



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**TROUT LAKE ROAD OVERPASS
FUTURE HIGHWAY 11/17
CITY OF NORTH BAY
GWP 5748-04-00, SITE 43-350
DISTRICT 54, SUDBURY, ONTARIO**

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**PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT**

for
Trout Lake Road Overpass
Future Highway 11/17
City of North Bay
GWP 5748-04-00
District 54, Sudbury, Ontario

1. INTRODUCTION

This report summarizes the results of a preliminary foundation investigation carried out for construction of an overpass on the alignment of Trout Lake Road at the Future Highway 11/17 in North Bay, Ontario. The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The overpass construction is contemplated for the Future Highway 11/17 fully controlled access freeway from 1.5 km south of the South Junction of Highway 17 on Highway 11 northerly for 5.4 km within the City of North Bay.

The report provides preliminary subsurface information pertaining to the proposed overpass foundations and approaches within about 20 m of the abutments.

2. SITE DESCRIPTION AND GEOLOGY

The site is located on the Trout Lake Road alignment about 300 m north of the intersection with the existing Highway 11/17 in North Bay. The overpass to be erected will carry the Future Highway 11/17 traffic over Trout Lake Road. The alignment of the overpass is considered to be north-south.

Land use in the vicinity of the site comprises residential properties and community facilities. The proposed structure footprint is currently occupied by residences and their landscaped areas between Barker and Pearson Streets. Photographs of the site are provided in Appendix A.



The topography is relatively flat except where an unnamed creek runs along the northwest corner in a narrow 2 m deep valley. The ground within the right-of-way is covered with grass, large trees and bushes. The native soils are typically represented by sand.

The project site is situated in the Algonquin Highlands physiography region within the Canadian Shield. The typical rock type in the project area is highly metamorphosed metasedimentary migmatitic biotite gneiss of the Precambrian. The bedrock is at various depths, less than 10 m at the site.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out on April 18, 19 and June 22, 2007, comprising 7 boreholes drilled to depths of 0.6 to 6.1 m at the locations shown on Drawing TLR-1, appended. Access to the proposed locations of the foundation elements was limited due to existing houses, private facilities and large trees. Consequently, the boreholes were put down as close to the foundation alignments as possible and borehole TLR-7 that was planned at the south abutment, could not be drilled.

The conditions within 20 m of the abutments were only inspected visually and inferred subsurface changes noted, since boreholes were not requested by MTO for preliminary design within these limits.

The locations of and ground surface elevations at the boreholes were provided by Del Bosco Surveying Ltd. Most boreholes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. One of the holes was a test pit dug manually.

Representative soil samples were recovered at frequent depth intervals in the boreholes using a conventional split spoon sampler during drilling. Standard penetration tests were conducted to assess the strength characteristics of the substrata.



Groundwater conditions at the borehole locations were assessed during drilling by visual examination of soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes. The boreholes were backfilled with a bentonite/cement mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole and test pit abandonment procedures.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. In addition, six grain size distribution analyses were carried out on selected soil samples. The grain size distribution charts are presented in Figures GS-TLR-1 and GS-TLR-2; the results are also summarized on the corresponding Record of Borehole sheets.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classification, inferred stratigraphy, boundary elevations, standard penetration test data, groundwater observations, the results of grain size distribution analyses and moisture content determination.

The borehole locations and stratigraphic cross-sections prepared from the borehole data are shown on Drawing TLR-1.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial topsoil overlying sand. Cobbles and boulders were encountered in four boreholes. Bedrock was inferred at depths of 0.6 to 6.1 m below grade. The strata encountered are summarized below.

4.1 Fill

Localised fill was present surficially in borehole TLR1. Composed of silty sand, the fill was 1.4 m thick and very loose in relative density (SPT-N value of 1). The fill contained cobbles and was penetrated at elevation 208.8.



4.2 Topsoil

Surficial topsoil was present in boreholes TLR2 to TLR6 and test pit TLR8. The topsoil ranged in thickness from 0.1 to 0.8 m and was penetrated at elevation 208.5 to 210.5.

4.3 Sand / Gravelly Sand

Underlying the fill or topsoil at depths of 0.1 to 1.4 m (elevation 208.5 to 210.5) in boreholes TLR1 to TLR5 and test pit TLR8 was cohesionless sand. This stratum was 1.4 to 5.9 m thick where penetrated and very loose to dense, typically loose to compact (SPT-N values of 1 to 33 and resistance of 40 to 50 blows per 75 to 150 mm penetration). The moisture content of the sand varied between 5 and 18%. The stratum extended to depths of 1.5 to 6.1 m (elevation 203.8 to 209.0) in boreholes TLR1 to TLR5 and was not penetrated at 1.1 m depth (elevation 208.2) in test pit TLR8. It is noteworthy that cobbles and boulders were encountered in the sand in boreholes TLR1, TLR2, TLR4 and TLR8.

Directly beneath the sand at a depth of 2.2 m (elevation 208.2) in borehole TLR5 was gravelly sand. This unit was compact to dense, with a moisture content of about 10%. The gravelly sand had a thickness of 400 mm and was penetrated at 2.6 m depth (elevation 207.8).

The results of grain size distribution analyses conducted on six samples of the sand / gravelly sand are presented in Figures GS-TLR-1 and GS-TLR-2.

4.4 Bedrock

Bedrock was inferred by refusal at depths of 0.6 to 6.1 m (elevation 203.8 to 210.3) in all boreholes with the exception of test pit TLR8 terminated in the sand at a depth of 1.1 m (elevation 208.2). The bedrock surface typically slopes down in the northwest direction at a gradient of 2 to 4%.



4.5 Groundwater

In the course of the field work, groundwater was observed in four boreholes. In the process of augering, water was detected at depths of 2.1 to 3.0 m (elevation 207.2 to 207.8). Upon completion of drilling, groundwater was at depths of 2.7 to 3.3 m (elevation 206.9 to 207.9) in boreholes TLR1, TLR2, TLR3 advanced in the north abutment/pier area and at 0.6 m depth (elevation 209.8) in borehole TLR5 put down in the vicinity of the south pier. No water was observed in the remaining boreholes. The groundwater is likely perched above the bedrock underlying the structure site. It is noted that the groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.

5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. M. Rapsey, Senior Technician, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer. Walker Drilling Co. Ltd. supplied the drilling equipment. The laboratory work was carried out in the PML laboratory in Toronto.

This Preliminary Foundation Investigation Report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M. P. Nascimento, P.Eng. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

**PART B
PRELIMINARY FOUNDATION DESIGN REPORT**

for
Trout Lake Road Overpass
Future Highway 11/17
City of North Bay
GWP 5748-04-00
District 54, Sudbury, Ontario

6. ENGINEERING RECOMMENDATIONS

6.1 General

This report provides preliminary foundation engineering comments and recommendations regarding design and construction of foundations, abutments and approach fill embankments for the proposed overpass to be located on the alignment of Trout Lake Road at the Future Highway 11/17 in North Bay, Ontario.

The overpass is planned with a centre span of 66.0 m in length and two end spans of 44.0 m each for a total length of 154.0 m between abutments, and with a width of about 29.5 m (ref. Preliminary General Arrangement (GA) drawing prepared by Stantec in July 2007).

The road grade on the Future Highway 11/17 at the overpass location will be near elevation 218.0 at both abutments. The approach embankments to the structure are envisaged to be about 8 m high (interpreted from the road grade and ground surface elevations shown on the preliminary GA drawing). The grade on Trout Lake Road will be slightly above elevation 210.0.

In summary, the subsurface stratigraphy revealed in the boreholes drilled at the site comprised a localised fill or surficial topsoil underlain by typically loose to compact sand with variable silt and gravel content. Cobbles and boulders were encountered in four boreholes. Bedrock was inferred to increase in depth from 0.6 to 6.1 m in the direction of the north abutment. Groundwater was at elevation 206.9 to 207.9 to the north of Trout Lake Road and elevation 209.8 to the south.

Based on the preliminary data and subject to results of a detailed field investigation, design and construction of foundations to support the proposed overpass is considered feasible at this site.



The sand stratum present at the site is variable in relative density, with loose to very loose zones. Consequently, use of spread footings placed in the sand to support the foundation loads is not suitable for this structure. From a foundation engineering perspective, end-bearing piles driven to bedrock are considered to be the preferred foundation system for the north abutment and north pier of the overpass and spread footings founded on bedrock for the south abutment and south pier.

The foundation frost penetration depth at this site is 2.0 m according to OPSD-3090.100. The seismic site coefficient for the conditions at the site is 1.0 – Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) 2006 Edition – for the anticipated foundation conditions.

A list of the standard specifications referenced in this report is enclosed in Table 1.

6.2 Foundations

6.2.1 Piles

Use of piles bearing on bedrock is considered feasible for the north abutment and north pier of the overpass. Construction of integral abutments supported on H-piles driven to bedrock is also feasible at this site provided that a trench is excavated in the bedrock at the south abutment to accommodate the minimum free pile length of 5 m. The H-piles driven to refusal on bedrock should be designed using the following axial resistance at ultimate limit states (ULS) for three pile sections:

<u>Factored Axial Resistance at ULS, kN</u>	
HP 310 x 110	2000
HP 310 x 132	2400
HP 310 x 174	3200



Cobbles and boulders were identified in the sand stratum in four boreholes. Taking into account that the sand is typically loose to compact, it is considered that damage to the piles when driving through cobbles/boulders is unlikely and, as a consequence, application of a reduction factor to account for potential damage during driving is not considered necessary.

The resistance at serviceability limit states (SLS) normally allows for 25 mm compression of the pile and founding medium. Considering the bedrock to be a non-yielding material, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm axial deformation of the pile is larger than the factored resistance at ULS.

The approach fill embankment as well as any fill placed below grade to deal with unsuitable/compressible soils within the limits of the pile foundation should comprise Granular A or Granular B Type II or Type III to enable driving of the piles and minimize the potential for damage during pile installation.

The piles should be provided with rock points (OPSD-3000.201 and SP 206S03) to minimize the potential for damage when setting on bedrock.

Pile caps should be provided with at least 2.0 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

6.2.2 Footings on Bedrock

At the south abutment and south pier, conventional spread footings founded on bedrock inferred at depths of 0.6 to 2.6 m (elevation 207.8 to 210.3) is the preferred means of supporting the foundation loads.



For preliminary design purposes, the bedrock is assumed to be medium to high strength and recommended preliminary geotechnical resistances for footings bearing on bedrock are as follows:

Factored Geotechnical Resistance at ULS, kPa	8,000
Geotechnical Resistance at SLS, kPa	N/A

The geotechnical resistance at SLS normally allows for 25 mm of total compression of the founding medium. Considering the bedrock to be unyielding, the design is not governed by settlement criteria since the loading required to produce the above deformation of the bedrock would be larger than the factored geotechnical resistance at ULS.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

6.2.3 Caissons

Installation of caissons founded on or socketed into bedrock would be impractical due to the high groundwater level, the presence of pervious sand as well as cobbles and boulders at the site. Consequently, it is considered that use of caissons to support the foundation loads is not suitable for this structure.

6.3 Approach Embankments

It is anticipated that the approach embankments will be constructed with earth borrow, granular material and/or rock fill. The design calls for the embankment to be about 8 m high at both abutments. The subgrade revealed in the boreholes drilled at the site consists of typically loose to compact sand. It is noted that part of the alignment may be currently occupied by residences and municipal roadway sections which will be removed.



Any topsoil or future rubble at the abutment locations and along the alignment of the approach fill within 20 m of the abutments should be stripped prior to placement of the embankment fill and the loose sand subgrade proof rolled.

In general, the embankments should be constructed following conventional MTO procedures (OPSD-200.010, 201.010, 202.010 and SP 206S03) and have side slopes inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rock fill. The settlement of the approach fill and native soils induced by the embankment loads is anticipated to be in the order of 60 mm.

The backfill to the abutments should comprise granular material placed in accordance with the requirements of SP 206S03 and OPSS 501 to minimize post-construction settlements.

6.4 Excavation Considerations

6.4.1 Excavation

Excavation for construction of the spread footings or pile caps is expected to extend through the fill and very loose to compact sand to a depth of up to 2.5 m. Excavation of these soils should be relatively straightforward.

The fill and loose to compact sand are classified as Type 3 material according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Very loose sand and native soils below the groundwater levels are classified as Type 4 material. Therefore, temporary cut slopes over the full depth of excavation should be cut at an inclination of 3H:1V.

Excavation of bedrock if required will be more difficult and necessitate conventional rock excavation techniques such as blasting (OPSS 120) and jack-hammering. The actual equipment required and method of excavation within the bedrock will be dependent upon the geometry of cut and relative depth of excavation into the bedrock. Blasting activities within the city limits may be restricted.



Mechanical means such as a large excavator equipped with a tiger-toothed bucket in conjunction with a jack-hammer or hoe ram is the preferred method of excavation to shallow depths in rock at foundation locations. Mass concrete could be employed to level minor variations in the bedrock surface.

Near vertical sidewalls may be utilised for excavations in bedrock. Examination of the sidewalls and removal of any loosened rock fragments should be carried out continually for the safety of workers.

6.4.2 Groundwater Control

Groundwater was below the anticipated base of excavations to the north of Trout Lake Road and some 2 m above the founding level in one of the boreholes advanced to the south. It is noteworthy that groundwater levels are subject to seasonal fluctuations and precipitation patterns.

It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations for construction of the north abutment and north pier. More positive groundwater control measures using a combination of cut-off trenches and sumps may be warranted south of Trout Lake Road.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

6.5 Lateral Earth Pressures

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p (kPa) may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation.

$$p = K(\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)
 γ = unit weight of free-draining granular material, kN/m^3



- h = depth below final grade, m
 q = surcharge load, kPa, if present
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
 where \emptyset = angle of internal friction of retained soil (35° for Granular A or Granular B Type II or Type III)
 δ = angle of friction between the soil and wall (23.5° for Granular A or Granular B Type II or Type III)

Free-draining granular material should be used as backfill behind the wall. The following parameters are recommended for design:

PARAMETERS	GRANULAR A or GRANULAR B TYPE II or TYPE III
Internal Friction Angle, \emptyset (degrees)	35
Unit weight, γ (kN/m ³)	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient of Passive Earth Pressure, K_p	3.69

The assigned geotechnical parameter values are the same for all granular materials in view of their similar physical characteristics.

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures.

The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. We refer to Figure C6.16 of the CHBDC for this computation. The subsoil/backfill should be considered as medium dense sand for the project.



A subdrain system (SP 405F03) and/or weep holes (OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. The subdrains tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.

7. ADDITIONAL STUDIES

The recommendations in this report are preliminary and based on PML's interpretation of the factual information obtained from a limited number of boreholes and a visual site assessment. Detailed foundation investigation will be required at the structure location during the Detail Design phase of the project. The interpretation and recommendations are only provided for planning purposes and feasibility studies.

Based on the 29.5 m wide structure, the limited number of borehole data and a visual site assessment for the overpass structure, the recommended additional scope of the foundation investigation is as follows:

- Boreholes should be carried out at the centre (1), 2.5 m in front (2) and 2.5 m behind (2) the north and south abutments centreline and four additional boreholes should be allowed at strategic locations within each foundation footprint to define the surface of the bedrock. At least three of the boreholes at the corners and at the centre of each foundation should be cored 3.0 m into the bedrock.
- Two boreholes should be carried out for each approach embankment 20 m from the abutments.



8. DISCUSSION OF FOUNDATION ALTERNATIVES

8.1 Advantages and Disadvantages of Foundation Alternatives

The following table summarizes the advantages and disadvantages and inferred risks/consequences of each of the foundation alternatives for the proposed Trout Lake Road overpass at Highway 11/17:

SPREAD FOOTINGS ON BEDROCK		DRIVEN PILES	
ADVANTAGES	DISADVANTAGES	ADVANTAGES	DISADVANTAGES
<ul style="list-style-type: none"> • Less costly than deep foundation alternative • Conventional design and construction of foundations • Allows semi-integral abutment design 	<ul style="list-style-type: none"> • Long-term maintenance costs of expansion joints for conventional abutment and deck design • May require mass concrete to achieve founding level subgrade on bedrock 	<ul style="list-style-type: none"> • Allows integral abutment design and construction • Lower long-term maintenance cost expansion joints with integral abutment design 	<ul style="list-style-type: none"> • More costly than shallow foundation alternative • Heavy equipment for pile driving is required • Potential sloping bedrock may cause pile installation difficulties • Requires bedrock excavation (south abutment) to achieve minimum required pile length

8.2 Preferred Foundation Option Considerations

From the foundation perspective, spread footings founded on bedrock and driven pile foundations are considered feasible. Although the spread footing foundations for conventional or semi-integral abutments are considered to be the least costly alternative for construction, the semi-integral or integral abutments will have lower long-term maintenance and user costs.

Consequently, the most economical alternative in the long term is the semi-integral abutments on spread footings. This foundation scheme is considered to be the preferred foundation system from the geotechnical standpoint, subject to the results of the foundation investigation conducted during the detailed design.



It is noted that the selection of a foundation scheme also depends on other considerations, such as structural design and road grades, which are being evaluated separately by Stantec.

9. CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

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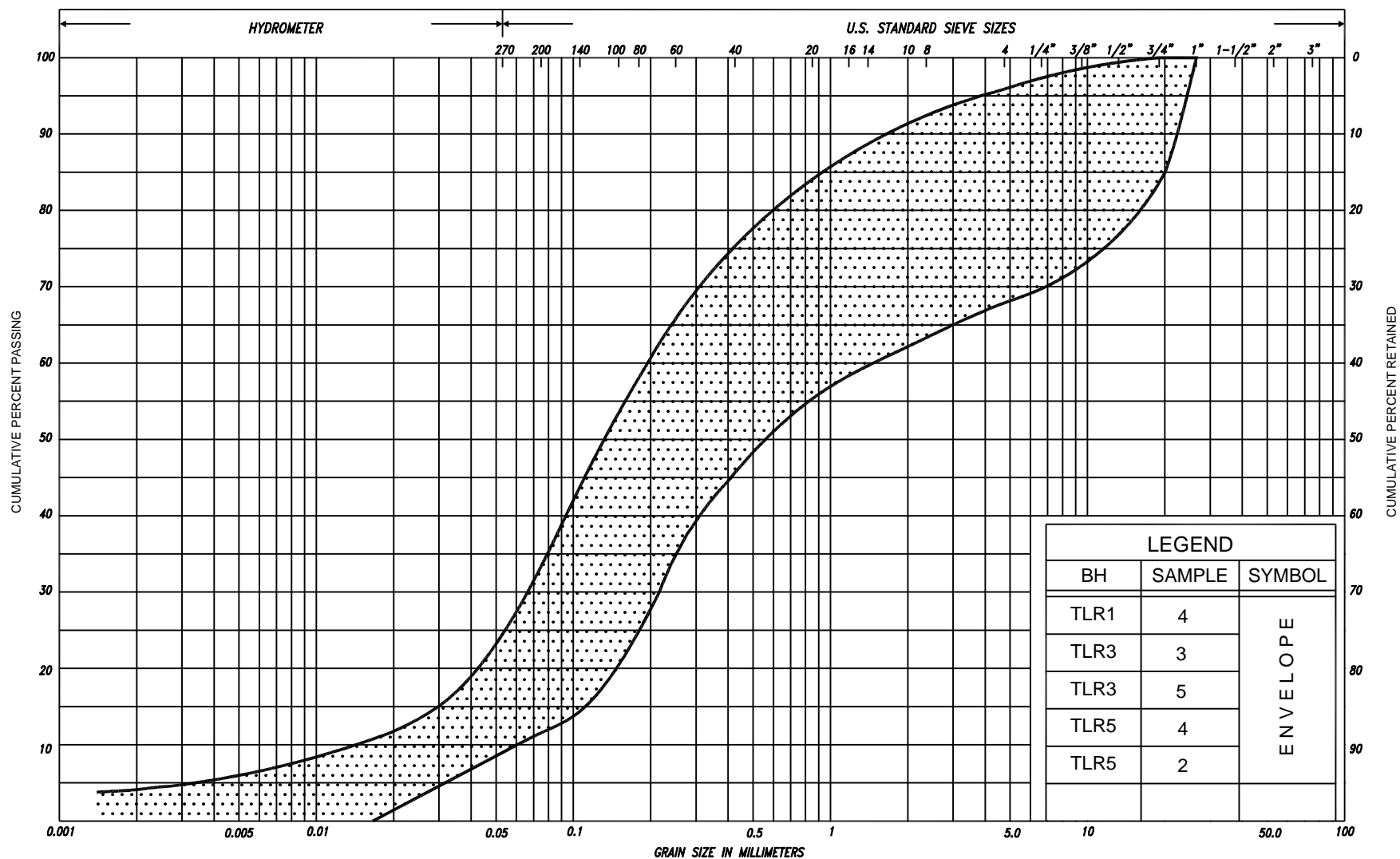
GD/CN/BRG:gd-lnr





TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
OPSS 120	General Specification for the Use of Explosives	November 2003
OPSS 501	Construction Specification for Compacting	November 2005
SP 206S03	Construction Specification for Grading	November 2006
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
OPSD-200.010	Earth/Shale Grading – Undivided Rural	November 2005
OPSD-201.010	Rock Grading-Undivided Rural	November 2005
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment	November 2005
OPSD-3000.201	Oslo Points for Foundation, Piles, Steel HP310	November 2005
OPSD-3090.100	Foundation Frost Depth for Northern Ontario	November 2005
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail	November 2005



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT							SAND									
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU	
					SAND												

GRAIN SIZE DISTRIBUTION

SAND, some to with silt, trace to with gravel, trace clay
to
GRAVELLY SAND, some silt

FIG No. GS-TLR-2

HWY: 11/17

G.W.P. No. 5748-04-00

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 475 J 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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING 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

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
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'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m^3	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No TLR1

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 177 N; 309 377 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
DATUM Geodetic DATE April 18, 2007 CHECKED BY C.N.

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)								
						○ UNCONFINED			+ FIELD VANE			● QUICK TRIAXIAL			× LAB VANE				
210.2	Ground Surface					20	40	60	80	100	20	40	60			GR	SA	SI	CL
0.0	Silty sand, cobbles	⊗																	
	Very loose Dark brown Moist (FILL)	⊗	1	SS	1														
208.8		⊗																	
1.4	Sand, trace silt	•																	
	Compact Brown Wet	•	2	SS	9														
	cobbles	•																	
		•	3	SS	17														
	with silt, trace gravel	•																	
	Grey	•	4	SS	9														
	with gravel	•																	
		•	5	SS	33														
		•																	
		•	6	SS	12														
204.9	End of borehole	•																	
5.3	Refusal on probable bedrock																		

RECORD OF BOREHOLE No TLR2

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 176 N; 309 364 E. ORIGINATED BY M.R.
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
 DATUM Geodetic DATE April 18, 2007 CHECKED BY C.N.

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			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
								○ UNCONFINED		+ FIELD VANE			w _p w w _L							
209.9	Ground Surface																			
0.0	Topsoil																			
0.2	Sand trace to some silt		1	SS	7															
	Loose Rusty Dry brown		2	SS	6															
	Brown Dry		3	SS	3															
	with silt																			
	Grey Saturated		4	SS	10															
			5	SS	18															
	some silt, some gravel some cobbles																			
			6	SS	40/8cm															
203.8			7	SS	50/15cm															
6.1	End of borehole																			
	Refusal on probable bedrock																			
</																				

METRIC

20
15 — 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No TLR4

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 145 N; 309 376 E. ORIGINATED BY M.R.
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
 DATUM Geodetic DATE April 18, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
210.5	Ground Surface							20	40	60	80	100					
0.0	Topsoil		1	SS	6		210							○			
0.1	Sand, with gravel cobbles																
	Compact Brown Moist		2	SS	23									○			
209.0							209										
1.5	End of borehole Refusal on probable bedrock																
	* Borehole dry																

METRIC

G.W.P.	5748-04-00	LOCATION	Co-ords. 5 131 101 N; 309 410 E.	ORIGINATED BY	M.R.
DIST	Sudbury	HWY	11/17	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
DATUM	Geodetic	DATE	April 19, 2007	CHECKED BY	C.N.


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


RECORD OF BOREHOLE No TLR6

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 096 N; 309 398 E. ORIGINATED BY M.R.
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
 DATUM Geodetic DATE April 19, 2007 CHECKED BY C.N.

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
210.9 0.0	Ground Surface					*			20	40	60	80	100					GR SA SI CL
	Topsoil																	
210.3 0.6	End of borehole Refusal on probable bedrock																	
	* Borehole dry																	

RECORD OF BOREHOLE No TLR8										1 of 1		METRIC	
G.W.P. 5748-04-00		LOCATION Co-ords. 5 131 196 N; 309 350 E.				ORIGINATED BY F.P.							
DIST Sudbury		HWY 11/17		BOREHOLE TYPE Shovel Probe Hand Sampling				COMPILED BY F.P.					
DATUM Geodetic		DATE June 22, 2007				CHECKED BY C.N.							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W _p W W _L				
209.3	Ground Surface												
0.0	Topsoil mixed with gravel, wood						209						
208.5													
0.8	Sand, trace silt												
208.2	trace gravel, trace rootlets												
1.1	occ. cobbles and boulders												
	End of borehole												
NOTE: Rubble noted at ground surface * Borehole dry													

METRIC

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

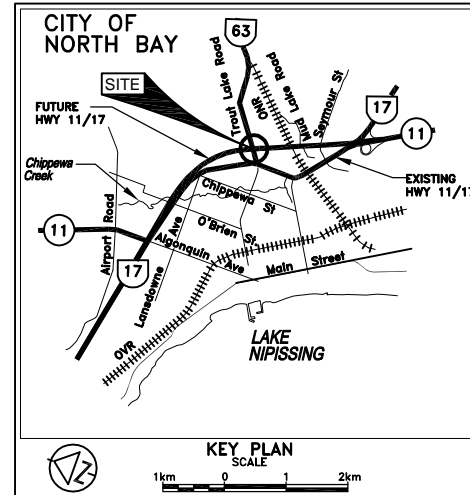
CONT No
GWP No 5748-04-00

TROUT LAKE ROAD OVERPASS
FUTURE HIGHWAY 11/17
BOREHOLE LOCATIONS AND SOIL STRATA

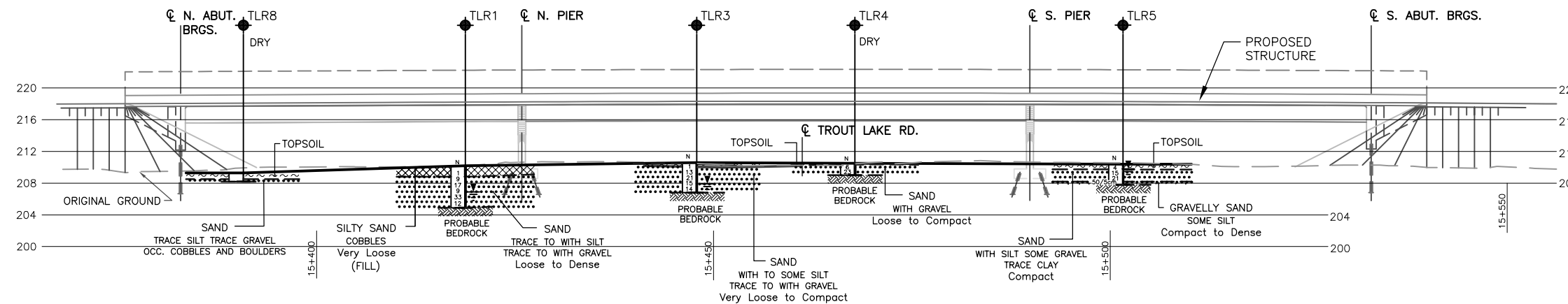
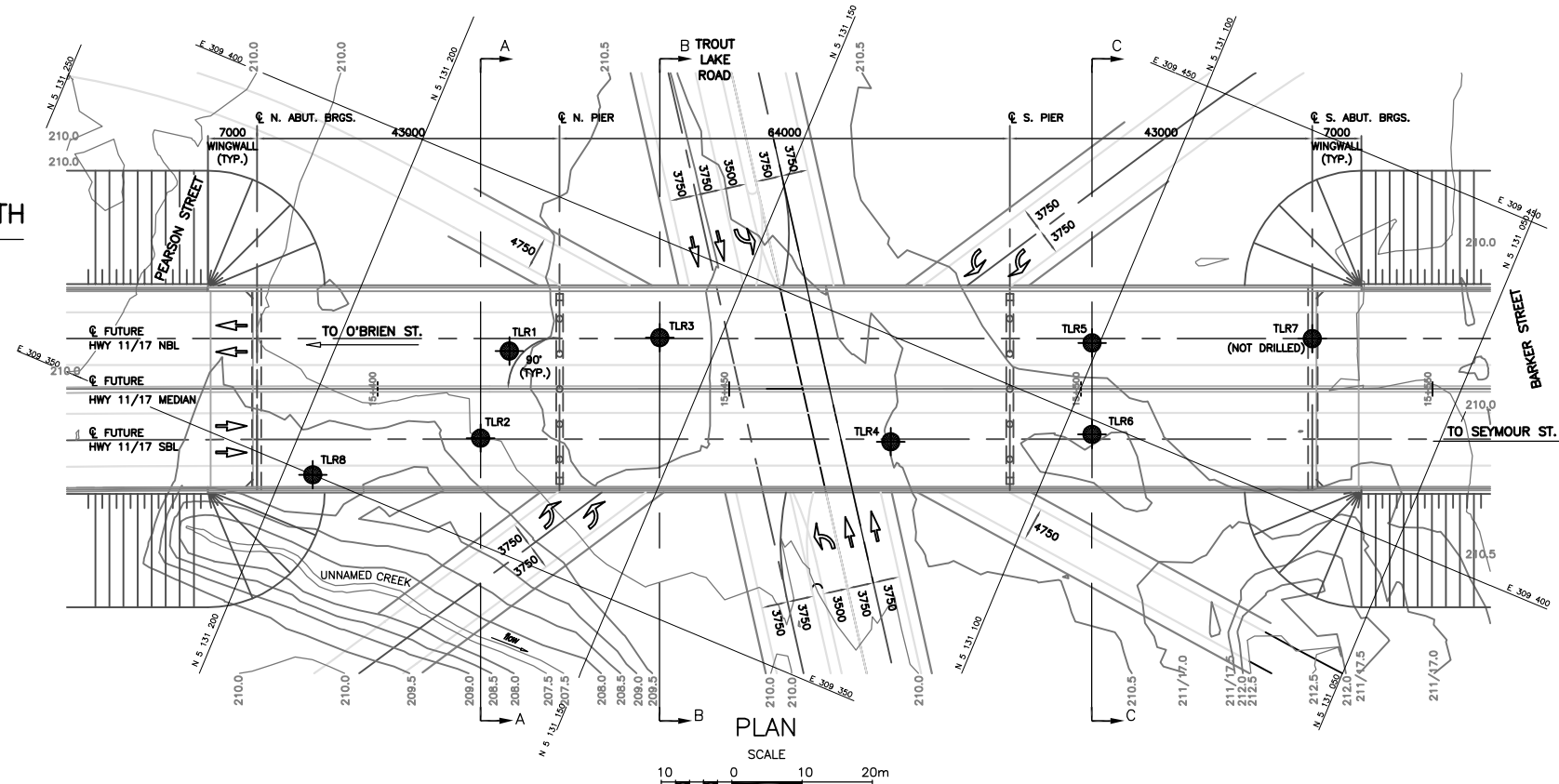


SHEET

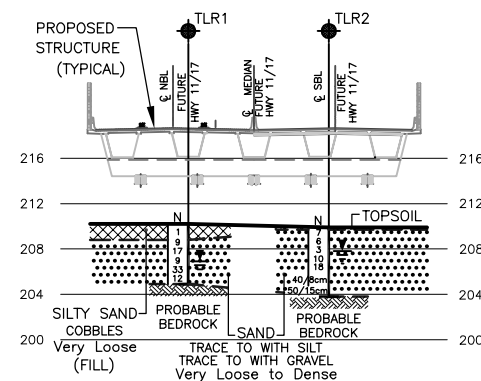
PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



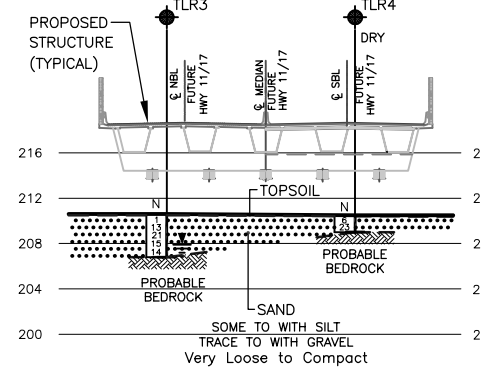
PROJECT NORTH



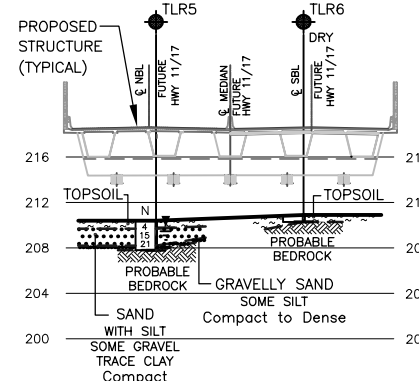
PROFILE FUTURE HWY 11/17



SECTION A-A



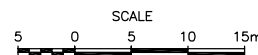
SECTION B-B



SECTION C-C

NOTES:

- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- COORDINATES AT BOREHOLE LOCATIONS WERE PROVIDED BY DEL BOSCO SURVEYING LTD.



LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60° Cone, 475 J / blow)		
	W L at time of investigation April 2007		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		

BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
TLR1	210.2	5 131 177	309 377
TLR2	209.9	5 131 176	309 364
TLR3	210.6	5 131 158	309 387
TLR4	210.5	5 131 122	309 386
TLR5	210.4	5 131 101	309 410
TLR6	210.9	5 131 096	309 398
TLR7		NOT DRILLED	
TLR8	209.3	5 131 196	309 350

NOTE

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-116

HWY No	11/17	DIST	SUDBURY
SUBM'D	FP	CHECKED	GD
DATE	NOV. 02, 2007	SITE	--
DRAWN	NA	CHECKED	CN
APPROVED	BRG	DWG	TLR-1

REF No.: STANTEC DRAWING: 65000580_SPUI_TROUT_LAKE_B5.dwg
dated JULY 2007



APPENDIX A

Site Photographs



Photograph 1: Looking south from Pearson Street. North abutment of structure is located at the rear of residence in photo. Note flat and densely treed terrain. (July 24, 2007)



Photograph 2: Looking north at south end of CSP culvert located at the northwest end of the proposed north abutment. (July 24, 2007)



Photograph 3: Looking northerly across Trout Lake Road from location near the west end of the proposed south pier. The north pier is proposed at the tree line in the middle of the photograph. (July 24, 2007)



Photograph 4: Looking east along Trout Lake Road from about centreline of proposed structure. Intersection of Laurentian Avenue is in the distance. (June 19, 2007)



Photograph 5: Looking west along south side of Trout Lake Road from about centreline of proposed structure. Intersection of existing Highway 11/17 in the distance. (June 19, 2007)



Photograph 6: Looking south from north side of Trout Lake Road. South pier and abutment are proposed within the densely treed area at centre of photograph. (July 24, 2007)