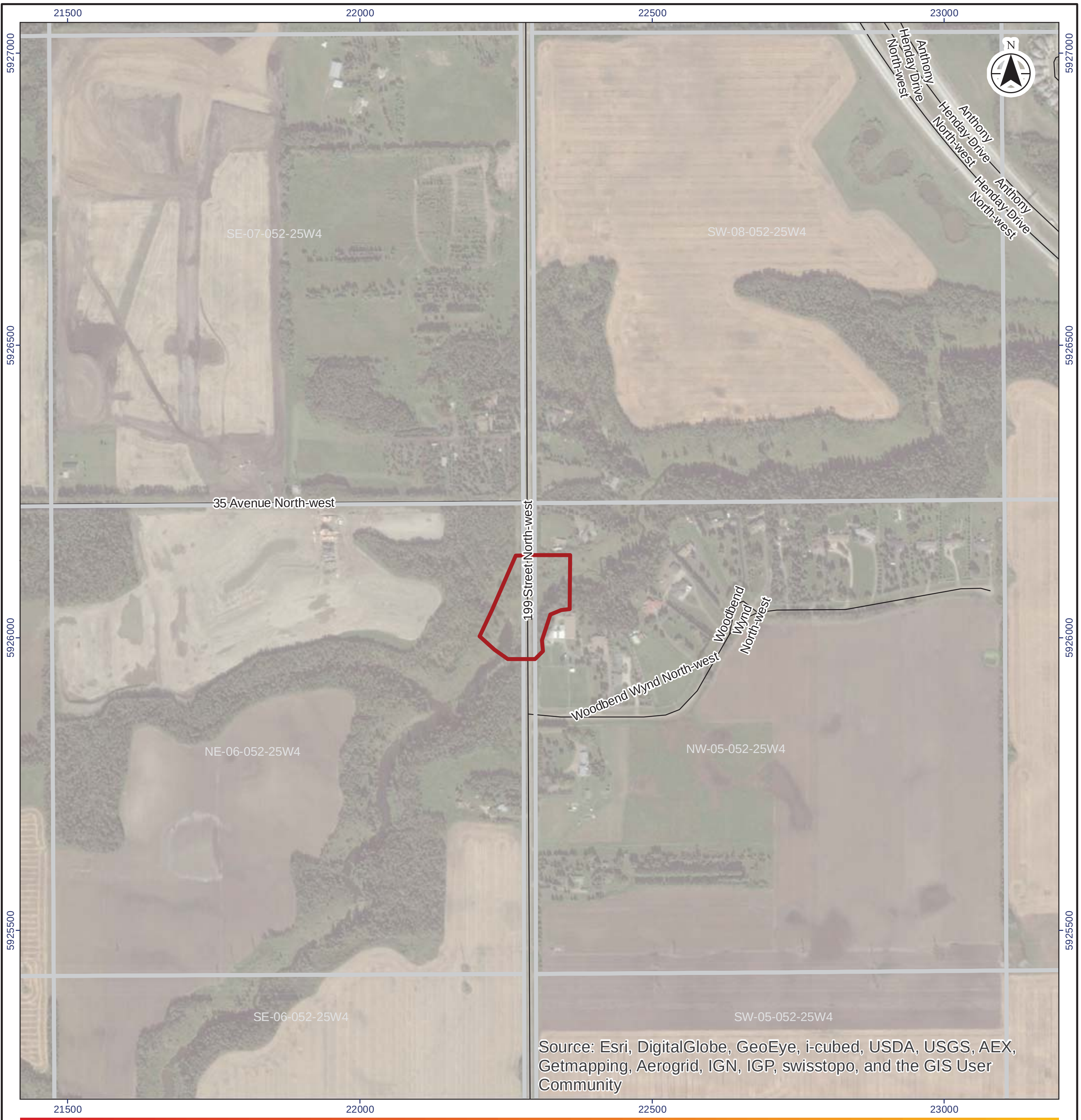
Terrace Engineering Ltd. (Terrace) 2014. ***Concept Bridge Planning Report – Wedgewood Creek*** *Crossing on 199 Street NW in the City of Edmonton*. Draft report prepared by G. Eitzen for the Riverview Ownership Group, May 2014.

Wildlife Act Wildlife Regulation. 1997. Alberta Regulation 143/1997. With amendments up to and including Alberta Regulation 124/2014

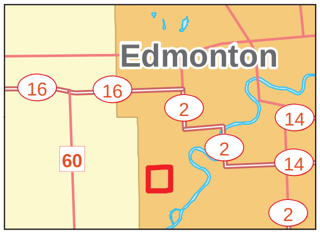
Appendix A

Site Location, Fish Habitat Map, Habitat Classification

\\Cd1001-c200\WORKGROUP\1102\active\110219229\gis\Figures\Fig1_229_SiteLocationPlan.mxd Revised: 2014-11-25 By: ssteenbergen



Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community



🟥 StudyArea

— Road

☐ Quarter Section

0 100 200 metres

1:10,000 (at original document size of 8.5x11)

Stantec

Project Location
06-052-25 W4M
Edmonton, Alberta

110219229
Prepared by SE on 2014-11-25
Review by SD on 2014-11-25

Client/Project
Riverview Owners Group

Figure No.
1

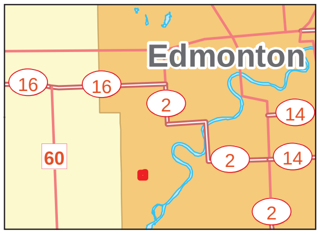
Title
Wedgewood Creek:
Site Location Plan

Page 01 of 01

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features Geogratis,©Department of Natural Resources Canada, All rights reserved.
3. Imagery obtained from Alberta Sustainable Resource Development, AS4986 27, 2008.

\\Cd1001-c200\WORKGROUP\1102\active\110219229\gis\Figures\Fig2_229_FishandFishHabitatAssessment2.mxd Revised: 2014-11-25 By: ssteenbergen

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community



● Electrofishing

● Water Quality Sample

→ Flow Direction

✕✕✕ Beaver_Dam

☐ Quarter Section

Habitat Unit*

🟩 Impoundment (IP1)

▨ Culvert

▨ Run (R3)

0 20 40 metres

1:2,500 (at original document size of 8.5x11)

Notes

1. Coordinate System: NAD 1983 3TM 114
2. Base features produced under license with Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

*Note: For definition of Habitat Classification see Appendix A

Stantec

Project Location
06-051-25 W4M
Edmonton, Alberta

110219229
Prepared by SE on 2014-11-25
Review by SD on 2014-11-25

Client/Project
Riverview Owners Group

Figure No.
2

Title
Wedgewood Creek:
Fish Habitat Assessment

Channel Unit Classification for Small Rivers or Streams (AT 2009)

Channel Unit	Class	Map Symbol	Description
Riffle		RF	Partially to totally submerged pebble to cobble substrate, causing moderate turbulence and ripples, little to no whitewater (some whitewater at points of constriction), moderate velocity (0.2 to 0.5 m/s), usually < 0.5 m depth, 1 - 4% slope.
Pool			Pools are deeper and wider than channel units immediately above or below it and are usually formed by the scouring or plunging action of water. Sub-surface velocities are slow (water surface may be fast and turbulent depending on formative feature) and the substrate usually composed of fines or small gravel.
	1	P1	High quality pool habitat based on depth and size. High instream cover from instream features (i.e., logs/boulders) and depth (> 1.2 m deep), provides overwintering habitat.
	2	P2	Shallower than P1 (0.6 - 1.2 m deep), moderate to high instream cover, not suitable for overwintering but provides juvenile and adult fish rearing habitat during open water.
	3	P3	Shallow (< 0.6 m deep) and small, low instream cover. Not suitable for overwintering or adult holding habitat but may provide rearing habitat for juvenile fish during open water
Flat		FL	Area characterized by low velocity and near-uniform flow; differentiated from pool habitat by high channel uniformity; more depositional than R3 habitat
Chute		CH	Area of channel constriction, usually due to bedrock intrusions; associated with channel deepening and increased velocity.
Dam	Beaver	BD	Structures causing complete or nearly complete channel blockage. Dams tend to accumulate more sediment/organic debris than scour pools.
Boulder Garden		BG	Substantial occurrence of large boulders providing significant instream cover; always in association with an overall channel unit such as a riffle (RF/BG) or run (e.g., R1/BG).
Run			Runs are typically deep, slow to swift flowing sections (> 0.2 m/s), with a gravel to boulder substrate. Defined thalweg, moderate slope and with no surface turbulence. Run units are differentiated into three classes, based on depth.
	1	R1	Deepest run (> 1 m), slow to fast water velocity, coarse substrate (cobble to boulder), high instream cover from substrate and depth.
	2	R2	Moderate depth (0.6 - 1.0 m), slow to fast water velocity, coarse substrate (cobble to boulder), moderate instream cover from substrate and depth.
	3	R3	Shallowest depth (0.3 - 0.6 m), slow to fast water velocity, coarse substrate (gravel to cobble), low instream cover.
Substrate	clay	cl	< 0.004 mm diameter, greasy feel between fingers
	silt	si	0.004 - 0.06 mm diameter, finer texture than sand
	sand	sa	0.06 - 2 mm diameter, gritty feel between fingers
	small gravel	gr(s)	2 - 16 mm diamter, sometimes called pea gravel
	gravel	gr	16 - 64 mm diameter
	small cobble	co(s)	64 - 128 mm diameter
	cobble	co	128 - 256 mm diameter
	boulder	bo or	> 256 mm diameter, any rock larger than a human head
	bedrock	bd	solid exposed rock with no overburden
	muck	mu	highly decomposed soft, fine organic material that may contain silt/clay
	detritus	dt	organic material composed of pieces of sticks, leaves, twigs and decayed plants

Appendix B
Field Survey Data

Stantec

Riverview Owners Group - Wedgewood Road Crossing

Site 1: Wedgewood Creek

UTM Location:	12N 323176 5927544	**Survey Date:**	October 8, 2014
Legal Location:	05-52-025 W4M	**Water Body Class:**	Class C
Crew Initials:	SD, SE	**Restricted Activity Period:**	Sept 16 - July 31

Physical Channel Transect Data

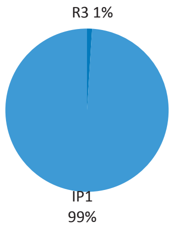
Transect # (Location)	**1 (↑100)**	**2 (↑50)**	**3 (CL)**	**4 (↓100)**	**5 (↓200)**	**6 (↓300)**
Channel Width (m)	-	-	-	-	-	-
Wetted Width (m)	21	11	5.0	3.2	10	13
Depth at LDB + 25% (m)	>1	0.70	0.82	0.50	>1	>1
Depth at LDB + 50% (m)	>1	0.80	0.91	0.47	0.10	>1
Depth at LDB + 75% (m)	>1	0.65	0.78	0.50	1.3	1.0
Max. Depth (m)	>1	>1	0.91	0.60	>1	>1
Gradient (%)	-	-	-	1	-	-
Dominant Habitat Unit	IP1	IP1	IP1	R3	IP1	IP1
Stream Bed						
Substrate (% of Transect Area) Organics	-	-	-	-	-	-
Fines	100	80	40	20	80	90
Small Gravel	-	15	30	50	20	10
Large Gravel	-	-	15	20	-	-
Cobble	-	-	10	10	-	-
Boulder	-	5	5	-	-	-
Bedrock	-	-	-	-	-	-
Embeddedness	H	H	M	M	VH	VH

Bank Measurements	**Left**	**Right**	**Left**	**Right**	**Left**	**Right**	**Left**	**Right**	**Left**	**Right**	**Left**	**Right**
Bank Height (m)	-		-		-		-		-		-	
Bank Slope (°)	-	-	-	-	-	-	-	-	-	-	-	-
Bank Stability	S	S	US	US	US	US	US	US	US	US	US	US
Dom. Bank Material	F	F	F	F	F	F	F	F	F	F	F	F
Subdom. Bank Material	F	F	F	F	F	F	F	F	F	F	F	F
Dom. Riparian Veg.	S	G	G	G	G	G	G	G	G	G	G	G
Subdom. Riparian Veg.	G	M	S	S	M	M	S	S	M	M	M	M

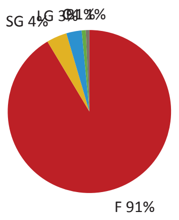
Habitat Inventory / Reach Data

Instream Cover (%):	99	Overhead Cover (%):	36
Dom. Instream Cover:	DC	Dom. Overhead Cover:	GF
Subdom. Instream Cover:	AV	Subdom. Overhead Cover:	UB
Maximum Depth (m)	1.0	Dom. Aquatic Veg. Type:	SB

Habitat Distribution



Substrate Composition



Water Quality Data		**Channel Characteristics**	
Time of Day (HH:MM):	09:35	Pattern:	IM
Water Temperature (°C):	8.7	Islands:	N
Dissolved Oxygen (mg/L):	10.49	Bars:	N
Sp. Conductivity (µs/cm):	771	Coupling:	DC
pH:	7.51	Confinement:	OC
Turbidity (NTU):	8.22	Flow Stage:	Flood

Fish Habitat Assessment Ratings

	Forage	**Coarse**	**Sportfish**
Spawning:	Good	Poor	Poor
Overwintering:	Good	Poor	Poor
Rearing:	Good	Poor	Poor
Migration:	Poor	Poor	Poor

Photo 1: Downstream at centerline (road) looking at IP1 habitat.

Photo 2: Upstream at centerline (road) looking upstream to culvert.

Fish Sampling Data

Method	**Effort**	
Backpack Electrofisher (EB)	304	(s)
No Trapping	--	(hr)

Electrofisher Settings

Volts	Freq. (Hz)	Duty Cycle (%)	Dist. (m)
200	60	12	30

Species	**Efish Catch** (n)	**Trap Catch** (n)	**Efish CPUE** (#fish/100s)	**Trap CPUE** (#fish/hr)	**Rel. Abundance** (% of total)
FINESCALE DACE	1		0.33		16.7%
BROOK STICKLEBACK	5		1.64		83.3%

General Comments

Observed 20 BRST downstream of culvert in IP1 habitat by a beaver dam.

Prepared by: Sam Eggink Reviewed by: Shona Derlukewich

Stantec

Riverview Owners Group - Wedgewood Road Crossing

Site 1: Wedgewood Creek

UTM Location:	12N 323176 5927544	**Survey Date:**	October 8, 2014
Legal Location:	05-52-025 W4M	**Water Body Class:**	Class C
Crew Initials:	SD, SE	**Restricted Activity Period:**	Sept 16 - July 31

Site Photographs

Photo 3: Finescale dace caught during electrofishing efforts downstream of the road crossing.

Photo 4: View upstream into exsisting culvert.

Photo 5: Upstream from road crossing looking at IP1 habitat due to beaver activity.

Photo 6: Beaver dam on a culvert upstream of the road crossing creating a temporary fish barrier and flooding areas upstream.

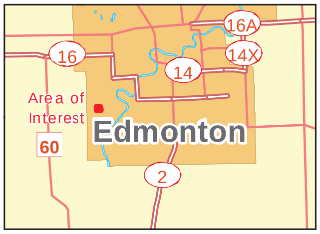
General Comments

Beaver activity affected flooded the area and flow was noted in areas downstream of beaver dams. The majority of the creek was beaver impounded creating temporary fish passage barriers.

Prepared by: Sam Eggink Reviewed by: Shona Derlukewich

APPENDIX E
HISTORICAL AIR PHOTOS

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1950.mxd Revised: 2015-05-26 By: jacho



▭ Construction Footprint

0 25 50 75 100
metres

1:3,000 (at original document size of 8.5x11)

Stantec

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

11029389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

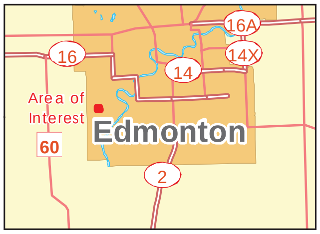
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

Figure No.
1950

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 136-58, 1950-05-04

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1962.mxd Revised: 2015-05-26 By: jacho



□ Construction Footprint

0 25 50 75 100 metres

1:3,000 (at original document size of 8.5x11)

Notes

1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 818-14, 1962-05-23

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

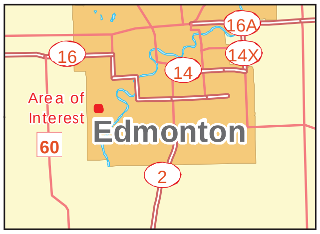
11029389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

Figure No.
1962

Title
Historic Aerial Photograph Review

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1967.mxd Revised: 2015-05-26 By: jacho



▭ Construction Footprint

0 25 50 75 100
metres
1:3,000 (at original document size of 8.5x11)

Stantec

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

110219389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

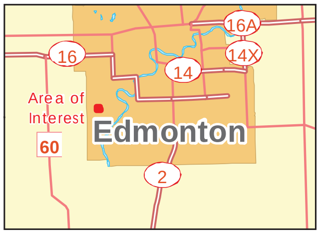
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

Figure No.
1967

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 979-43, 1967-08-15

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1972.mxd   Revised: 2015-05-26 By: jacho



▭ Construction Footprint

0 25 50 75 100 metres

1:3,000 (at original document size of 8.5x11)

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

110219389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

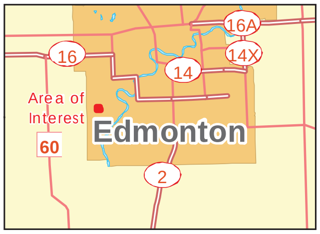
Figure No.
1972

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 1207-46, 06-10-1972

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1977.mxd Revised: 2015-05-26 By: jacho

🟧 Construction Footprint



Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 1592-37, 1977-06-16

0 25 50 75 100 metres

1:3,000 (at original document size of 8.5x11)

Stantec

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

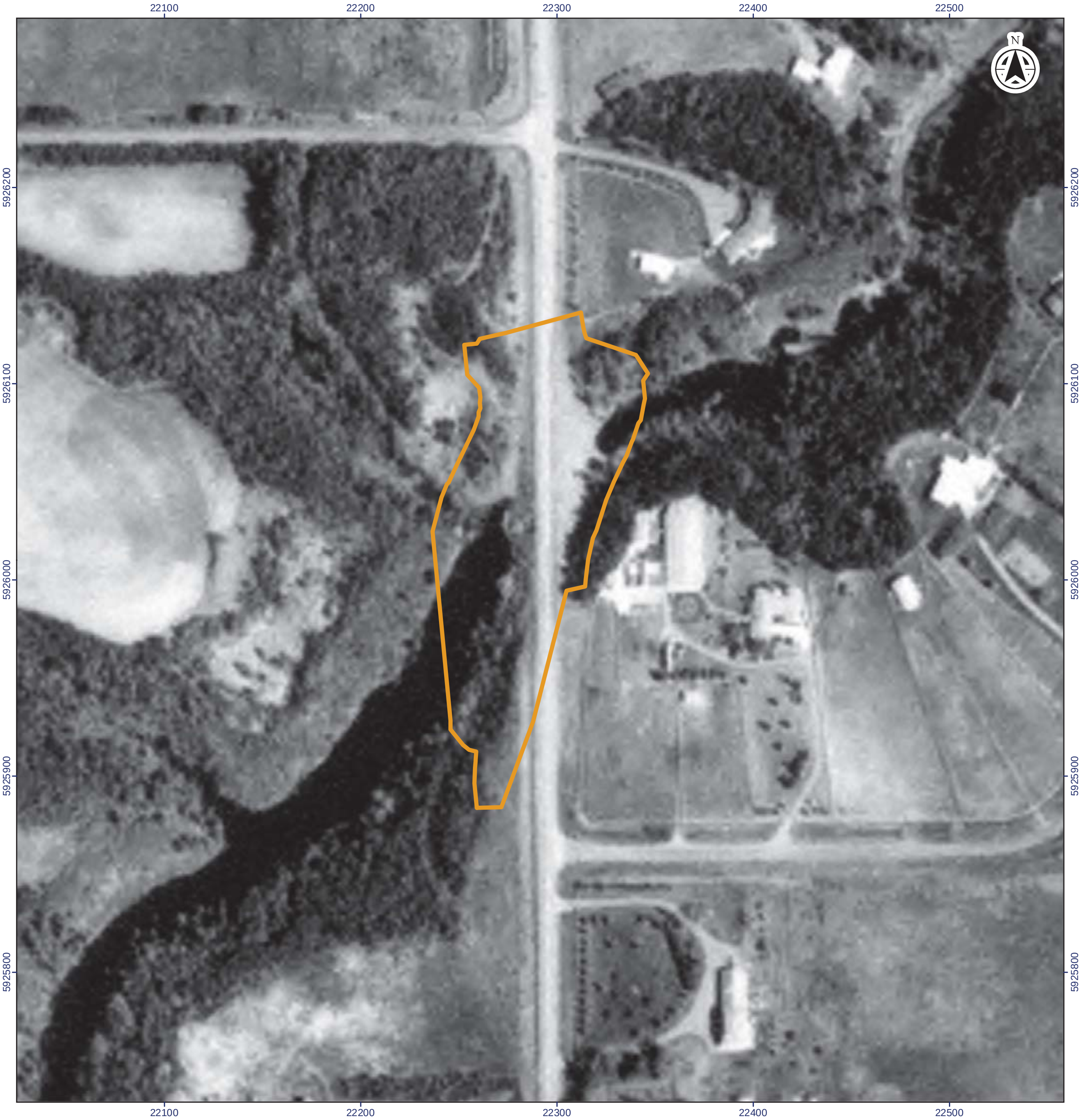
110219389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

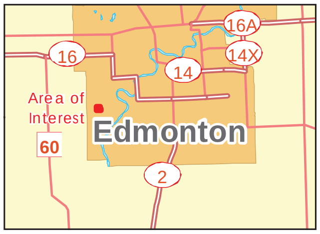
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

Figure No.
1977

Title
Historic Aerial Photograph Review

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1987.mxd Revised: 2015-05-26 By: jacho





▭ Construction Footprint

0 25 50 75 100 metres

1:3,000 (at original document size of 8.5x11)

Stantec

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

110219389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

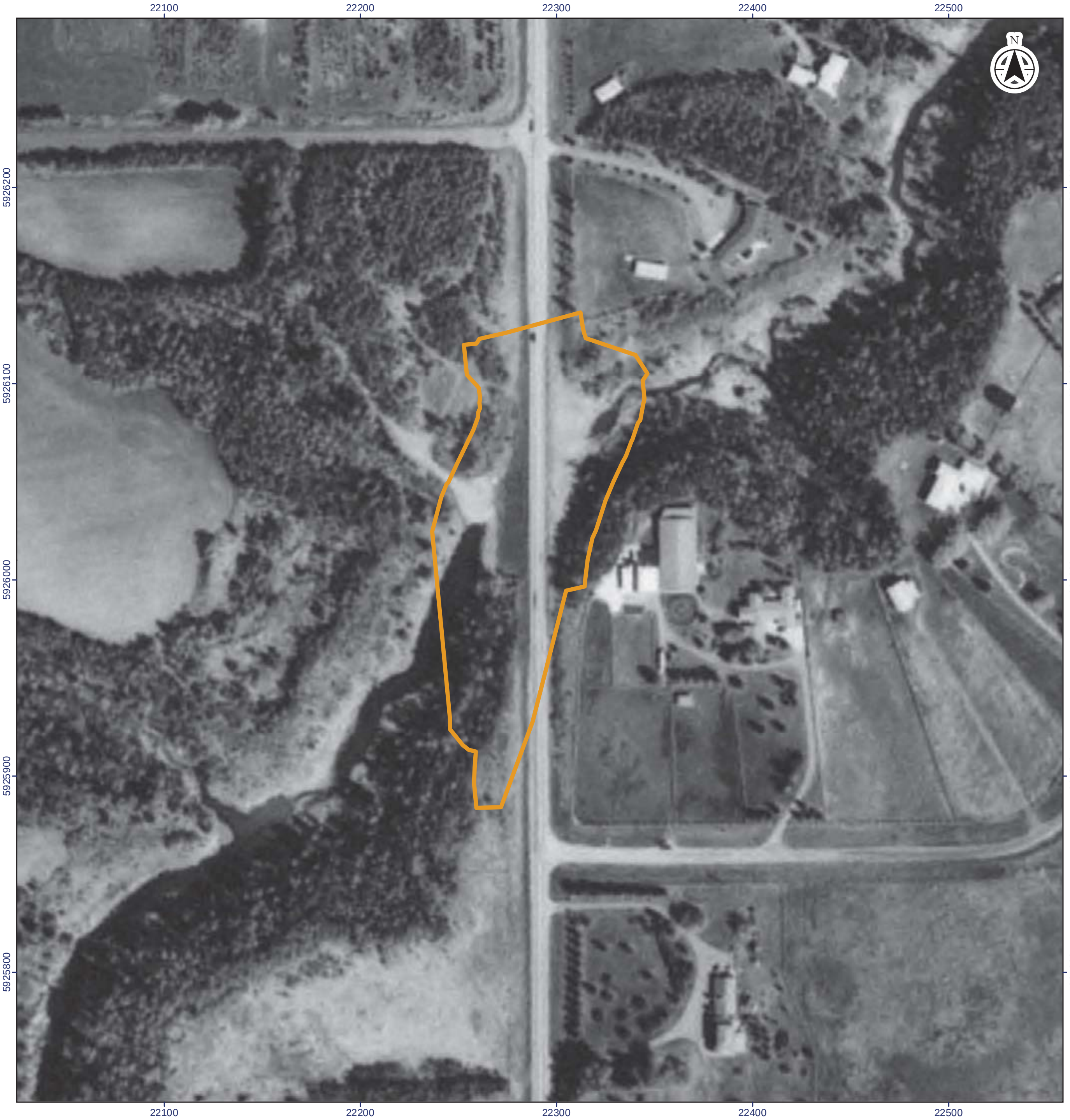
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

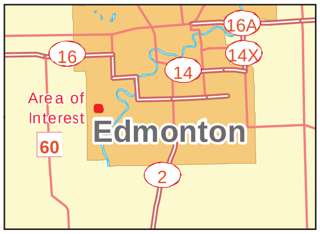
Figure No.
1987

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 3590-39, 1987-06-01

V:\1102\active\110219229\gis\Figures\Historic\Subsite_1993.mxd Revised: 2015-05-26 By: jacho





▭ Construction Footprint

1:3,000 (at original document size of 8.5x11)

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

110219389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

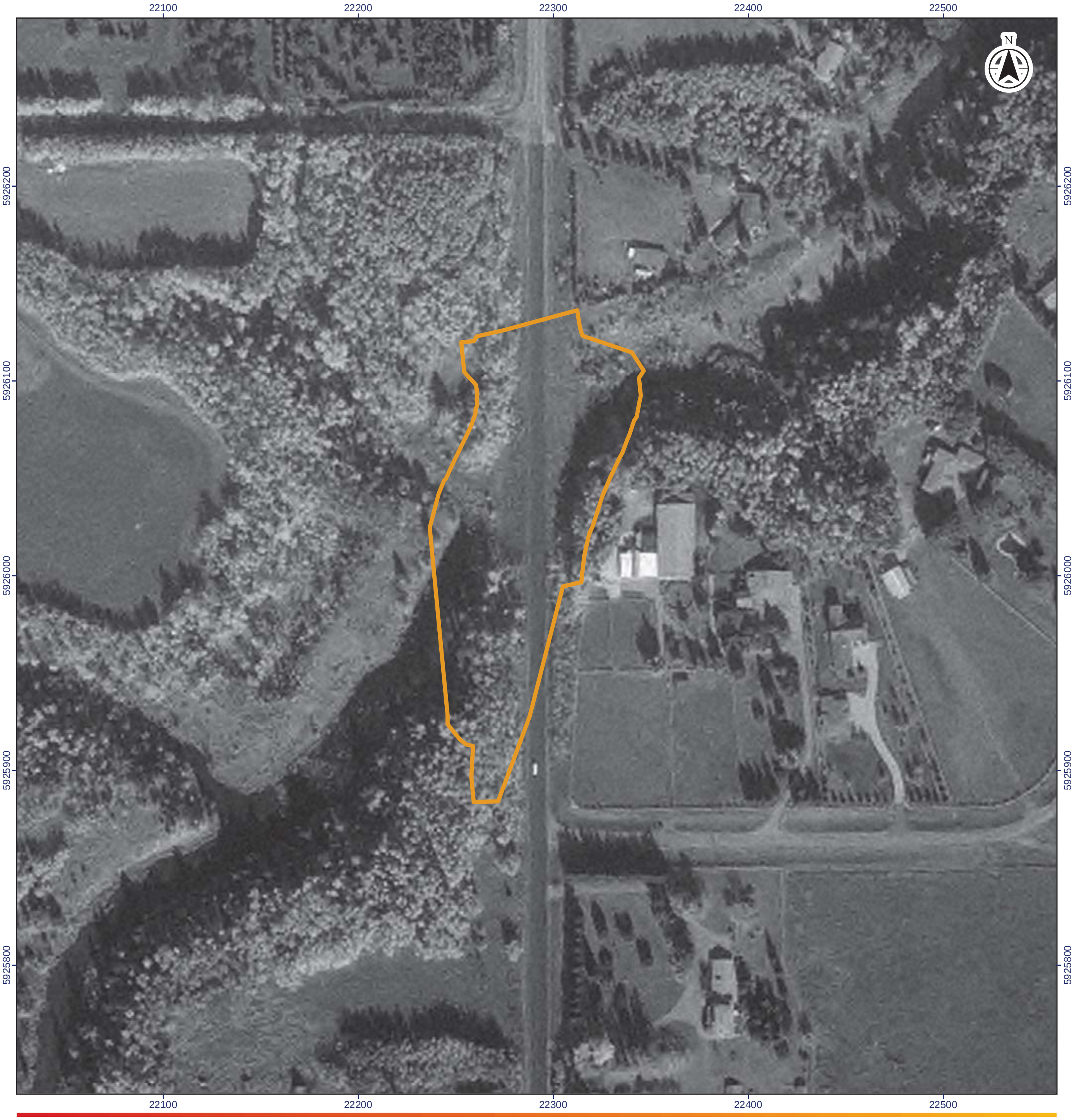
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

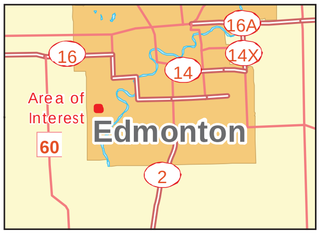
Figure No.
1993

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 4383-209, 1993-05-12

V:\1102\active\110219229\gis\Figures\Historic\Subsite_2002.mxd Revised: 2015-05-26 By: jacho





▭ Construction Footprint

0 25 50 75 100 metres

1:3,000 (at original document size of 8.5x11)

Stantec

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

11029389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

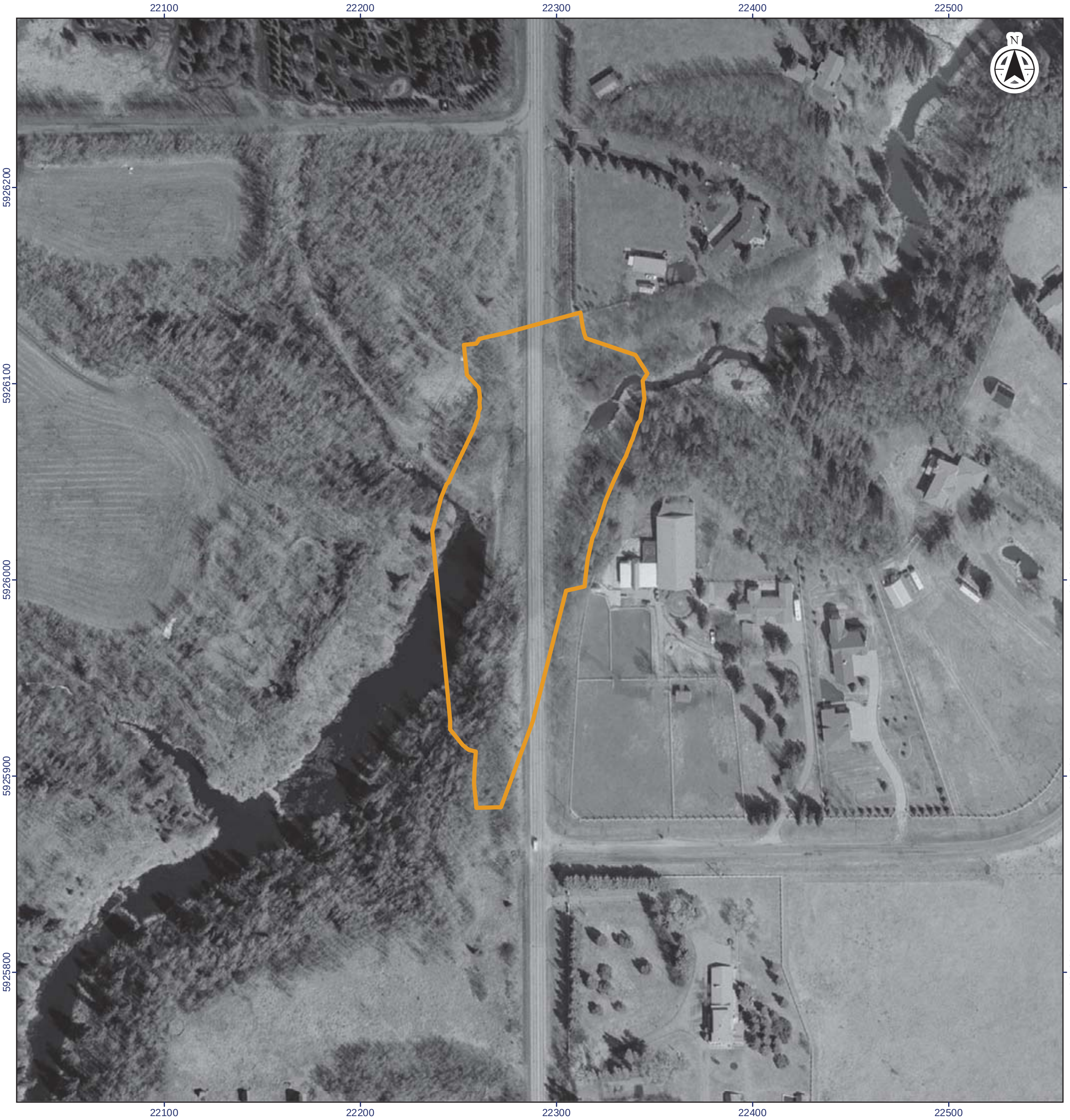
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

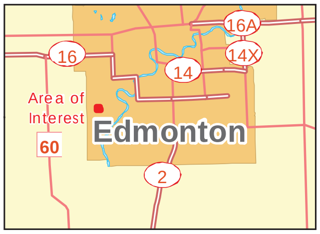
Figure No.
2002

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: Alberta Environment and Sustainable Resource Development, 5462B-67, 2002-10-08

V:\1102\active\110219229\gis\Figures\Historic\Subsite_2007.mxd Revised: 2015-05-26 By: jacho





▭ Construction Footprint

0 25 50 75 100
metres
1:3,000 (at original document size of 8.5x11)

Notes

1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: City of Edmonton Transportation, 2007

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

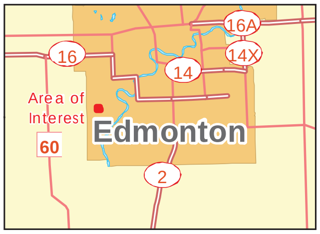
11029389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

Figure No.
2007

Title
Historic Aerial Photograph Review

V:\1102\active\110219229\gis\Figures\Historic\Subsite_2012.mxd Revised: 2015-05-26 By: jacho



🟧 Construction Footprint

0 25 50 75 100 metres

1:3,000 (at original document size of 8.5x11)

Project Location
NE¼ 06-052-25 W4M
NW¼ 05-052-25 W4M
Edmonton, Alberta

11029389
Prepared by MK on 2015-04-27
Review by JC on 2015-04-27

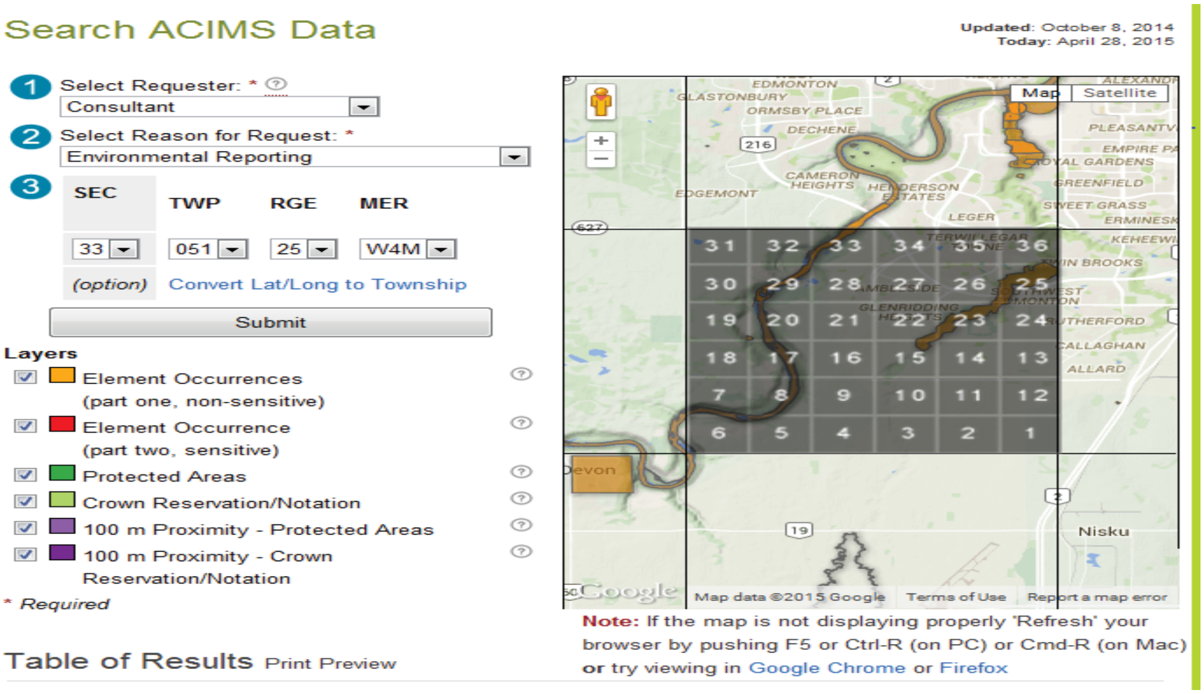
Client/Project
Riverview Owners Group
199 St Road Widening at Wedgewood Creek

Figure No.
2012

Title
Historic Aerial Photograph Review

Notes
1. Coordinate System: NAD 1983 3TM 114
2. Base features: Geogratis, ©Department of Natural Resources Canada, All rights reserved.
3. Imagery: City of Edmonton Transportation, 2012

APPENDIX F
DESKTOP AND FIELD DATA SUMMARY



Search ACIMS Data

Updated: October 8, 2014
Today: April 28, 2015

① Select Requester: * Consultant

② Select Reason for Request: * Environmental Reporting

③

SEC	TWP	RGE	MER
33	051	25	W4M
(option)	Convert Lat/Long to Township		

Submit

Layers

- ☑ 🟧 Element Occurrences (part one, non-sensitive)
- ☑ 🟥 Element Occurrence (part two, sensitive)
- ☑ 🟩 Protected Areas
- ☑ 🟩 Crown Reservation/Notation
- ☑ 🟪 100 m Proximity - Protected Areas
- ☑ 🟪 100 m Proximity - Crown Reservation/Notation

* *Required*

Note: If the map is not displaying properly 'Refresh' your browser by pushing F5 or Ctrl-R (on PC) or Cmd-R (on Mac) or try viewing in Google Chrome or Firefox

Table of Results Print Preview

Date: 28/4/2015
Requestor: Consultant
Reason for Request: Environmental Reporting
SEC: 33 **TWP:** 051 **RGE:** 25 **MER:** 4

Alberta Parks

🟧 Non-sensitive EOs: 2 (*Data Updated:Oct 2014*)

M-RR-TTT-SS	EO_ID	ECODE	S_RANK	SNAME	SCOMNAME	LAST_OBS_D
4-25-051-33	22925	IMGASN3050	SU	Ferrissia rivularis	Creeping Ancylid	2001-XX-XX
4-25-051-33	4536	NBMUS6W010	S2	Scouleria aquatica	moss	6/11/1979

Next Steps: See FAQ

🟥 Sensitive EOs: 0 (*Data Updated:Oct 2014*)

M-RR-TTT	EO_ID	ECODE	S_RANK	SNAME	SCOMNAME	LAST_OBS_D

No Sensitive EOs Found: Next Steps - See FAQ

🟩 Protected Areas: 0 (*Data Updated:April 2013*)

M-RR-TTT-SS	PROTECTED AREA NAME	TYPE	IUCN

No Protected Areas Found

🟩 Crown Reservations/Notations: 0 (*Data Updated:April 2013*)

M-RR-TTT-SS	NAME	TYPE

No Crown Reservations/Notations Found

Environment and Sustainable
Resource Development

Fish and Wildlife Internet Mapping Tool (FWIMT)

(source database: Fish and Wildlife Management Information System (FWMIS))

Species Summary Report

Report Created: 13-Apr-2015 16:10

Species present within the current extent :

Fish Inventory	Wildlife Inventory	Aquatic Inventory	Stocked Inventory
BROOK STICKLEBACK	LEAST FLYCATCHER	No records found.	No records found.
FATHEAD MINNOW	SORA		

Buffer Extent

Centroid (X,Y):	Projection	Centroid: (Qtr Sec Twp Rng Mer)	Buffer Radius:
588634, 5922690	10-TM AEP Forest	NW 5 52 25 4	2 kilometers

Wildlife Contact Information

Primary Contact

Name: Delaney Anderson **Phone:** 780-415-1328 **Email:** Delaney.Anderson@gov.ab.ca **Town:**

Alternative

Name: **Phone:** **Email:** **Town:**

Fisheries Contact Information

Primary Contact

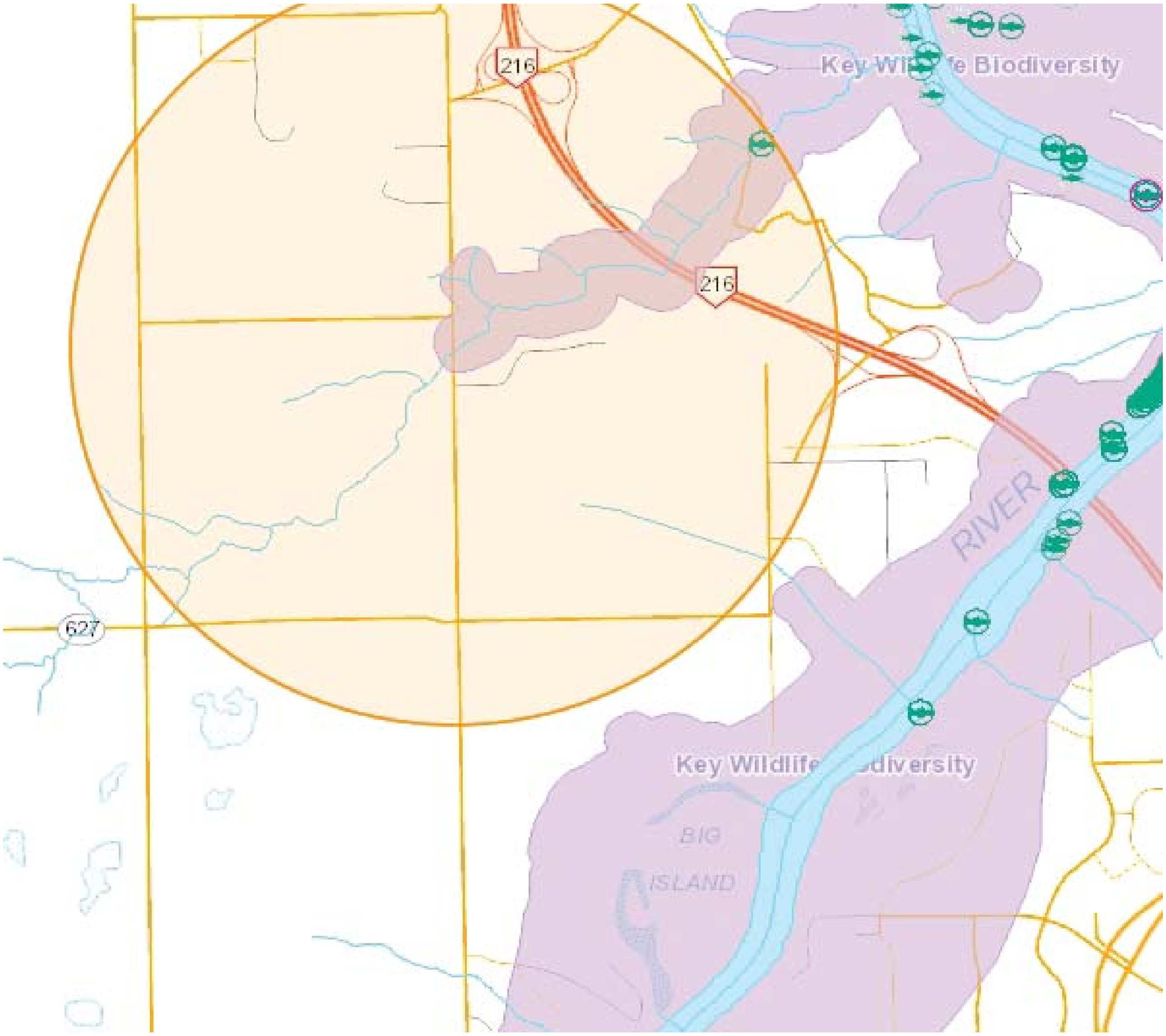
Name: FRLs:Denyse Gullion **Phone:** 780-675-8205 **Email:** Denyse.Gullion@gov.ab.ca **Town:** Athabasca

Alternative

Name: **Phone:** **Email:** **Town:** Athabasca

13-Apr-2015 16:10

Map Results



Comprehensive Species List

Scientific Name	**Common Name**	**Provincial Scientific Name**	**Provincial Common Name**	**Plant Forborm**
Anemone canadensis	Canada anemone, Canadian anemone	*Anemone canadensis*	Canada anemone	Forb
Aralia nudicaulis	wild sarsaparilla	*Aralia nudicaulis*	wild sarsaparilla	Forb
Chamerion angustifolium	fireweed			Forb
Equisetum arvense	common horsetail, field horsetail, scouringrush, western horsetail	*Equisetum arvense*	common horsetail	Forb
Equisetum hyemale	horsetail, scouring horsetail, scouringrush, scouringrush horsetail,	*Equisetum hyemale*	common scouring-rush	Forb
Eurybia conspicua	eastern showy aster, western showy aster	*Eurybia conspicua*	showy aster	Forb
Fragaria virginiana	thickleaved wild strawberry, Virginia strawberry, wild strawberry	*Fragaria virginiana*	wild strawberry	Forb
Galium boreale	northern bedstraw	*Galium boreale*	northern bedstraw	Forb
Geranium richardsonii	Richardson's geranium	*Geranium richardsonii*	wild white geranium	Forb
Heracleum sphondylium ssp. montanum	common cowparsnip, cow parsnip	*Heracleum lanatum*	cow parsnip	Forb
Lathyrus ochroleucus	cream pea, cream peavine, pale vetchling peavine	*Lathyrus ochroleucus*	cream-colored vetchling	Forb
Maianthemum stellatum	starry false lily of the valley, star-flower Solomon's-seal, star false	*Smilacina stellata*	star-flowered Solomon's-seal	Forb
Mertensia paniculata	tall bluebells	*Mertensia paniculata*	tall lungwort	Forb
Moehringia lateriflora	blunt-leaf grove-sandwort, bluntleaf sandwort, grove sandwort	*Moehringia lateriflora*	blunt-leaved sandwort	Forb
Pyrola asarifolia	bog wintergreen, liverleaf wintergreen, pink wintergreen	*Pyrola asarifolia*	common pink wintergreen	Forb
Symphyotrichum ciliolatum	Lindley's aster	*Symphyotrichum ciliolatum*	Lindley's aster	Forb
Taraxacum officinale	blowball, common dandelion, dandelion, faceclock	*Taraxacum officinale*	common dandelion	Forb
Vicia americana	American deervetch, American purple vetch, American vetch	*Vicia americana*	wild vetch	Forb
Agropyron	other wheatgrasses, wheatgrass			Graminoid
Anthoxanthum nitens				Graminoid
Bromus inermis	awnless brome, smooth brome	*Bromus inermis*	smooth brome	Graminoid
Carex siccata	dryspike sedge	*Carex siccata*	hay sedge	Graminoid
Poa pratensis	Kentucky bluegrass	*Poa pratensis*	Kentucky bluegrass	Graminoid
Amelanchier alnifolia	juneberry, Pacific serviceberry, Saskatoon serviceberry, western	*Amelanchier alnifolia*	saskatoon	Shrub
Cornus sericea ssp. sericea	redosier dogwood, Siberian dogwood, Tatarian dogwood			Shrub
Corylus cornuta	beaked hazel, beaked hazelnut, western hazel	*Corylus cornuta*	beaked hazelnut	Shrub
Prunus virginiana	chokecherry, common chokecherry, Virginia chokecherry	*Prunus virginiana*	choke cherry	Shrub
Ribes oxyacanthoides	Canadian gooseberry	*Ribes oxyacanthoides*	northern gooseberry	Shrub
Rosa acicularis	prickly rose	*Rosa acicularis*	prickly rose	Shrub
Rubus idaeus	American red raspberry, common red raspberry, western red	*Rubus idaeus*	wild red raspberry	Shrub
Salix lucida	shining willow	*Salix lucida*	shining willow	Shrub
Shepherdia canadensis	russet buffalo-berry, russet buffaloberry	*Shepherdia canadensis*	Canada buffaloberry	Shrub
Symphoricarpos occidentalis	western snowberry, wolfberry	*Symphoricarpos occidentalis*	buckbrush	Shrub
Viburnum edule	squashberry	*Viburnum edule*	low-bush cranberry	Shrub
Viburnum opulus	European cranberrybush, Guelder rose, viburnum	*Viburnum opulus*	high-bush cranberry	Shrub
Betula papyrifera	paper birch	*Betula papyrifera*	white birch	Tree
Picea glauca	Black Hills spruce, Canadian spruce, cat spruce, skunk spruce,	*Picea glauca*	white spruce	Tree
Populus balsamifera	balsam poplar	*Populus balsamifera*	balsam poplar	Tree
Populus tremuloides	quaking aspen	*Populus tremuloides*	aspen	Tree
Lonicera dioica	limber honeysuckle	*Lonicera dioica*	twining honeysuckle	Vine