

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED WATERMAIN AND SANITARY
SEWER REPLACEMENT, HIGHWAY 17,
TOWN OF MATTAWA, ONTARIO,
G.W.P. 339-00-00, GEOCRES 31L-142**

LEA Consulting Ltd.

TRANETOB01235AA-AB
August 18, 2010

August 18, 2010

LEA Consulting Ltd.
625 Cochrane Drive, Suite 900
Markham, Ontario
L3R 9R9

Attention: Mr. Rick Baldasti, P.Eng.

Dear Sir:

RE: Foundation Investigation and Design Reports - Proposed Watermain and Sanitary Sewer Replacement, Highway 17, Town of Mattawa, Ontario, G.W.P. 339-00-00, Geocres 31L-142

Coffey Geotechnics Inc (Coffey) is pleased to present the Foundation Investigation and Design Reports for the proposed watermain and sanitary sewer replacement along Highway 17, within the Town of Mattawa Ontario.

Please call us at (416) 213-1255 should you require further clarification on any aspects of the reports.

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P. Eng.

Manager, Transportation

Distribution: Original held by Coffey Geotechnics Inc.
7 bound and 1 unbound originals to LEA Consulting Ltd.

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PROPOSED WATERMAIN
AND SANITARY SEWER REPLACEMENT
HIGHWAY 17, TOWN OF MATTAWA,
ONTARIO, G.W.P. 339-00-00
GEOCRES 31L-142**

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**FOUNDATION INVESTIGATION REPORT
PROPOSED WATERMAIN AND SANITARY SEWER REPLACEMENT
HIGHWAY 17, TOWN OF MATTAWA, ONTARIO
G.W.P. 339-00-00**

1 INTRODUCTION

The project involves the tunnelling of a proposed new watermain and a sanitary sewer replacement on Highway 17, under an existing culvert at Station 21+425, in the Town of Mattawa. The existing culvert is 1.9 m x 1.2 m in size and skewed at 36 degrees relative to the Highway 17 centreline. The site is located on Highway 17 some 60 km east of North Bay, Ontario.

Coffey Geotechnics Inc. (Coffey) was retained by LEA Consulting Ltd. (LEA) to conduct a foundation investigation for the proposed watermain and sanitary sewer.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes, and to determine the engineering characteristics of the subsurface materials by means of field and laboratory tests.

This report provides factual information concerning the subsurface conditions, in situ and laboratory test results, based on the foundation investigation undertaken.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

The project site falls within the physiographical region known as the Algonquin Highlands, according to The Physiography of Southern Ontario by Chapman and Putnam, 1984. Topography of the area is rolling in nature. Overburden within the area is generally shallow and is underlain by Precambrian rocks with frequent outcrops. Thickness of overburden can often vary over short distances.

According to Map 2544, Bedrock Geology of Ontario, by the Ministry of Northern Development and Mines, the bedrock underlying the area consists primarily of gneisses of various origins and compositions. Overburden in the project area is glaciofluvial outwash deposits of sand and gravel or glacial till. The till in the area is predominantly of silty sand matrix with high clasts content.

The proposed watermain and sewer will be constructed under the existing culvert which carries the creek flow south to north under Highway 17. Within the project site, Highway 17 crosses through the Town of Mattawa with residential and commercial establishments on each side of the highway. The project site is about 200 m away from the Ottawa River. Boulders are found near the shore of Ottawa River.

Site photographs are presented in Appendix C.

3 FIELD AND LABORATORY WORK

The fieldwork for this project was performed on March 24, 2010 and consisted of drilling and sampling of a total of two boreholes, as requested by the MTO Regional Office in North Bay. Borehole 1 was put down at Station 21+412, 3.0 m right of the Highway 17 centreline and was advanced to 5.4 m below the ground surface. Borehole 2 was located at Station 21+445, 5.0 m left of the Highway 17 centreline and advanced

to 6.3 m below the ground surface. Continuous flight hollow stem auger was used in the overburden in both Boreholes 1 and 2. In Borehole 1, auger refusal was encountered at 2.0 m depth in the overburden and the borehole was subsequently advanced by coring through cobbles/boulders to the top of bedrock surface. Similarly, upon auger refusal in Borehole 2 due to possible cobbles/boulders at 2.1 m depth, wash boring with N type casing and coring was utilized to advance to the top of the bedrock surface. Bedrock in both boreholes was proven utilizing coring and obtaining core samples using NQ core barrel.

Two (2) probeholes, P1 and P2, were drilled without sampling to refusal depths to check the auger refusal depths at Station 21+442, 3.0 m right from the Highway 17 centreline and Station 21+415, 5.0 m left from the Highway 17 centreline, respectively. These probeholes were advanced to investigate the depths to the surface of bedrock and the possibility of encountering cobbles/boulders.

Landcore Drilling Inc. of Chelmsford, Ontario carried out the drilling, testing and sampling work of all boreholes. Fieldwork was conducted under the supervision and direction of technical staff (Mr. Gem Jiang, E.I.T.) from Coffey.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were measured by the client's surveyors. The survey information was provided to us.

In the overburden, sampling in the boreholes was carried out at frequent depth intervals by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer over a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground over a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of granular (cohesionless) soils (gravels, sands and coarse silts) or the consistency of cohesive soils (clays and clayey silts). As mentioned before, below the refusal depths the boreholes were advanced by coring.

Water level observations in the open boreholes (or casing) were made during the drilling and at the completion of each borehole.

The soil and rock core samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets in Appendices A and on the laboratory test results sheets in Appendix B.

4 SUMMARIZED SUBSURFACE CONDITIONS

Boreholes 1 and 2 were advanced for the foundation investigation for the proposed watermain and sanitary sewer crossings at Station 21+412 and 21+445 on Highway 17, respectively. Locations of these boreholes are presented in Drawing 1.

Boreholes 1 and 2 were advanced on the paved portion of the road at Elevation (El.) 156.0 and 155.8 m. A 0.8 m thick pavement structure was encountered from the existing grade. A sand deposit was contacted at 0.8 m from the existing ground (El. 155.2 and 155.0 m). This deposit was found to extend to 2.9 and 1.4 m

below the road surface or to El. 153.1 and 154.4 m in Boreholes 1 and 2, respectively. In Borehole 2, a deposit of silty sand was found to underlie the sand deposit at El. 154.4 m and this deposit was found to extend to El. 151.5 m (4.3 m below ground level). Underlying the sand in Borehole 1 and silty sand in Borehole 2, bedrock was encountered at 2.9 and 4.3 m below existing ground or at El. 153.1 and 151.5 m. The boreholes were advanced into bedrock by coring and were terminated within bedrock at 5.4 and 6.3 m below the existing grade or at El. 150.6 and 149.5 m, respectively. Probeholes 1 and 2, which were advanced to auger refusal without sampling, encountered refusal at 2.3 m (El. 153.6 m) and 3.4 m (El. 152.6 m), respectively.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. Appendix D presents rock core photographs and Appendix E presents the Explanation of Terms Used in Report. The borehole Location Plan and the inferred subsurface profile are presented in Drawing 1.

The following descriptions of the individual strata are to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the subsurface conditions may vary in between and beyond the borehole locations.

4.1 Pavement Structure

In Boreholes 1 and 2, which were advanced from the paved portion of the highway, a 110 – 125 mm thick asphalt layer was contacted. Underlying the asphalt is a layer of sand and gravel fill (i.e. base course) that is 0.1 – 0.2 m in thickness. A subbase of sand fill with some gravel and traces of silt, 0.6 – 0.7 m in thickness, was encountered below the sand and gravel base course. The total thickness of the pavement structure at the borehole locations was found to be 0.8 m.

Standard penetration tests performed in the base and subbase courses yielded N-values of 26 and 67 blows/0.3 m, indicating that these materials are compact to very dense. The recorded N-values indicate that these materials have probably received a systematic compaction when they were first placed.

4.2 Sand

A deposit of sand was found to underlie the pavement structure in Boreholes 1 and 2. This deposit ranges from 0.6 and 2.1 m in thickness and was contacted at 0.8 m below the existing ground or at El. 155.2 and 155.0 m, and extends to 2.9 and 1.4 m below grade (El. 153.1 and 154.4 m). This sand layer contains traces of gravel and silt.

In Borehole 1, in this deposit rock coring techniques were utilized to advance the borehole through boulders from 1.8 to 2.9 m below the existing ground or at El. 154.2 to 153.1 m.

The grain-size distribution of two (2) samples retrieved from the upper zones of this deposit is presented in Figure B-1 in Appendix B. The distributions are summarized below.

Gravel:	1 %
Sand:	92 – 94 %
Silt and Clay:	5 – 7 %

This deposit is considered as a granular (non-cohesive) material.

Standard Penetration Tests performed in this deposit yielded N-values which range from 18 to 85 blows/0.3 m which indicate that this deposit is in a compact to very dense condition. The higher SPT N-values in Borehole 1 may however be due to the presence of cobbles and boulders.

4.3 Silty Sand

Borehole 2 encountered a 2.9 m thick silty sand deposit underlying the sand deposit at El. 154.4 m or 1.4 m below existing ground level. This layer contains some gravel and extends to 4.3 m below the ground surface or to the surface of the bedrock at El. 151.5 m. Cobbles and boulders were encountered occasionally at shallower depths (i.e. between 1.4 and 2.6 m from existing ground level).

Grain-size distributions of two (2) samples obtained from this soil unit are presented in Figure B-2 in Appendix B and summarized as follows:

Gravel:	10 -16 %
Sand:	39 - 60 %
Silt:	20 - 47 %
Clay:	4 %

A layer of silty fine sand, about 0.5 m thick, was contacted in Borehole 2 at 3.2 m below the existing ground or Elevation 152.6 m. This layer contains traces of gravel and exhibits a dilatant behaviour in the presence of water.

Grain-size analysis was conducted on a sample retrieved from this layer as presented in Figure B-3 in Appendix B.

Gravel:	1 %
Sand:	54 %
Silt:	38 %
Clay:	7 %

This deposit is considered a granular (i.e. non-cohesive) material.

Standard penetration tests conducted in this soil unit yielded N-values between 31 and 59 blows/0.3 m. These results indicate that this basically granular material is dense to very dense.

It should be noted that this soil unit may possibly be a residual soil from extremely weathered bedrock as signs of rock structure and fabric were observed.

4.4 Bedrock

At depths of 2.9 m (El. 153.1 m) and 4.3 m (El. 151.5 m), a pinkish grey gneiss bedrock was contacted in Boreholes 1 and 2, respectively, and was proven by NQ coring.

Boreholes 1 and 2 were advanced into the bedrock for a vertical distance of 2.5 and 2.0 m, respectively. The measured percentage of total core recovery (TCR) of the cores was 87 to 100 % while the RQD values vary from 0 in Borehole 2 to 42 to 57 % in Borehole 1. These results indicate a rock quality which ranges from very poor in Borehole 2 and poor to fair in Borehole 1.

Visual and tactile observations show that the bedrock cores retrieved from Borehole 2 are relatively weaker than the cores from Borehole 1. Soil samples retrieved from Borehole 2, near the auger refusal depth at 2.0 m, also displayed a rock structure and fabric and may be residual from the extreme weathering of the bedrock (e.g. friable). These observations suggest that bedrock may be highly to extremely weathered at some locations, but not throughout the entire project area since signs of extreme weathering were not observed on the rock core samples retrieved from Borehole 1.

As mentioned before, the surface of the bedrock was contacted at Elevations 153.1 and 151.5 m in Boreholes 1 and 2, respectively. It is of interest to note that auger refusal, possibly on boulders/bedrock, was encountered at depths of 2.3 m and 3.4 m or El. 153.6 and 152.6 m in Probeholes P1 and P2, respectively.

4.5 Groundwater Conditions

Groundwater levels were observed in casings of open borehole prior to coring (Borehole 1); 24 hours after completion in Borehole 2; and upon completion of probeholes (P1 and P2). In Borehole 2 where wash boring and NQ coring were used (i.e. water introduced into the boreholes), the recorded water level on completion of the borehole may not be reliable. The recorded water level observations in the boreholes and the probeholes are shown on the individual Record of Borehole sheets and are summarized in the following table.

Table 4.5.1 - Groundwater Observations

Borehole Number	Ground Elevation (m)	Date of Measurement	Water Level Depth / Elevation (m)
1*	156.0	March 24, 2010	Dry to 2.0 **/**
2*	155.8	March 25, 2010 (24 hours after completion)	1.4 / 154.4*
P1	155.9	March 25, 2010	2.0 / 153.9**
P2	156.0	March 25, 2010	Dry**

*water used for wash boring and/or coring

** not stabilized

*** observation made prior to coring of boulder/bedrock

Water level observed at the creek averaged at 1.4 m below the existing Highway 17 road level or approximately at El. 154.6 – 154.4 m. From the groundwater observations from boreholes and probeholes, as well as the natural moisture contents of the recovered soil samples, it is our opinion that the groundwater table at the time of our investigation was at 1.4 m below the road surface level or at about El. 154.6 – 154.4 m, similar to the water level in the watercourse.

It should be noted that the observed groundwater levels represent the conditions at the time of our investigation and they are subject to fluctuations, both seasonally and in response to major weather events. As well, the water level at the project site would also be influenced by the water level in the existing watercourse.

For and on behalf of Coffey Geotechnics Inc.

Winnie Chan, E.I.T.

Engineer-in-Training

Ramon Miranda, P.Eng.

Manager, Transportation

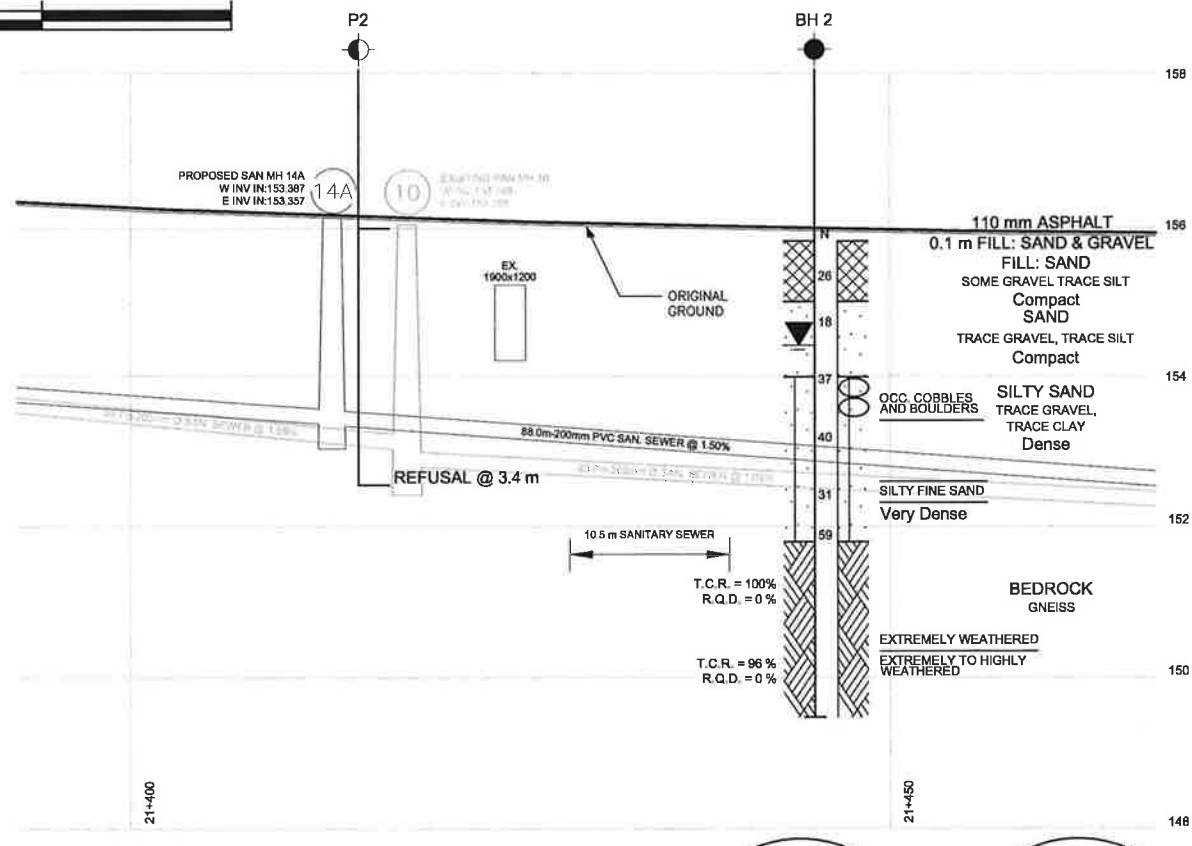
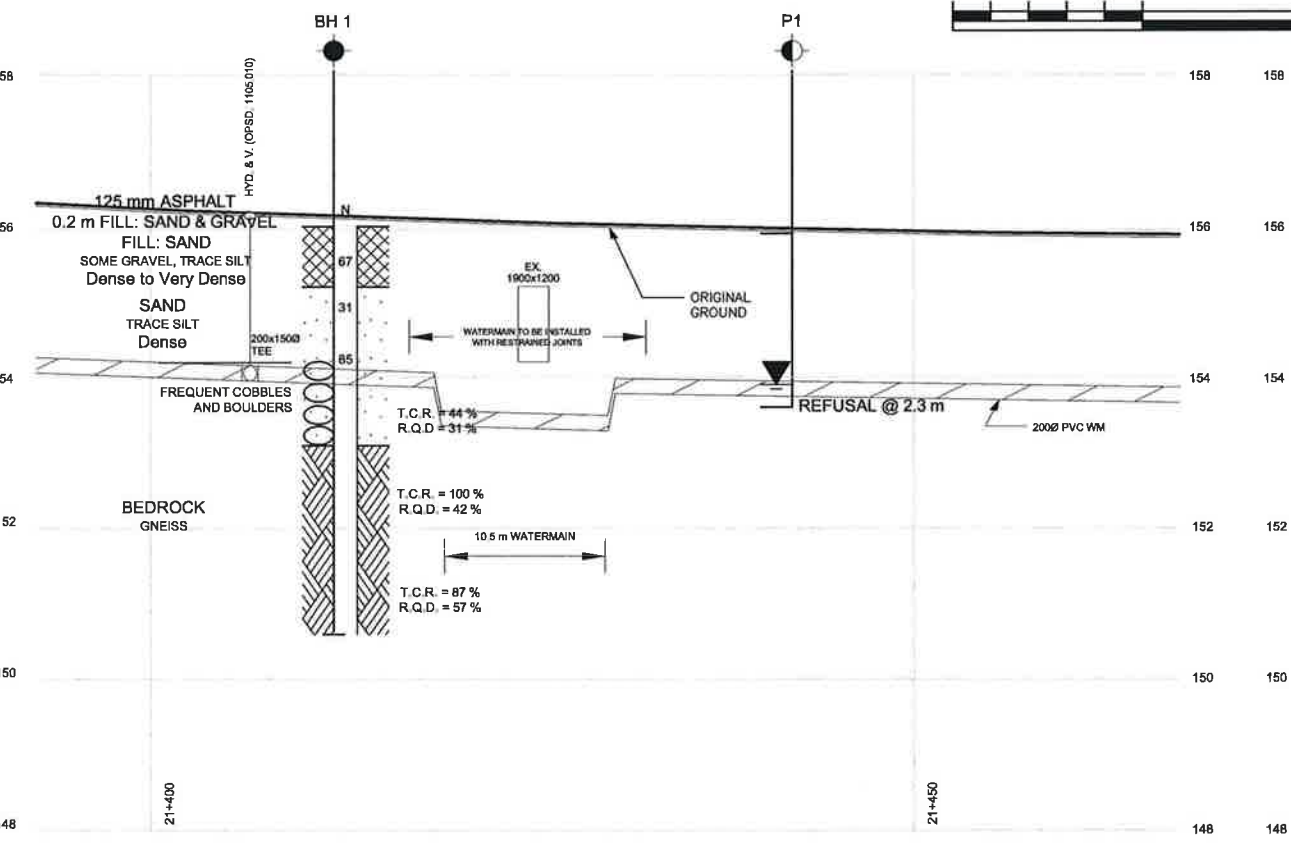
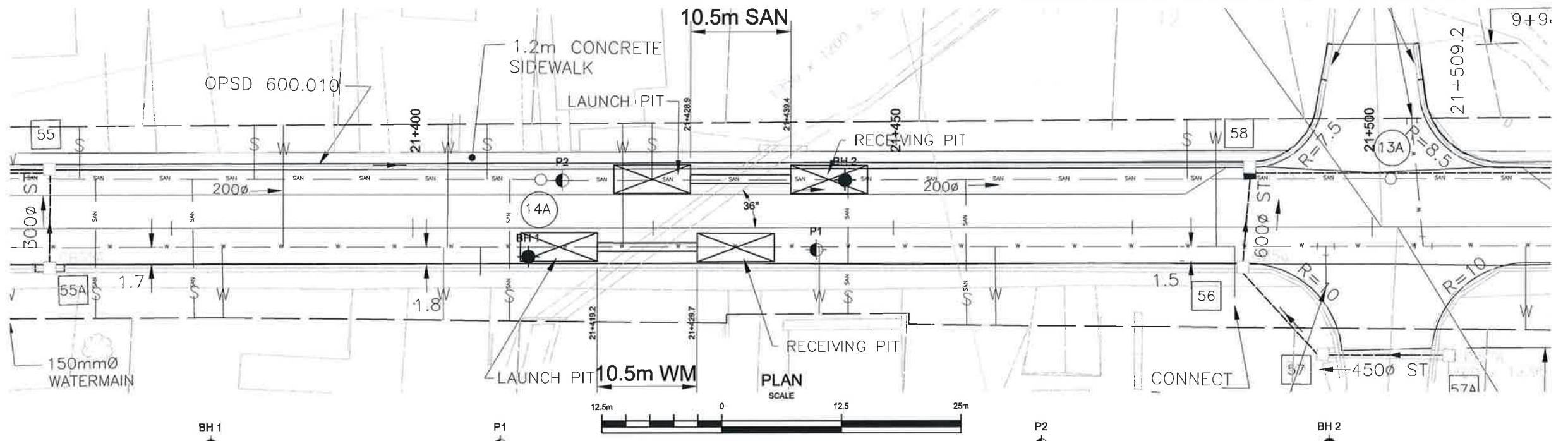


Zuhtu Ozden, P.Eng.

Senior Principal



Drawing



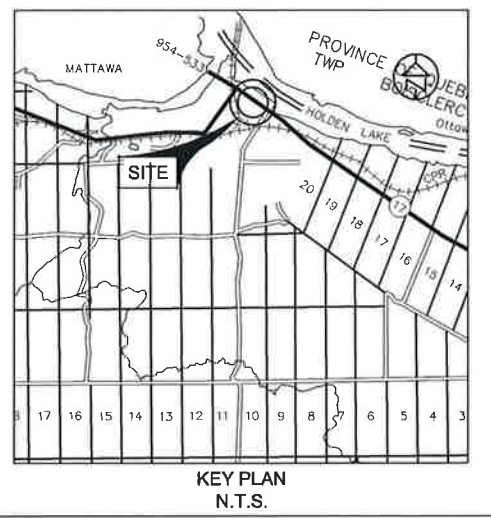
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.
GWP: 339-00-00

HIGHWAY 17, MATTAWA
BOREHOLE LOCATION PLAN
AND SOIL STRATA

SHEET



LEGEND

- Borehole
- Confirmatory Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer
- Existing Sanitary Sewer
- Existing Watermain

No.	ELEVATION	STATION	OFFSET
BH 1	156.0	21+412	3.0m Rt C/L
BH 2	155.8	21+445	5.0m Lt C/L
P1	155.9	21+442	3.0m Rt C/L
P2	156.0	21+415	5.0m Lt C/L

-NOTE-
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION



Geocres No 31L-142	TRANETO01235AA	DIST
SUBMD	CHECKED	DATE Aug 16, 2010
DRAWN	PHK	CHECKED RM
APPROVED	ZO	DWG
1		

Appendix A

Record of Borehole Sheets

TRANETO01235AA

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

GWP 339-00-00 LOCATION Sta: 21+412 ; 3.0 m Rt C/L Hwy 17 ORIGINATED BY GJ
DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY WC
DATUM Geodetic DATE 3/24/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR X LAB VANE					WATER CONTENT (%) w _p — w — w _L
156.0	GROUND SURFACE		1	AS									
0.0	125 mm ASPHALT		2	SS	67								
155.2	0.2 m FILL: Sand & Gravel												
0.8	FILL: Sand some gravel, tr. silt brown, v. dense, moist		3	SS	31								
	SAND												
	tr. silt, brown, dense, moist		4	SS	85								
	freq cobbles and boulders		5	RC									
153.1			6	RC									
2.9	BEDROCK												
	pinkish grey gneiss		7	RC									
150.8													
5.4	End of Borehole Borehole dry before coring at 2.0 m.												

TRANFT0801235AA

1 OF 1

METRIC

ORIGINATED BY GJ

COMPILED BY WC

CHECKED BY ZO

+³, ×³: Numbers refer to Sensitivity



TRANETOB01235AA

RECORD OF BOREHOLE No P1

1 OF 1

METRIC

GWP 339-00-00 LOCATION Sta: 21+442 ; 3.0 m Rt C/L Hwy 17 ORIGINATED BY GJ
DIST HWY HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
DATUM Geodetic DATE 3/25/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES								
155.9 0.0	GROUND SURFACE												
	Straight Auger												
153.6 2.3	End of Probe Hole Auger refusal @ 2.3 m. Water level @ 2.0 m (not stabilized*) upon completion. Hole caved-in @ 2.1 m upon completion.												

+³ ×³ : Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE



TRANETOB01235AA

RECORD OF BOREHOLE No P2

1 OF 1

METRIC

GWP 339-00-00 LOCATION Sta: 21+415 : 5.0 m LI C/L Hwy 17 ORIGINATED BY GJ
DIST HWY HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY WC
DATUM Geodetic DATE 3/25/2010 CHECKED BY ZO

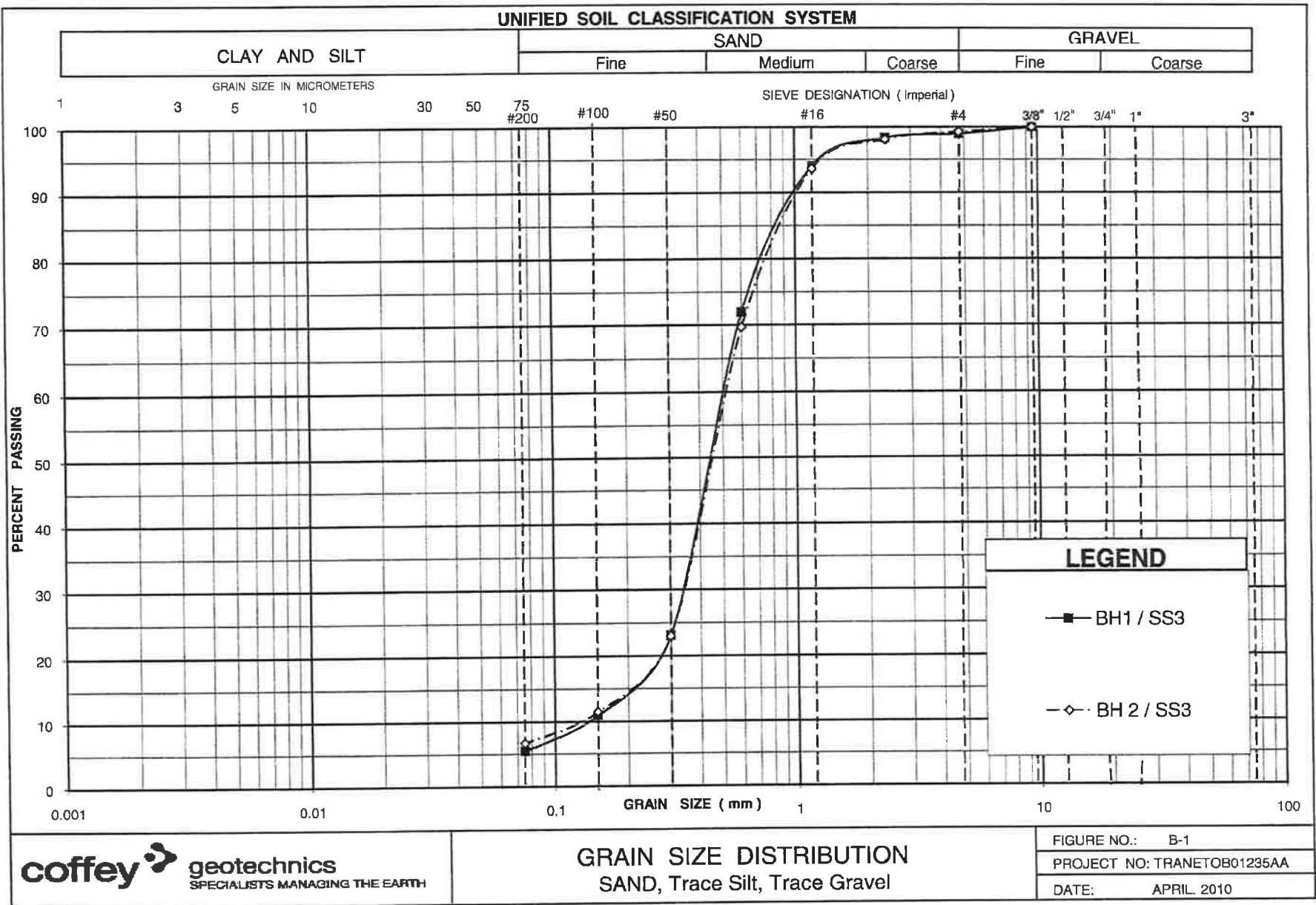
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
FLEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100			
156.0 0.0	GROUND SURFACE						156										
	Straight Auger						155									Auger grinding hard from 1.1 to 1.5 m	
							154										
152.6 3.4	End of Probe Hole. Hole was dry upon completion.						153										

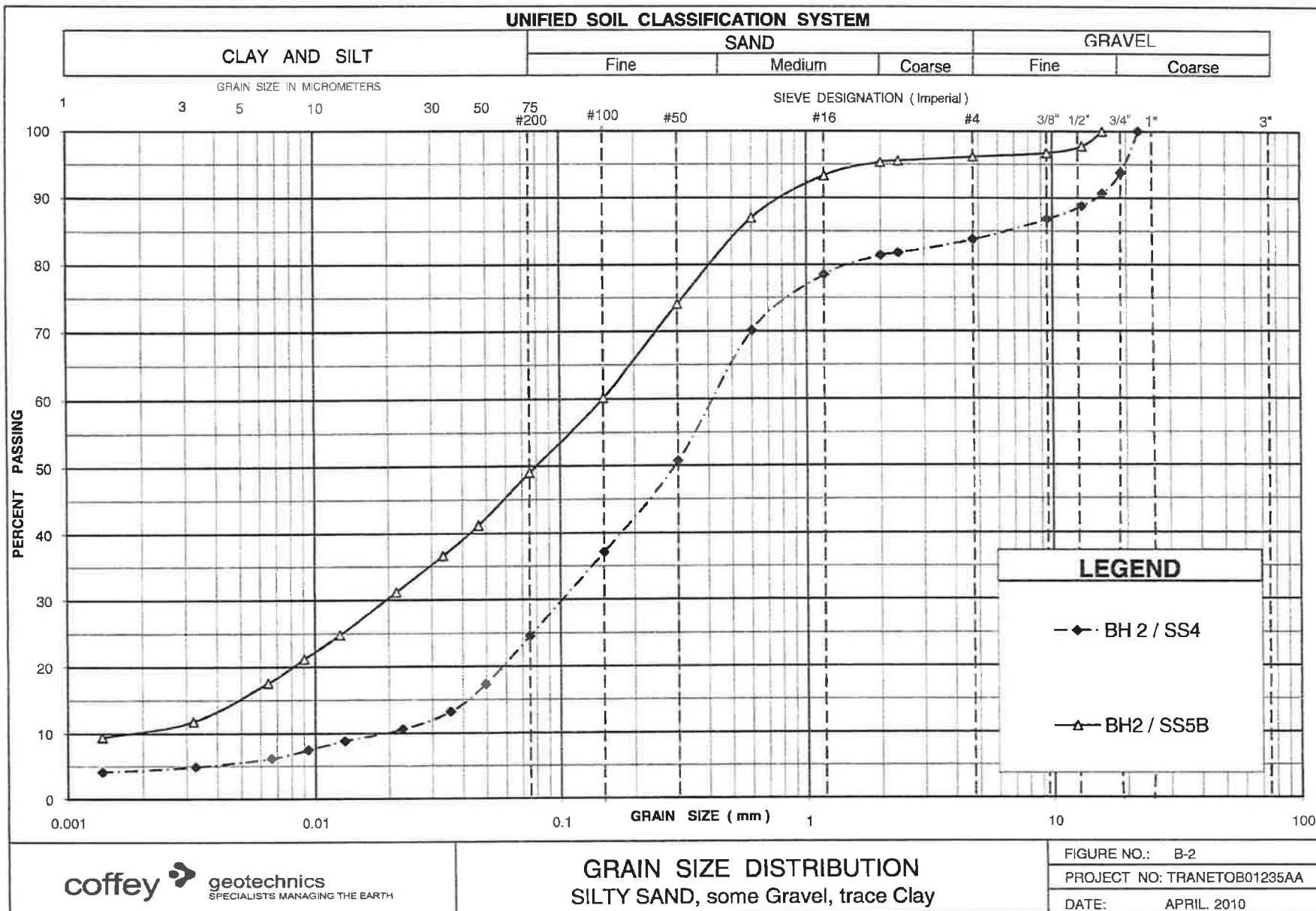
+ 3, X 3; Numbers refer to
Sensitivity

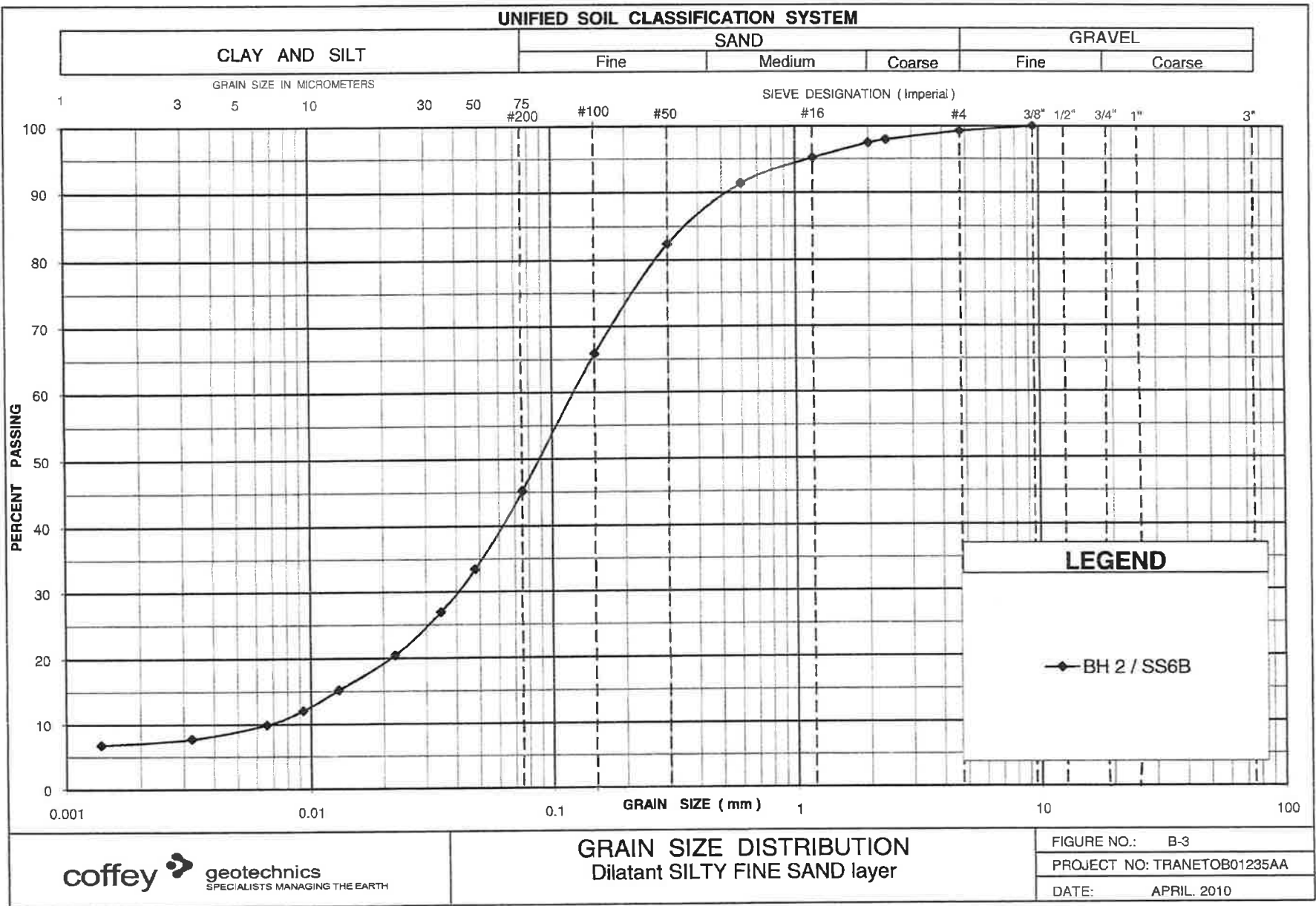
20
15 5
10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results







Appendix C

Site Photographs



Figure C1 – Left (North) side of Highway 17, looking East



Figure C2 – Boulders along the shoreline of Ottawa River near project site

Appendix D

Rock Core Photographs



Figure D1 – Rock core sample retrieved from Borehole 101



Figure D2 – Rock core sample retrieved from Borehole 102

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

U_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	$^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$^\circ$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	W_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ'	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED WATERMAIN AND
SANITARY SEWER REPLACEMENT,
HIGHWAY 17, TOWN OF MATTAWA,
ONTARIO, G.W.P. 339-00-00
GEOCRES 31L-142**

LEA Consulting Ltd.

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**FOUNDATION DESIGN REPORT
PROPOSED WATERMAIN AND REPLACEMENT OF SANITARY SEWER
HIGHWAY 17, TOWN OF MATTAWA, ONTARIO
G.W.P. 339-00-00**

5 DISCUSSION & RECOMMENDATIONS

5.1 General

As part of the rehabilitation of Highway 17, creek crossings of a watermain and a sanitary sewer, located about 75 m west from the junction of Highway 17 and Brook Street on both south and north sides of the highway in the Town of Mattawa, are proposed. The creek flows under the highway via an existing concrete box culvert which is skewed to the centerline of the highway at an angle of 36 degrees. The dimensions of the existing culvert are 1.95 x 1.2 x 30.5 m (width x height x length). The project entails the installation of a 10.5 m long, 200 mm diameter PVC (polyvinyl chloride) pipe on the north side of the Highway 17 for the sanitary sewer and a similar size PVC pipe for the watermain on the south side of the highway, under the existing culvert. The existing culvert maintains the creek flow at Station 21+425, towards Ottawa River. The proposed watermain and sanitary sewer locations, size, invert elevations and the approximate cover thickness below the existing road grade are summarized in Table 5.1.

Table 5.1 Proposed Watermain Crossing

	Station	Proposed PVC Pipe Diameter (mm)	Proposed Pipe Length (m)	Approximate Proposed Liner Invert Depth from the road grade / Elevation (m)*	Approximate Earth Cover Thickness (m) above obvert elevation (m)*
Watermain	21+419 to 21+430 (south side of the highway)	200	10.5	2.9/153.2	2.5
Sanitary Sewer	21+429 to 21+440 (north side of the highway)	200	10.5	3.2/152.9	2.8

*Assuming a 0.4 m diameter steel liner plate

Two sampled boreholes and two probe boreholes (i.e. Boreholes P1 and P2 drilled to check auger refusal depth without sampling and in-situ testing) were advanced at the site. Boreholes 1 and 2 encountered, below an about 0.8 m thick pavement structure, sand and silty sand deposits to the bedrock surface at depths of 2.9 and 4.3 m or El. 153.1 and 151.5 m. The sand and silty sand overburden at the site contains cobbles and boulders below depths of about 2 m or El. 154.0 m (Borehole 1) and 153.7 m (Borehole 2). Boreholes P1 and P2 encountered auger refusal at depths of 2.3 m and 3.4 m or El. 153.6 and 152.6 m, respectively, probably on boulders, overlying the bedrock.

It is our opinion that the groundwater level at the time of our investigation was at about El. 154.5 m similar to the creek water level. It should however be pointed out that the groundwater is subject to seasonal fluctuations and fluctuations in response to major weather events and water level in the water course.

5.2 New Watermain and Sanitary Sewer Construction Options

The most common and cost effective method is normally an open-cut excavation, provided that some possible disruption in traffic would be acceptable. In this particular case, however, as the existing culvert will need to be at least partially removed to effect the construction, tunnelling would appear to be an attractive solution. For both options, the subsurface conditions are considered unfavourable as:

- the existing watercourse at the time of our investigation appeared to be fast flowing;
- the groundwater table was high but may fluctuate;
- the overburden consists of granular soils (which are pervious and will need to be stabilized by dewatering), as well as containing frequent cobbles and boulders, based on the borehole data;
- the bedrock is close to the proposed pipe invert levels which will render dewatering difficult; it may also interfere with tunnelling operations, if encountered during tunnelling;
- the bedrock is relatively competent in one borehole and very weak and friable in the other (i.e. variable bedrock characteristics).

For these reasons, an open cut excavation may be difficult, based on practicability of construction including removal and reconstruction of the existing culvert and traffic disruptions. Tunnelling may therefore, have to be resorted to, as is being planned by the designers. However, a suitable tunneling method may need to be implemented with due consideration to the presence of flowing sand and silty sand with cobbles and boulders underlying the site and also possibility of intersecting the bedrock locally with the proposed pipe alignment, together with the potential difficulty of dewatering of the sand and silty sand near the bedrock surface.

The following are possible typical options which are normally considered for these types of project:

- Option 1: Open Excavation with Shoring
- Option 2: Jack and Bore
- Option 3: Tunnelling with Hand Mining methods
- Option 4: Pipe Jacking with TBM
- Option 5: Micro-Tunneling
- Option 6: Pipe Ramming
- Option 7: Horizontal Directional Drilling (HDD)

The selection of the preferred option will depend, among other factors, the construction cost, practicability of construction, risk of ground subsidence, scheduling, etc.

A classification of the soil for tunnelling purposes, as commonly used in Ontario, is given in Appendix G.

As shown in the design drawing provided to us by LEA (see Appendix F), the invert elevation of the PVC pipes (i.e. sanitary sewer and watermain) will be at about El. 152.9 to 153.4 m. As a steel liner/casing is

required for tunnelling, the liner/casing invert elevation would be at least 0.15 m lower than the proposed PVC pipe. These elevations are below the auger refusal depths (probably on boulders) of Boreholes 1, 2 and P1. Based on the borehole results, the new watermain proposed on the south side of the existing highway will be typically constructed through a 'bouldery' and 'flowing' (see Appendix G) sand near the bedrock surface (i.e. the assumed invert elevation of the liner is about 0.2 m above the bedrock). The borehole results also show that the proposed sanitary sewer, which will be constructed on the north side of the road, will extend to the silty sand overburden with a somewhat less chance of encountering boulders, cobbles and bedrock than the proposed watermain based on the findings of our investigation. The water bearing silty sand in Borehole 2 can also be classified as a 'flowing' as well as 'bouldery' in accordance with the classification given in Appendix G.

The following table represents an overview of the seven aforementioned methods and is intended only to assist the designers in their choice of most suitable method. However, contractors may come up with more suitable options or variations of such methods.

Table 5.2 Summary of Construction Options

Construction option	Comments	Recommendations
Open Excavation with Shoring	Removal and reconstruction of the existing culvert will be required. Difficulty with creek flow diversion during the construction. Difficulties with dewatering due to the shallow bedrock depth.	May be objectionable based on the dewatering requirements and traffic disruptions.
Jack and Bore	Dewatering is required before and during the excavation. Difficulty with bouldery ground. Difficulty with dewatering due to the shallow bedrock depth.	May be objectionable based on the dewatering requirements and prevailing bouldery ground condition. Larger size liner can be considered as an alternative but will have more chance to intersect the bedrock during tunnelling.
Tunnelling with Hand Mining methods	Tunnel diameter is less than 1.0 m which is the minimum required diameter for this method.	Not recommended based on the size of tunnel and the need for extensive dewatering.
Pipe Jacking with TBM using earth pressure balance	Considered uneconomical	Considered objectionable based on cost.
Micro-Tunneling	Considered uneconomical	Would be objectionable based on cost.

Construction option	Comments	Recommendations
Pipe Ramming	<p>Dewatering is required before and during the excavation. Difficulty with bouldery ground. Difficulty with dewatering due to the shallow bedrock depth.</p> <p>Vibration may be objectionable to residents and may damage the existing culvert.</p>	<p>May be objectionable based on the dewatering requirements and prevailing bouldery ground condition.</p> <p>Ramming induced noise and vibrations and may not be acceptable within the commercial and residential area.</p> <p>Larger size liner can be considered as an alternative but will have more chance to intersect the bedrock during the tunnelling.</p>
Horizontal directional drilling (HDD)	<p>HDD can be used in the tight space such as an urban area without disruption of traffic.</p> <p>However, the length and angle of entrance and exiting may be too steep for this project.</p> <p>As well, loss of fluid may occur. Difficulty with bouldery ground. Hard rock drill bit equipped HDD can be considered.</p>	<p>May be objectionable based on the presence of cobbles and boulders.</p>

Details of each proposed options are discussed below.

5.2.1 Open Cut Construction

The inverts of the pipes are expected to be at El. 152.9 to 153.2 and since the top of the roadway embankment is at about El. 156 m, an approximately 3 m deep excavation will be required, on each side of the road.

The conventional open cut excavation technique will probably require a vertical excavation with the ground supported by temporary shoring due to limited space beside the road. For open cut construction, a portion of the existing culvert will need to be removed and subsequently re-constructed after the installation of the pipe. Temporary creek water diversion is also required during the construction. Shoring is normally provided by using a tight steel interlocking sheet pile enclosure or soldier piles and lagging. The sheet piles will need to be installed deep enough to resist the expected loading conditions and reduce hydraulic head as well. The installation of sheet piles or soldier piles in this case will be difficult to construct due to the boulders and cobbles as well as the shallow bedrock throughout much of the site. For example, the holes for the support of soldier piles will need to be installed into the bedrock, which will increase cost, especially where the bedrock is relative sound. Similarly difficulties will be experienced when driving sheet piling due to boulders and sheet pile may not be able to extend into the bedrock. This may make the use of a sheet pile enclosure impractical.

From the findings of our investigation, it appears that the pipe invert will be in the water bearing sand and silty sand, which must be properly dewatered and stabilized before installing the pipe. It is recommended that the dewatering be effective to at least 0.5 m below the bottom of the excavation (i.e. water level should be lowered to not less than 0.5 m below the bottom of the excavation level to stabilize the sandy soil). Normally well-pointing and/or properly filtered deep wells are utilized for dewatering. In this instance, however, difficulties will arise due to the shallow bedrock and boulders at the site. A further discussion of dewatering methods is given in Section 5.6 of this report.

After dewatering, the stabilized undisturbed sand and silty sand would provide a suitable support for the pipe and normal Class 'B' bedding can be used. The pipe will likely be installed in a larger diameter steel casing due to the anticipated compaction efforts required for the support of the reconstructed part of the culvert. The minimum thickness of the bedding underneath the pipe should be in accordance with applicable standards but not less than 200 mm. The bedding should be extended to above the invert of the pipe as per specifications. The bedding should consist of well-graded materials such as Granular 'A' materials to prevent the sand and silty sand from infiltrating into the voids of a poorly graded bedding material. As an alternative where the bedrock is found at a shallow depth beneath the pipe, the overburden can be excavated to the surface of the bedrock and the pipe can be supported on weak concrete immediately above the bedrock (i.e. overburden replaced with concrete).

It should also be pointed out that rock excavation will be required if the proposed pipe alignment intersects bedrock during the construction.

We recommend that these difficult conditions be fully explained to the Contractor by means of an NSSP.

5.2.2 Tunnelling Under the Roadway

Tunnelling procedures depend upon a number of factors, the most important of which are the groundwater conditions and the soil type through which the tunnel must pass. Table in Appendix G presents the classification of the ground according to terminology used by tunnel laborers (commonly known as the Tunnelman's Ground Classification System), the soil types and the probable tunnel working condition for each classification.

According to the Tunnelman's Ground Classification System, the sand and silty sand encountered at the level of the proposed watermain and sanitary sewer are classified as combination of 'bouldery' and 'flowing'.

In bouldery ground, problems may be encountered in advancing the tunnel. Hand mining or blasting ahead of tunneling equipment may be required depending on the site conditions (i.e. size and distribution of boulders). Also, hard rock drill bit equipped horizontal directional drilling can be considered. Alternatively, large size of liner/casing (i.e. larger than the expected boulder size) which may enable relatively smaller size boulders to be dealt with, can be considered.

Flowing ground condition is often caused by seepage pressures. For sand and silty sand below the water table, available data and experience suggests the material will flow into the tunnel if the effective grain size D_{10} is in excess of about 0.005 mm. In this case, the sand and silty sand encountered in the boreholes falls into this category. These types of soils are considered problematic in tunneling, even when closable face shields or tunnelling machines are used. Most flowing ground can be transformed into more stable ground

by drainage (dewatering), or by grouting. Grouting at this site will cause some environmental issues if chemical grouting is selected because of the proximity of the site to Ottawa River.

The following is a discussion of various tunnelling methods with the above considerations in mind.

5.2.2.1 Jack and Bore Method

Jack and bore method forms a borehole from a drive shaft to a reception shaft by means of rotating cutting head. Spoil is transported back to the drive shaft by helical auger flights rotating inside a liner/casing. The liner/casing is jacked in place simultaneously with the augering operation. After the installation of the liner/casing, the utility pipe is installed inside the liner/casing and the gap between the liner/casing and the pipe is grouted or sometimes inner pipe is fixed by spacer inside liner/casing. The size of the liner/casing must be at least about 200 mm larger than the largest outside diameter of the carrier. The maximum casing diameter used in this operation is limited to about 1.3 m for most contractors in Ontario. Instead of steel liner/casing, plastic liner/casing can be used if its physical properties are sufficient. But in this instance the use of a plastic liner is considered unsuitable.

Based on the results of the investigation, the proposed watermain and sanitary sewer will be constructed primarily through the water bearing sand and silty sand deposits with boulders. These subsurface conditions are problematic for a jack and bore operation because the sand and silty sand deposits are below the groundwater table (i.e. flowing soil condition) and also bouldery. For the case of flowing ground condition, dewatering and/or grouting is normally performed prior to proceeding with the boring. However, dewatering at the site will be difficult due to the shallow bedrock at the site. For bouldery ground condition, blasting or hand mining ahead of tunneling equipment will be required. Alternatively, relatively larger size of liner/casing (i.e. larger than the expected boulder size) can be also considered for bouldery ground condition but it will increase the chance to intersect bedrock especially on the south side of the road during the proposed watermain installation.

Thrust block needs to be installed to distribute the loads in a uniform manner to prevent the misalignment of liner/casing.

We recommend that the feasibility of this method be discussed with a specialist contractor.

5.2.2.2 Tunnelling by Hand Mining

In a hand mining operation, the excavation of the tunnel is accomplished manually and a temporary ground support system is required during the operation. The temporary ground support system can be steel or concrete segmental liner or steel ribs with wood lagging. Groundwater control will be required to minimize water leakage into the tunnel. Workers are required inside the tunnel to perform the excavation and/or spoil removal. The excavation will be accomplished by hand mining with the assistance of small excavation tools.

With this method, control of alignment and grade is accomplished by overmining in the direction of the change and the pipe will move into the overmined area as it is pushed forward.

The main advantage of this technique is that it can be economical, and that large boulders can be removed.

This technique is limited by the difficulty of controlling the grouting quality and its impact to the environment. Higher risk of ground subsidence may also be encountered with anticipated settlements could be in the order of 20 to 50 mm. In addition, with this method, the tunnel project will take a longer time to complete.

In order to accommodate the working space for workers, the minimum diameter of the tunnel using hand mining is about 1 m.

Since this method requires prior and during construction dewatering, similar to the methods discussed before, as well as larger diameter tunnel, it is not recommended for this project.

5.2.2.3 Pipe Jacking with TBM

The method of pipe jacking with TBM installs a prefabricated pipe through the ground from a drive shaft to a reception shaft. The pipe is pushed by jacks located in the drive shaft and the jacking force is transmitted through the pipe to the face of the excavation. The excavation with this method is accomplished by a TBM (Tunnel Boring Machine) and the spoil is transported out of the jacking pipe and shaft manually or mechanically. Typically, pipe jacking with TBM is applicable to tunnels with relatively larger diameter (e.g. 1 to 3 m).

This tunneling method is so versatile that it can be executed with virtually any ground conditions (except large boulders) with adequate precautions. In unstable soil conditions such as the present case (i.e. wet sand), an earth Pressure Balance Machine (EPBM) or slurry shield TBM is required to counterbalance the ground and hydrostatic pressures and to minimize ground subsidence. With EPBM, dewatering of the wet sand would be less stringent.

The main disadvantage of this method is its high capital and set-up costs. This technique also requires good operator skill and experience. In addition, this method has a very tight alignment and grade tolerance since the permanent lining (the pipe segments) is being installed during the tunnel operation. If large boulders are encountered, hand-mining may have to be employed which could lead to project delay and extra costs.

With pipe jacking and the use of EPBM, if operated properly, the maximum ground settlement is expected to be minimal in the order of 10 to 35 mm, which is considered acceptable under the highway.

In our opinion this method will not be cost effective in the present case, and is therefore not recommended based on cost factor.

5.2.2.4 Micro-Tunnelling

This technique is the improvement of the pipe jacking technique with TBM. It is a remotely controlled, guided pipe-jacking process that provides continuous support to the excavation face. The guidance system usually consists of a laser mount in the drive shaft, communicating a reference line to a target mounted inside the tunnelling machine. This technique provides ability to control excavation face stability by applying mechanical or fluid pressure to counterbalance the earth and hydrostatic pressures.

The main advantage of this technique is that it is sophisticated and will most likely complete the project in shorter time. It will also complete the tunneling operation with even less ground subsidence, if operated properly, which is estimated to be about less than 15 mm. It is however ineffective when tunneling through boulder ground.

The main disadvantage of this method is its very high cost and it is therefore not recommended based on economics and due to the presence of boulders.

5.2.2.5 Pipe Ramming

In a pipe ramming operation, a pneumatic ramming tool attached to the rear of a steel casing drives the casing into the ground with repeated percussive blows. The installed pipe usually has an open end that allows the soil to enter the casing during the installation. The spoils inside the casing can be removed either during or after the installation (after installation is preferred in some cases to prevent to create voids and slumps during the ramming process), by auger, compressed air or water jetting. In addition, pipe ramming can accommodate boulders and rock fragments as large as the liner/casing itself during the process. After completing the installation, PVC pipe will be installed inside the liner/casing and the gap between the liner/casing and the pipe will be grouted or PVC pipe can be fixed inside liner/casing by spacer.

This method of installation is mostly used on pipes between 0.1 and 1.4 m in diameter and for pipe installation over relatively short distances (i.e. less than 45 m). Although longer distance (up to 100 m) has been achieved in favorable ground condition (i.e. soft to firm clayey soil), the length of the tunnel installed by pipe ramming is limited by the ground conditions. Based on the investigation results, the proposed watermain and sanitary sewer will be advanced mainly through sand and silty sand with boulders. These subsurface conditions are probably less feasible for a pipe ramming operation because this method is difficult to apply to the granular soils below the groundwater table (unless it is sufficiently dewatered). As well, boulders encountered during the investigation are not a favorable condition for pipe ramming (especially for the larger boulders). Alternatively, relatively larger size of liner/casing (i.e. larger than the expected boulder size) can be also considered for bouldery ground condition but it will increase the chance to intersect bedrock especially on the south side of the road during the proposed watermain installation. Another disadvantage of pipe-ramming is the vibrations created and noise generated which may be objectionable within the town. Also vibrations may cause settlements due to liquefaction of the silt and the fine sand, as well as causing damage to the existing culvert. While it is unlikely to be suitable, the feasibility of this method can be discussed with a specialist contractor.

5.2.2.6 Horizontal Directional Drilling (HDD)

This method consists of pilot boring, back reaming and pipe pulling. Drilling begins with a small diameter pilot hole along a designated alignment, using flexible drill rods with remote controlled steering system. After the pilot boring, a back reamer is installed and drilled back through the pilot hole to achieve the required diameter for the pipe to be installed. Typical ratio of diameter of reamer to pipe is 1.3 to 1.5. Special drilling fluid is used to prevent the collapse of borehole as well as providing a lubricant for the drilling and flushing spoils. Selection of reamer and drilling fluid highly depends on the ground conditions. Hard rock drill bit may need to be considered for this case.

The feasible diameter of pipe or tunnel in this method is 0.5 to 1.5 m and drive length can be up to 2000 m. Based on the results of the investigation and the proposed depth of the pipe, the proposed watermain and sanitary sewer will be constructed through the sand and silty sand with boulders. These subsurface conditions may be least feasible for a horizontal directional drilling operation due to the presence of

boulders, even if the drilling mud can support the soil and prevent a 'flowing' ground condition into the hole during the drilling.

Reference should be made to OPSS 450 for this method, as well for related specifications.

Details should be discussed with a specialist contractor to determine its feasibility, however, it is our opinion that this option is not feasible for this project due to the prevailing ground conditions.

5.2.3 General Comments

From the above discussion, it is obvious that the prevailing subsoil conditions are favourable neither for an open cut construction nor for tunneling.

We recommended that these and other methods be discussed with a specialist contractor(s).

5.3 Settlements

Settlement caused by tunnelling is the aggregate of two basic types of settlement, which consist of ground loss or 'immediate' settlement, and consolidation settlement.

The 'immediate' settlement is the direct result of the movement of ground into the tunnel heading. The factors which influence the magnitude of immediate settlement due to tunnelling include soil strength and stiffness, the method of tunnelling and the quality of tunnel operations (including the method of handling localized factors such as encountering boulders).

Consolidation settlement is not considered for this project.

Settlement due to tunnelling is difficult to estimate. Generally, with good workmanship settlements over the tunnel centerline should not exceed 25 mm and settlements beyond the tunnel centerline would decrease to zero. In addition to these settlements, settlements due to dewatering will need to be taken into consideration. The magnitude of settlement due to dewatering will depend on the depth of the lowering the water table. For example, assuming a uniform 2 m lowering of groundwater level from about El. 154 m to El. 152 m can be expected to cause a settlement of the order of 20 mm. This would decrease to less than 5 mm say about 30 m beyond the limits of dewatering. A settlement of this magnitude is believed not to be detrimental to the existing culvert structure and pavement of the existing highway. However, the settlements should be further looked into when the details of construction and dewatering are known. In addition, the possible effects of dewatering settlement on any adjacent structures and underground services should also be looked into.

5.4 Design Parameters

The following bulk units for materials above the tunnel crown can be assigned.

Granular pavement fill:	21.0 kN/m ³
Sand:	20.0 kN/m ³
Silty Sand:	19.0 kN/m ³

For the soils surrounding the tunnel, the estimated coefficient of lateral earth pressure at rest, K_0 , could be taken as 0.5.

5.5 Construction Considerations

For the open cut construction, braced excavation may be required due to the space limitation immediately beside the existing highway. Removal and reconstruction of the existing culvert and creek water diversion are also necessary for open cut construction. Since space is unlikely to be sufficient to permit an open-cut excavation, the open cut excavations, as well as excavations for tunnelling shafts will need to be carried out inside a temporary shoring enclosure. The depth of overburden excavation at the shaft locations adjacent to the existing culvert location will likely be about 3 to 3.5 m. The excavations are expected to be carried through a compact to dense sand and silty sand with boulders overlying bedrock. Since groundwater table at the time of our investigation was assumed similar to the creek water level at about El. 154.5 m, dewatering will be required to facilitate the excavation and to preserve the load carrying capability of the soil. Entry and exit seals at the shaft wall should be provided to prevent to inflow of groundwater, soil, slurry and lubricants.

The design of the braced excavations/shaft should take into account horizontal soil loads, hydrostatic water pressure (if any), any surcharge due to construction loadings and/or adjacent embankment and any traffic loads.

Shoring is normally done by soldier piles and lagging or sheet piling. Difficulties are anticipated in either case due to boulders. Auger holes for soldier piles will likely require socketing into the bedrock which may be difficult where the bedrock is relatively intact and sound. Similarly sheeting may be difficult due to boulders and may not penetrate the relatively sound bedrock.

The shoring system should be designed by a Professional Engineer experienced in this type of work. The following table presents recommended parameters for temporary shoring design.

Table 5.5 Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Granular Pavement Fill	0.31	0.47	3.25	21.5
Sand	0.32	0.48	3.12	20.0
Silty Sand	0.33	0.50	3.00	19.0
Extremely Weathered Rock	0.24	0.38	4.20	22.0

Construction must be carried out in accordance with the latest edition of the Ontario Health and Safety Act and its Regulations (i.e. Occupational Health and Safety Act O. Reg 213/91). The soils encountered at the site can be classified as Type 3 soil above the groundwater level and Type 4 soil below the groundwater level, unless they were properly dewatered.

The protection system for the retention of the existing highway and/or the tunnel shafts should be designed as per SP539S01 – Protection Schemes. Due to the proximity of the potential tunnel shafts to the highway, the performance level of the protection system is recommended as Level 2, i.e. lateral movement of any portion of the protection system shall not exceed 25 mm.

As well, SP902 S01 – Excavation and Backfilling to Structures, where applicable, must be adhered to.

Depending on the construction method selected for the proposed watermain and sanitary sewer, it may be necessary to provide a Non-Standard Special Provision (NSSP) in the contract documents to specify the requirement of the selected construction method.

Consideration should be given to provide an NSSP to alert the contractor of the presence of cobbles and boulders in overburden sand and silty sand deposit and closeness of bedrock to the propose tunnel invert elevation, as well as dewatering requirements.

5.6 Dewatering

The design of the dewatering system will depend on the required draw-down for the selected construction method.

With the prevailing subsurface conditions, the most expedient method of dewatering in our opinion would normally be well-pointing. Well-points are usually effective in lowering water levels to about 5.0 m below the header pipe elevation. If well points can be established at about El. 156 m in single rows along the road as well as some distance along the road out of the tunnel alignment, a well designed well-point system can lower the water table to about 1.0 m below the pipe invert (i.e. to about El. 151.5 m). However, in this instance it is unlikely that it will be possible to establish well points due to the shallow bedrock. In this case, properly filtered deep wells can be considered as an alternative or combination of both well pointing and deep wells can be considered. This aspect should be discussed with a specialist contractor.

A hydrological study along with a pumping test may be needed to be conducted to verify the success of a well-designed system, as well as its effect in the surrounding areas, including the structures and water wells. It is likely that permission will have to be sought from the Ministry of the Environment (MOE) for water withdrawal in excess of 50,000 litres/day. Based on the grain-size distribution curves presented in Appendix B and the Allen Hazen relationship for D_{10} , for preliminary estimating purposes, the coefficient of permeability, k , of the sand and silty sand can be taken as being about 4×10^{-2} cm/sec and 4×10^{-4} cm/sec, respectively.

The effects of dewatering (e.g. settlements) on the adjacent structures (including the existing buildings), underground services, as well as the road should be included in the hydrological study.

5.7 Instrumentation and Monitoring

It is recommended that if tunneling is utilized, instrumentation in the form of settlement and heaving monitoring be carried out for the watermain and sanitary sewer. In this case, surface settlement markers (e.g. surveyors nails) could be installed in the paved portion, while settlement rods can be used in the road shoulder. The quantity of excavated materials will be compared with the theoretical volume of excavation in order to assess the risk of over-excavation. Such monitoring is necessary to confirm that any settlement/movement associated with the proposed construction would be within tolerable limits.

5.8 Soil Disposal and Drilling Fluid Recycling

The excavated materials from the construction can be stockpiled and checked for contamination prior to removal/disposal off-site, in order to determine which disposal option is best suited for the excavated materials.

The excavated materials from tunnel construction are suspended and removed by drilling fluid. This drilling fluid with spoils can be extracted by vacuum machine and also can be recycled.

It is recommended that a programme of geotechnical/material inspection and testing be carried out during the construction phase of the project to confirm that the conditions exposed in the excavations are consistent with those encountered in the boreholes and the design assumptions, and to confirm that the various project specifications and materials requirements are being met.

6 CLOSURE

The Limitations of Report, as quoted in Appendix I are an integral part of this report.

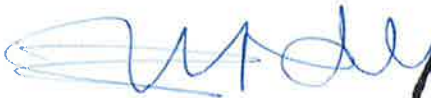
For and on behalf Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



Ramon Miranda, P.Eng.

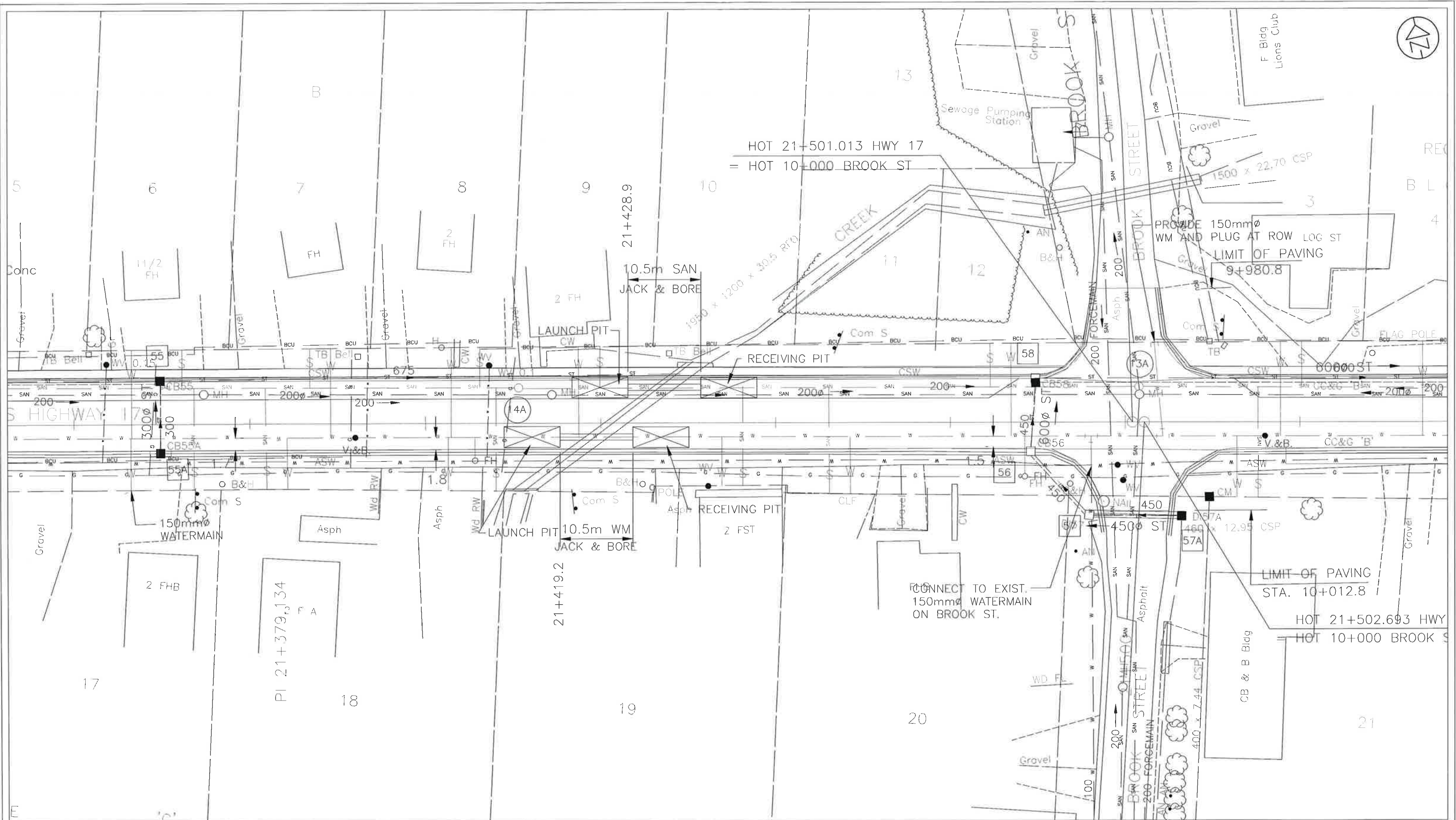


Zuhtu Ozden, P.Eng.



Appendix F

Proposed Tunnel Alignment Drawings



NOTES:

1. This drawing is enlarged from contract drawing #50 prepared by LEA Consulting Ltd.



drawn	PHK
checked	RM
approved	ZO
date	Aug. 16, 2010
scale	As shown

coffey  **geotechnics**
SPECIALISTS MANAGING THE EARTH

CONT:	2010-5124	WP:	339-00-00
project:	PROPOSED WATERMAIN AND SANITARY SEWER REPLACEMENT HIGHWAY 17, TOWN OF MATTAWA, ONTARIO		
title:	SITE PLAN		
project no:	TRANETOB01235AA	figure no:	F-1

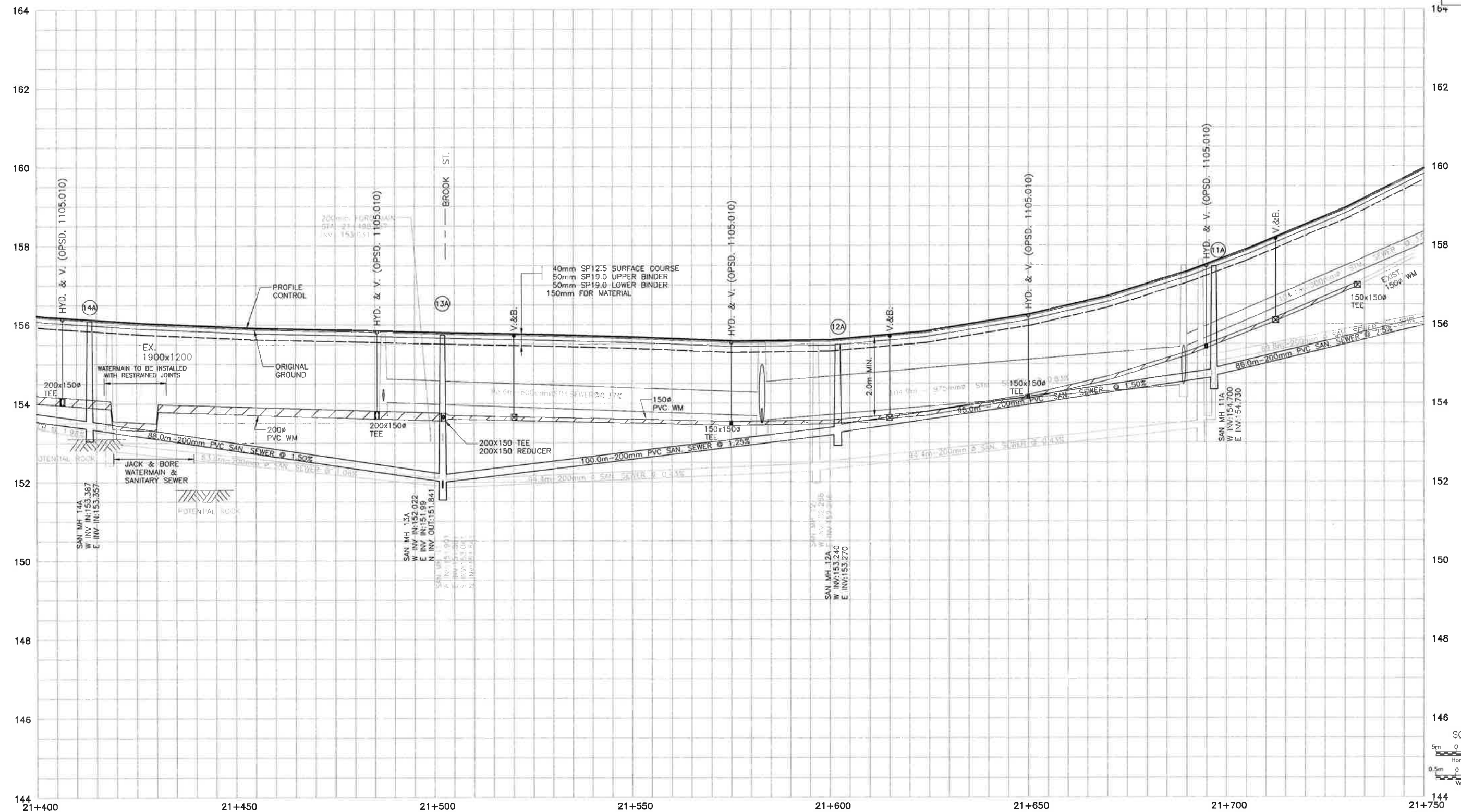
A. FDR = IN-PLACE FULL DEPTH RECLAMATION OF ASPHALT PAVEMENT AND UNDERLYING GRANULAR

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT	2010-5124
WP	339-00-00

SHEET
51

FIGURE
F-2



DRAWING NAME: F:\GEOTransp\ACTIVE PROJECT\2008\01235 - TRAN0801235A - Hwy17, 656 Metcova\Drawings\Received Drawings\Highway 17\Aug18-2010\8628 17 PF-10.dwg (LAYOUT1)
PLOT DATE: 2010/08/17 2:02:23 PM

Appendix G

Tunnelman's Ground Classification and Probable Working Conditions

Tunnelman's Ground Classification and Probable Working Conditions

Soil Classification	Representative Soil Types	Tunnel Working Conditions
Hard	Very hard calcareous clay; cemented sand and gravel	Tunnel heading may be advanced without roof support
Firm	Loess above GWT; Various calcareous clay with low plasticity	Tunnel heading may be advanced without roof support and the permanent support can be constructed before the ground will start to move
Slow Ravelling and Fast Ravelling	<i>Fast Ravelling</i> occurs in residual soils or in sand with clay binder below the GWT. Above the GWT, the same soils may be <i>Slowly Ravelling</i> or even Firm	Chunks or flakes of material begin to drop out of roof or the sides sometime after the ground has been exposed. In <i>Fast Ravelling</i> ground, the process starts within a few minutes; otherwise it is classed as Slow Ravelling
Squeezing	Soft or medium-soft clay	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel (may not be noticed in tunnel but cause surface subsidence)
Swelling	Heavily pre-compressed clays with a plasticity index in excess of about 30; Sedimentary formations containing layers of anhydrite.	Like squeezing ground, moves slowly into tunnel, but the movements is associated with a very considerable volume increase in the ground surrounding the tunnel.
Cohesive Running and Running	<i>Cohesive running</i> occurs in clean, fine moist sand <i>Running</i> occurs in clean, coarse or medium sand above the GWT	The removal of the lateral support of any surface rising at an angle of more than about 34° to the horizontal is followed by a 'run,' whereby the material flows like granulated sugar until the slope angle becomes equal to about 34°. If the 'run' is preceded by a brief period of raveling, the ground is called <i>Cohesive Running</i>
Very Soft Squeezing	Clays and silts with high plasticity index	Ground advances rapidly into the tunnel in a plastic flow
Flowing	Any ground below the GWT that has an effective grain size in excess of about 0.005 mm	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides but also through the bottom. If the flow is not stopped, it continues until the tunnel is completely filled.
Bouldery	Boulder glacial till; rip-rap fill; some land slide deposits, some residual soils. The matrix between boulders may be gravel, sand, silt, clay or combinations of thereof.	Problems occurred in advancing shield or in forepoling; blasting or handmining ahead of machine may become necessary.

Appendix H

List of Standard Specifications

List of Standard Specifications

OPSS

- 450 CONSTRUCTION SPECIFICATION FOR PIPELINE AND UTILITY
INSTALLATION IN SOIL BY HORIZONTAL DIRECTIONAL DRILLING
(not included)

SPECIAL PROVISIONS

- SP539S01 Protection Schemes
- SP902 S01 Excavation and Backfilling to Structures

Appendix I

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.