



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT  
for**

**DELANEY LAKE/MUD LAKE ROAD OVERPASS  
FUTURE HIGHWAY 11/17  
CITY OF NORTH BAY  
GWP 5748-04-00  
DISTRICT 54, SUDBURY, ONTARIO**

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PML Ref.: 05TF058C  
Index No.: 074FIDR  
Geocres No.: 31L-115  
November 2, 2007



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**PART A**  
**PRELIMINARY FOUNDATION INVESTIGATION REPORT**

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

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**1. INTRODUCTION**

This report summarizes the results of the preliminary foundation investigation carried out for the proposed construction of the Delaney Lake/Mud Lake Road Overpass at the Future Highway 11/17 in the City of North Bay, District of Sudbury, Ontario. Peto MacCallum Ltd. (PML) conducted the preliminary investigation for Stantec Consulting Limited (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed new overpass will carry the Future Highway 11/17 northbound and southbound lanes over the Delaney Lake and the Mud Lake Road. Stantec prepared the preliminary General Arrangement (GA) drawing for the structure that was dated July 2007.

According to the preliminary GA drawing, the Future Highway 11/17 will be a four-lane highway running north-south at the Delaney Lake/Mud Lake Road Overpass.

This report provides preliminary subsurface information pertaining to the proposed overpass foundations and approach embankments within about 20 m of the abutments.

**2. SITE DESCRIPTION AND GEOLOGY**

The contemplated Future Highway 11/17 structure is located about 80 m east of the intersection of Station Road and Mud Lake Road. The site is about 1,000 m north from the existing Highway 11 and Highway 17 interchange.



Land use in the vicinity of the site comprises industrial and municipal plants, isolated residences and the railway station on the Station Road.

Topographically, the structure site is located on undulating terrain marked by exposed rock outcrops. Mud Lake Road runs parallel to the western 15 to 25 m wide section of Delaney Lake (also designated Mud Lake). Southwest of the proposed structure site, Johnston Creek flows westerly from Delaney Lake. Swampy areas exist along this creek and to the west of the west end of Delaney Lake. The ground cover comprises grasses and typical swamp vegetation and bush.

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 soil/bedrock interface is at variable depths, generally shallow, and extensive bedrock exposures are visible at site.

### **3. INVESTIGATION PROCEDURES**

The subsurface investigation was carried out on April 19 and 30, 2007. In the view of obvious bedrock exposures, the investigation comprises a total of 10 test holes (8 boreholes and 2 test pits) which were identified with the prefix MLR. The test holes were advanced to depths of 0.0 to 2.3 m at the locations shown on Drawing MLR-1. Three soundings of the bottom of the Delaney Lake were taken at the pier location. The level of the bottom of the lake, estimated from the measurements are shown on the Foundation Drawing MLR-1. Photographs of the crossing are enclosed in Appendix A. The test pits were reported in MTO record of borehole forms with the borehole type indicated as excavated test pits.

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

Del Bosco Surveying Ltd. laid out and surveyed the test hole locations. PML cleared the locations of the test holes for the presence of underground services and utilities. All elevations in this report are expressed in metres.



The boreholes were advanced manually where bedrock exposures were found or by using continuous flight hollow stem augers powered by a track mounted CME-55 drill rig and the test pits were dug by a CAT 312 track mounted excavator. The equipment was supplied and operated by specialist contractors, working under the full-time supervision of a Field Supervisor from the PML engineering staff. All test holes were backfilled in accordance with the MTO guidelines and MOE Reg. 903 for borehole and test pit abandonment procedures.

Where soil cover was encountered in the boreholes, soil samples were recovered using the Standard Penetration Test method. In the test pits, chunk samples were obtained. The consistency and/or relative density of these soils were estimated from observation of the test pit walls and manual examination of the samples.

The groundwater conditions at the test hole locations were assessed by visual examination of the soil and where appropriate, by measurement of the water level in the open holes. The water level observations are noted on the attached record of boreholes.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, soil classification and laboratory testing. The laboratory test program comprised the following tests:

- Natural moisture content determinations (4)
- Grain Size analyses (2)
- Atterberg Limit (1)

The results of the laboratory natural moisture content determinations, grain size analyses and Atterberg limit results are shown on the Record of Borehole sheets. The grain size distribution charts are presented on Figures GS-MLR-1 and GS-MLR-2. The Plasticity Chart showing the Atterberg limits is presented on Figure PC-MLR-1.



#### **4. SUMMARIZED SUBSURFACE CONDITIONS**

##### **4.1 General**

Refer to the Record of Borehole sheets for the details of the subsurface conditions including soil classifications, inferred stratigraphy, soil and rock boundary levels and groundwater observations.

The test hole locations and the preliminary layout of the Mud Lake Road Overpass as well as the longitudinal soil profile and cross-sections are presented on the Foundation Drawing MLR-1.

The soil stratigraphy in the test holes comprises units of surficial topsoil, peat or fill underlain by cohesive clayey silt and/or cohesionless sand. The sand and fill mantle probable bedrock. Bedrock outcrops are present in six test holes.

##### **4.2 Fill**

Fill units 0.9 and 1.5 m thick are encountered in test holes MLR4 and MLR3, drilled in the Mud Lake Road. The units are composed of silty sand with cobbles in test hole MLR4 and asphalt surface treatment over gravelly sand some silt in test hole MLR3. The fill units exhibit loose and compact relative densities in the test holes MLR4 and MLR3, respectively. The fill extended to 0.9 and 1.5 m depths, elevations 203.8 and 204.2.

The grain size distribution chart of a representative fill sample (test hole MLR3) is shown on Figure GS-MLR-1. The water content determination on this fill sample was 16%.

##### **4.3 Topsoil/Peat**

A surficial deposit of topsoil 200 mm thick is present in test holes MLR1 and MLR2 extending to elevations 205.9 and 205.5, respectively.



A local 900 mm thick unit of peat is identified below the fill unit in test hole MLR4 drilled at the margin of Delaney Lake at a depth of 0.9 m (elevation 204.2) and extending to a depth of 1.8 m (elevation 203.3). The water content of this peat sample was 154%.

#### **4.4 Clayey Silt Till**

Below the topsoil, a cohesive glacial till comprised of clayey silt and sand trace gravel layer occurs in the test hole MLR1 extending to probable bedrock at 1.3 m depth (elevation 204.8). This unit has a firm to stiff consistency.

The grain size distribution chart of a representative sample of the clayey silt till is shown on Figure GS-MLR-2 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-MLR-1. The liquid limit of this sample was 18 and the plastic limit 10, giving the plasticity index of 8. The water content of this sample was 6%.

#### **4.5 Sand**

A 0.5 to 1.0 m thick stratum of sand with silt with gravel locally containing boulders is present below the topsoil and peat layers extending to the probable bedrock surface at depths of 1.2 and 2.3 m (elevations 204.5 and 202.8) in test holes MLR2 and MLR4, respectively.

The sand unit typically exhibits a loose to compact relative density. The water content determination value of one sand sample was 21%.

#### **4.6 Bedrock**

Bedrock is present at the surface of test holes MLR5, MLR6, MLR7, MLR8, MLR9 and MLR10, at elevations 206.3 to 208.3. The depth to the bedrock surface as inferred by refusal to further advance and varies from 1.2 to 2.3 m, elevations 202.8 to 204.8 in the test holes MLR1, MLR2, MLR3 and MLR4. The bedrock level in the Delaney Lake at the location of the pier is at or below the level of the bottom of the lake encountered at elevation 203.7 to 203.9 from three soundings.



It is inferred that the bedrock comprises metasedimentary gneiss which are typically encountered in the North Bay area. The bedrock level was not established at the sounding locations.

#### **4.7 Groundwater**

Groundwater was measured only in the test hole MLR4 at depth 0.9 m (elevation 204.2). No water was observed in the remaining test holes. It is inferred that the water level at the site is influenced by the water level in Delaney Lake, that was at about elevation 204.8 at the time of the investigation. The groundwater levels are subjected to fluctuations due to seasonal and rainfall patterns.

#### **5. MISCELLANEOUS**

The field work was carried out under the supervision of Mr. M. Rapsey, Senior Technician, and the direction of Mr. C.M.P. Nascimento, P.Eng., Senior Foundation Engineer. Abraflex (2004) Ltd. supplied the drilling equipment and Bruman Leasing Ltd. supplied the excavator. The laboratory work was carried out in the PML laboratory in Toronto.

This Preliminary Foundation Investigation Report was prepared by Mr. C.M.P. Nascimento, P.Eng., with the assistance of Ms. N.S. Balakumaran, BSc., and independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

**PART B**  
**PRELIMINARY FOUNDATION DESIGN REPORT**  
for  
Delaney Lake/Mud Lake Road Overpass  
Future Highway 11/17  
City of North Bay  
GWP 5748-04-00  
District 54, Sudbury, Ontario

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**6. ENGINEERING RECOMMENDATIONS**

**6.1 General**

Part B of this report provides the preliminary foundation engineering recommendations regarding design and comments for construction of the proposed Delaney Lake/Mud Lake Road Overpass for the Future Highway 11/17. The recommendations are preliminary and based on the results of the limited subsurface investigation, that was outlined in the Part A of this report.

Based on the preliminary GA drawing, the Delaney Lake/Mud Lake Road overpass will be a two span structure with a total length of 78 m between abutments. The proposed 34 m wide bridge deck will carry six lanes of traffic (four lanes and two ramps) for the Future Highway 11/17 and will be about 12 m high above Mud Lake Road at about elevation 219.0. The approach embankments will also be 12 m high at the abutments.

In summary, the soil stratigraphy revealed in the test holes generally indicates that north and south abutments are located on bedrock outcrops. The pier location is partially underlain by the existing Mud Lake Road embankment fill followed by peat and cohesionless sand over inferred bedrock. The pier extends partially into the Delaney Lake where the bedrock surface level was not established below the bottom (elevations 203.7 to 203.9). The inferred bedrock surface was found at relatively shallow depths of 1.5 and 2.3 m, elevations 203.8 and 202.8 in two boreholes drilled near the pier location. At the abutment locations, the bedrock was encountered at the ground surface or under sparse soil cover as inferred from visual observation.

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



## 6.2 Foundations

### 6.2.1 General

Based on the preliminary data, founding the proposed overpass structure on spread footings placed on bedrock (abutments and centre pier) or structural fill (abutments) is considered feasible. Footings may be used for conventional or semi-integral abutment design.

The alternative scheme with pile foundations at the north and south abutments for integral abutment design alternative is also possible. This alternative will require the placement of a fill pad within the approach embankment to drive the piles. The excavation of a trench in bedrock to accommodate the minimum free pile length of 5 m below the abutment stem may not be required at this site in view of the 12 m high fill at each abutment.

The seismic site coefficient for the stratigraphic conditions at this site is 1.0 [soil profile Type I, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6].

### 6.2.2 Spread Footings on Bedrock

For the preliminary design the following are the anticipated depths below existing ground surfaces and elevations of the bearing surfaces of the abutments and pier spread footings founded on bedrock:

FOUNDATION ELEMENT	PML BH NO.	FOUNDING DEPTH (m)	FOUNDING ELEV.
North Abutment	MLR7 (east)	0.0	208.3
	MLR9 (west)	0.0	208.0
Centre Pier(*)	MLR3 (east)	1.5	203.8
	MLR4 (west)	2.3	202.8
South Abutment	MLR8 (east)	0.0	206.6
	MLR10 (west)	0.0	207.5

(\*) Rock level in Delaney Lake was not established below bottom of lake (elevation 203.7 to 203.9).



For preliminary design purposes, medium to high strength bedrock is assumed. The recommended preliminary bearing resistances for footings bearing on this 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.

Variations in the level of the bedrock should be anticipated in particular at the pier location which extends into Delaney Lake. The differences in elevation should be made up with mass concrete.

The foundation frost depth for structure foundations at this site is 2.0 m, according to OPSP-3090.100. Frost protection is not required for spread footings placed directly on the bedrock.

The lateral loads imposed on the foundations will be partly resisted by the friction developed between the underside of the concrete footing and the native soils. These forces should be calculated in accordance with the CHBDC. A coefficient of friction equal to 0.7 may be assumed between concrete footings and the bedrock.

#### 6.2.3 Spread Footings on Structural Fill

Construction of the abutment footings on structural fill placed in the approach embankment could also be employed to support the foundation loads. The structural fill should comprise Ontario Provincial Standards Specifications (OPSS) Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

Footings should not be constructed on rockfill. However, rockfill may be placed adjacent to the Granular 'A' core.



The recommended bearing resistance for 2.5 m wide footings constructed on structural fill (bearing resistance independent of fill thickness at this location because the engineered fill should be placed directly on the bedrock) is as follows:

Factored Geotechnical Resistance at ULS, kPa	900
Geotechnical Resistance at SLS, kPa	350

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 2.0 m was assumed for computation of the ULS resistance.

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

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.70 is recommended for footings placed on granular fill.

#### 6.2.4 Piles

For the preliminary design of piles for integral abutments, steel H-piles driven to refusal on bedrock should be used. The anticipated depths and elevations of the bedrock surfaces are the same as those indicated for spread footings placed on the bedrock.

Based on the preliminary GA drawing and assuming a 4 m deep abutment stem, the pile tips will be established at the top of the bedrock found at about elevation 208.0 for the north abutment and 207.0 for the south abutment, subject to structural detail design.



Based on bedrock medium to high strength assumed at the pile tips, the preliminary factored axial resistance at ultimate limit states (ULS) for the three pile sections noted below is considered to be appropriate:

<b>Pile Section</b>	<b>Factored Axial Resistance at ULS (kN)</b>
HP 310 x 79	1,450
HP 310 x 110	2,000
HP 310 x 132	2,400

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be a non-yielding material and the short pile length required, the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile are much larger than the ULS factored capacity.

Two concentric CSPs that extend at least 3 m below the abutments should be placed around the pile to create an annular space for the foundation scheme of integral abutments. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Refer to MTO Report SO-96-01 for further details.

The need to provide the piles with driving shoes or rock points (OPSD-3000.100, 3000.201 and SP 903S01) to minimize the potential for damage when setting on the bedrock or driving through boulders should be assessed at the Detail Design stage.

A minimum of 2.0 m of soil cover or the equivalent thermal equivalent insulation should be provided for frost protection to the pile caps.

The lateral loading could be resisted fully or partially by battered piles. For vertical piles such as those used for integral abutments, the resistance to lateral loading will be derived from the soils in front of the piles.



### **6.3 Approach Embankments**

Boreholes were not carried out for the approach embankments to the Delaney Lake/Mud Lake Road Overpass. We anticipate, however that the approach embankments are likely to be founded on bedrock, in view of the bedrock outcrop and shallow bedrock conditions encountered at the north and south abutments. Further subsurface investigations should be carried out at these locations for detail design.

The approach embankments should be designed and constructed in accordance with OPSD-200.010, 201.010, 202.010 and SP 206S03. The side slopes of the approach embankments will be stable where they are inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rockfill.

It is noted that where the embankment fill height exceeds 8 or 10 m for earth and rockfill, respectively a 2 m wide mid-height berm will be required. The earth fill slopes, if employed, should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation.

The backfill to the abutments will be about 12 m high. The backfill should be constructed with granular materials adjacent to the abutments to minimize the post-construction settlement of the road surface due to "consolidation" of the backfill. The magnitude of the "consolidation" of these fills placed in accordance with the requirements of SP 206S03 and OPSS 501, should be in the order of 50 mm and be essentially complete within 3 to 6 months after placement of the fill.

### **6.4 Construction Considerations**

#### **6.4.1 Excavation**

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations. For this purpose, the topsoil, cohesionless sandy gravel or sandy soils encountered in the boreholes are considered Type 3 soils. The rock is considered Type 1 soil according to OHSA.

The rock excavation, if required, should be carried out in a manner that minimizes disturbance to the founding level bedrock.



#### 6.4.2 Groundwater Control

According to the preliminary GA drawing, the foundation of the pier is at least partly located in the Delaney Lake. It is anticipated that a special underwater construction protection scheme such as cofferdams and unwatering will be required during excavation and construction to establish the footing founding level.

Several cofferdam alternatives such as braced sheet piling, earth-type timber crib or double-wall sheet piles may be considered depending on the conditions in the lake determined at the detail design stage. The contractor is responsible for the selection, preparation and performance of a detailed design for the construction scheme.

No water was observed during the course of the field work at the abutments locations. It is considered that seepage from soil and rock fissures or surface water run-off that enters the excavation should be readily handled by conventional sump pumping techniques.

Groundwater conditions should be further assessed during detail design by drilling boreholes to the full depth contemplated for the proposed foundation construction.

#### 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)  
 $\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$   
 $h$  = 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  $\phi$  = angle of internal friction of retained soil ( $35^\circ$  for Granular A or Granular B Type II or Type III)  
 $\delta$  = angle of friction between the soil and wall ( $23.5^\circ$  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, $\phi$ (degrees)	35
Unit weight, $\gamma$ (kN/m <sup>3</sup> )	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.

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 pipes should be installed on a positive grade and lead to frost-free outlets.



## **7. SCOPE OF ADDITIONAL FOUNDATION INVESTIGATION**

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 foregoing interpretation and recommendations are only provided for planning purposes and feasibility studies.

Based on the proposed 34 m wide structure, the limited number of borehole data and a visual assessment of the overpass structure site, 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 pier, 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 elements should be cored 3.0 m into the bedrock.
- Two boreholes should be carried out for each of the approach embankments.

## **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 Delaney Lake / Mud Lake Road Overpass at Highway 11/17.



### **Spread Footings Founded on Bedrock**

#### **Advantages**

- Less costly than deep foundation alternative
- Conventional design and construction of foundations
- Allows semi-integral abutment design

#### **Disadvantages**

- Long-term maintenance costs of expansion joints for conventional abutment and deck design.
- May require mass concrete to achieve level founding subgrade on bedrock

### **Footings on Structural Fill**

#### **Advantages**

- Reduced height of abutment stem
- Allows semi-integral abutment design

#### **Disadvantages**

- More costly than spread footing foundation alternative on bedrock
- Requires structural fill construction
- Construction of structural fill pad requires wider area than footings on bedrock

### **Driven Piles**

#### **Advantages**

- Allows integral abutment design and construction
- Lower long-term maintenance costs of deck expansion joints with integral abutment design
- Negligible settlements of foundations

#### **Disadvantages**

- More costly than shallow foundation alternatives
- Heavy equipment for pile driving is required.

## **8.2 Preferred Foundation Option Considerations**

From the foundation perspective, spread footings founded on bedrock or structural fill and driven pile foundations are considered feasible. The spread footing foundations founded on bedrock 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 costs.



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

It is noted that the selected foundation alternative 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. C.M.P. Nascimento, P.Eng., with the assistance of Ms. N.S. Balakumaran, BSc, and independently reviewed by Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.

A handwritten signature in blue ink, appearing to read "C. M. P. Nascimento", is written over the circular professional engineer stamp.

C. M. P. Nascimento, P.Eng.  
Senior Foundation Engineer



A handwritten signature in blue ink, appearing to read "Brian R. Gray", is written over the circular professional engineer stamp.

Brian R. Gray, MEng, P.Eng.  
MTO Designated Principal Contact

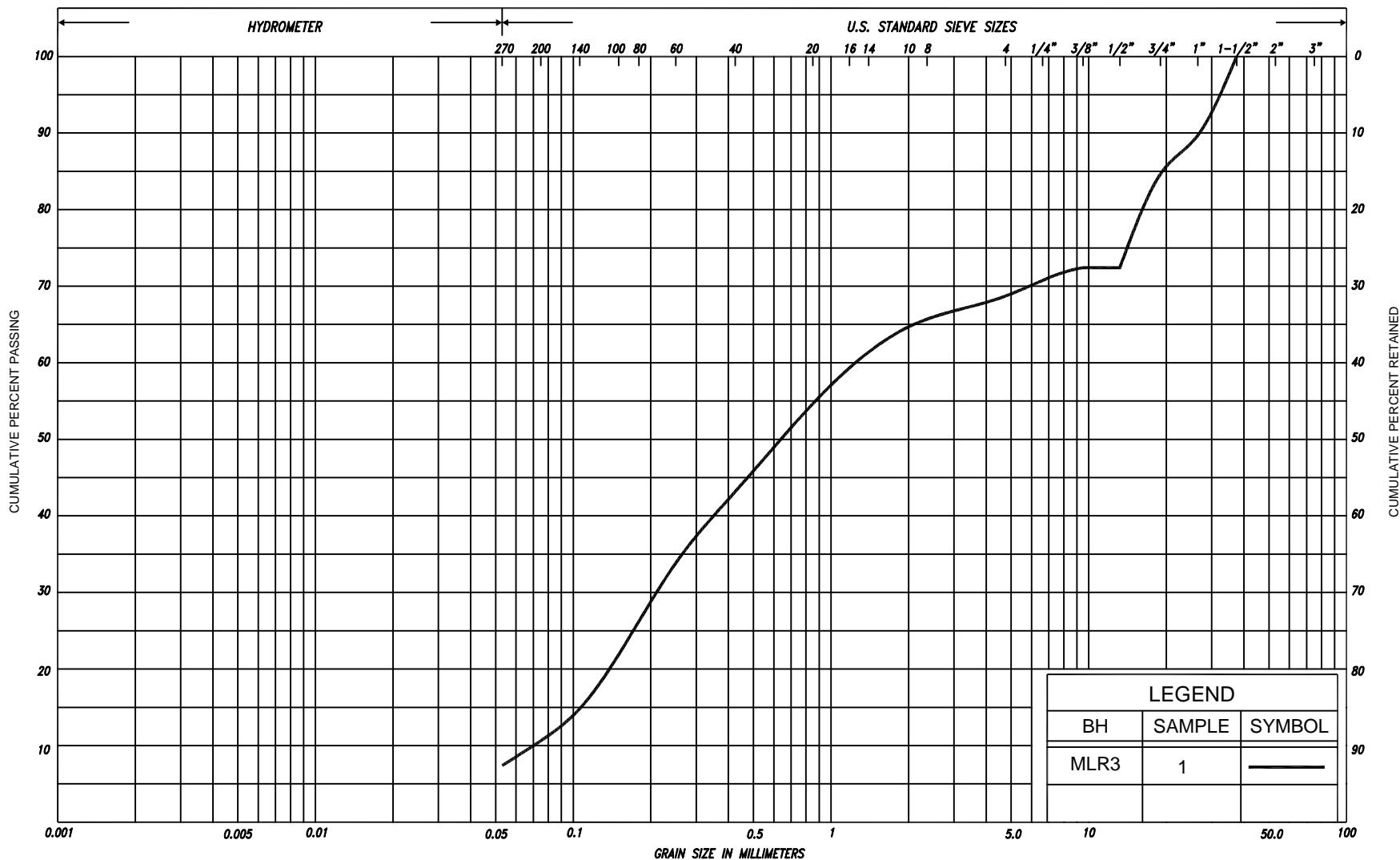


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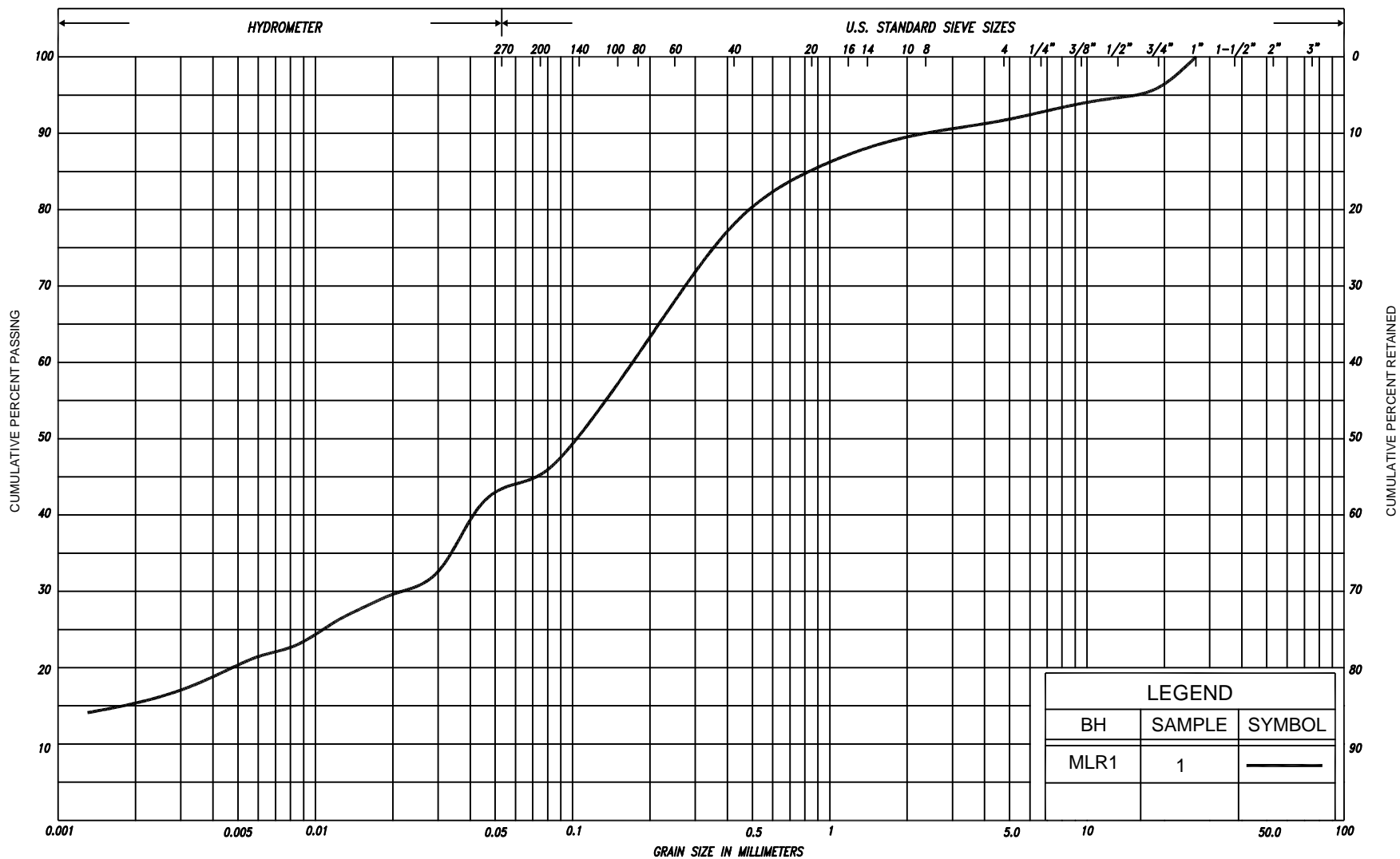


**TABLE 1**  
**LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT**

<b>DOCUMENT</b>	<b>TITLE</b>	<b>DATE</b>
OPSS 501	Construction Specification for Compacting	November 2005
OPSS 571	Construction Specification for Sodding	November 2001
SP 206S03	Construction Specification for Grading	November 2006
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
SP 903S01	Construction Specification for Piling	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-3000.100	Foundation Piles Steel H-Pile Driving Shoe	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



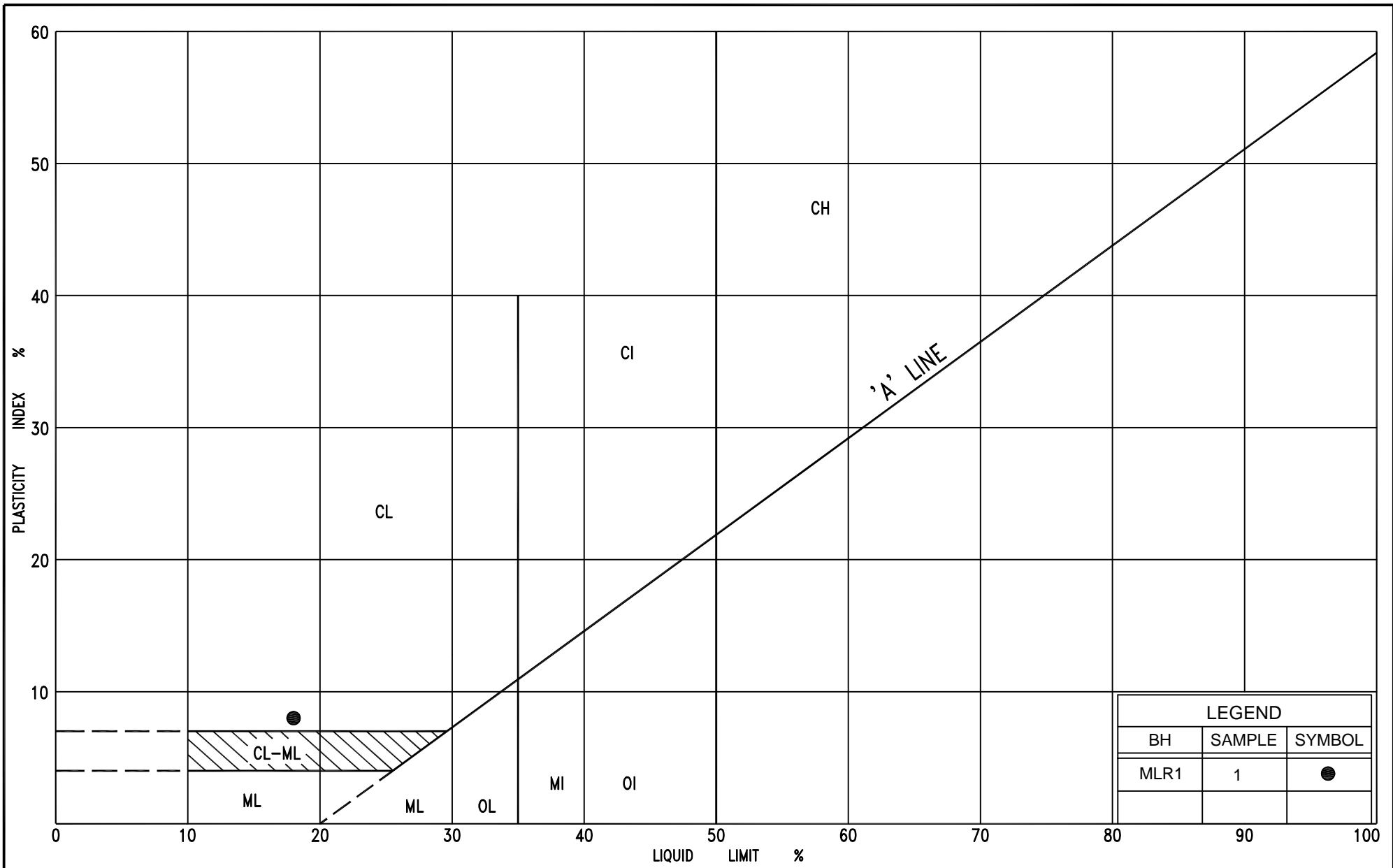
SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COB BLES	UNIFIED		
CLAY	FINE		MEDIUM		COARSE		SAND										
							FINE		MEDIUM		COARSE		GRAVEL			COBBLES	
CLAY			SILT			V. FINE		FINE		MED.		COARSE		GRAVEL			U.S. BUREAU



LEGEND		
BH	SAMPLE	SYMBOL
MLR1	1	—

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

# GRAIN SIZE DISTRIBUTION CLAYEY SILT and SAND, trace gravel (TILL)



## 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 (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	$e_{\max}$	1, %	VOID RATIO IN LOOSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	$e_{\min}$	1, %	VOID RATIO IN DENSEST STATE
$\rho_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	$S_r$	%	DEGREE OF SATURATION	$I_D$	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$w_L$	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
$\rho$	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_p$	%	PLASTIC LIMIT	$D_n$	mm	n PERCENT - DIAMETER
$\gamma$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_s$	%	SHRINKAGE LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\rho_{\text{sat}}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
$\gamma_{\text{sat}}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
$\rho'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	$\text{kN}/\text{m}^2$	SEEPAGE FORCE
e	1, %	VOID RATIO						



RECORD OF BOREHOLE No MLR1										1 of 1		METRIC					
G.W.P. 5748-04-00			LOCATION Co-ords. 5 130 420 N; 309 695 E.					ORIGINATED BY M.R.									
DIST Sudbury HWY 11/17			BOREHOLE TYPE Excavator					COMPILED BY T.X.									
DATUM Geodetic			DATE April 30, 2007					CHECKED BY C.N.									
SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
206.1	Ground Surface						206										
0.0	Topsoil																
0.2	Clayey silt and sand trace gravel																
204.8	Firm to Brown Moist stiff (TILL)		1	CS	-		205								0.1		8 47 30 15
1.3	End of borehole Refusal on probable bedrock																
	* Borehole dry																


# RECORD OF BOREHOLE No MLR2

1 of 1

**METRIC**

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 414 N; 309 682 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Excavator COMPILED BY T.X.  
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
205.7	Ground Surface						205											
0.0	Topsoil							20 40 60 80 100										
0.2	Sand, with silt with gravel, boulders							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
204.5	Loose to Brown Moist compact																	
1.2	End of borehole Refusal on probable bedrock																	
	* Borehole dry																	

RECORD OF BOREHOLE No MLR3										1 of 1		METRIC	
G.W.P. 5748-04-00		LOCATION Co-ords. 5 130 402 N; 309 703 E.				ORIGINATED BY M.R.							
DIST Sudbury HWY 11/17		BOREHOLE TYPE Continuous Flight Solid Stem Augers				COMPILED BY T.X.							
DATUM Geodetic		DATE April 19, 2007				CHECKED BY C.N.							
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>				
205.3	Ground Surface												
0.0	Asphalt surface treatment over gravelly sand some silt						205						
	Compact Brown Wet (FILL)		1	SS	15		204						
203.8	End of borehole Refusal on probable bedrock												
1.5													
	* Borehole dry												

RECORD OF BOREHOLE No MLR4										1 of 1		METRIC				
G.W.P. 5748-04-00		LOCATION Co-ords. 5 130 392 N; 309 691 E.				ORIGINATED BY M.R.										
DIST Sudbury HWY 11/17		BOREHOLE TYPE Continuous Flight Solid Stem Augers				COMPILED BY T.X.										
DATUM Geodetic		DATE April 19, 2007				CHECKED BY C.N.										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		ELEVATION SCALE	SHEAR STRENGTH kPa								
205.1	Ground Surface						20	40	60	80	100					
0.0	Silty sand, cobbles					205										
	Loose Dark brown Moist															
204.2	(FILL)					204										
0.9	Peat, fine fibrous															
	Dark brown Wet															
203.3	Sand, with silt		1	SS	15	203										
1.8	Compact Grey Wet															
202.8	End of borehole															
2.3	Refusal on probable bedrock															
<div style="display: flex; justify-content: space-between; margin-bottom: 10px;"> <span>* 2007 04 19</span> </div> <div style="display: flex; justify-content: space-between;"> <div style="width: 45%;">  Water level observed during drilling   Water level measured after drilling </div> <div style="width: 50%; text-align: center;"> <p>20</p> <p>15 — 5</p> <p>10</p> </div> </div>																

# RECORD OF BOREHOLE No MLR5

1 of 1

**METRIC**

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 346 N; 309 726 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Manual Sampling COMPILED BY T.X.  
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER * CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					W <sub>p</sub> W W <sub>L</sub>		
206.3	Ground Surface							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	kN/m <sup>3</sup>	GR SA SI CL
0.0	Bedrock at surface														
	* Borehole dry														

# RECORD OF BOREHOLE No MLR6

1 of 1

METRIC

G.W.P.	5748-04-00	LOCATION	Co-ords. 5 130 341 N; 309 723 E.	ORIGINATED BY	M.R.
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DIST Sudbury HWY 11/17 BOREHOLE TYPE Manual Sampling COMPILED BY T.X.

DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

[illegible]

# RECORD OF BOREHOLE No MLR7

1 of 1

**METRIC**

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 433 N; 309 690 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Manual Sampling COMPILED BY T.X.  
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER * CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					W <sub>p</sub> W W <sub>L</sub>		
208.3	Ground Surface							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	kN/m <sup>3</sup>	GR SA SI CL
0.0	Bedrock at surface														
	* Borehole dry														

**RECORD OF BOREHOLE No MLR8**

1 of 1

**METRIC**

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 362 N; 309 727 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Manual Sampling COMPILED BY T.X.  
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE				SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)							GR	SA	SI	CL
206.6	Ground Surface																				
0.0	Bedrock at surface																				
	* Borehole dry																				



**RECORD OF BOREHOLE No MLR10**

1 of 1

**METRIC**

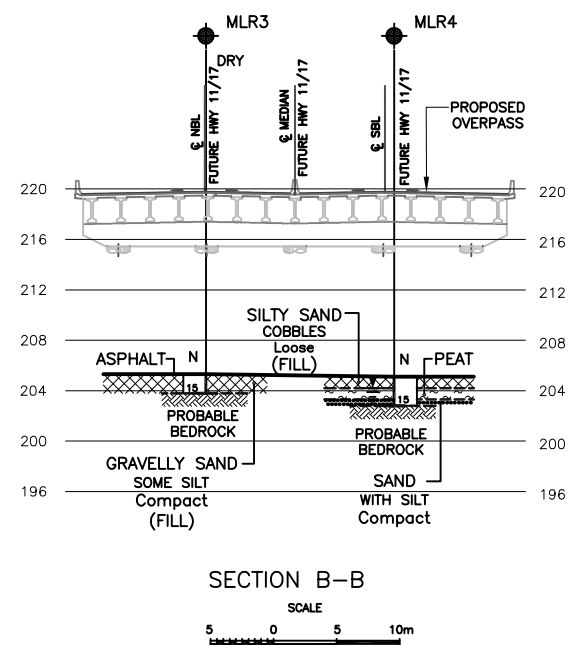
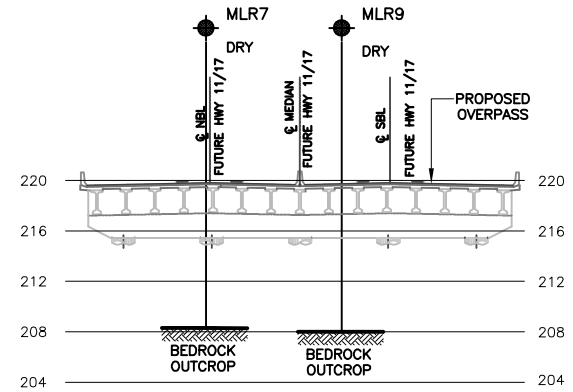
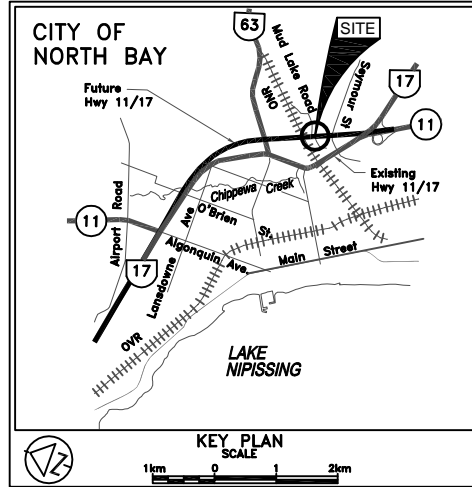
G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 355 N; 309 711 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Manual Sampling COMPILED BY T.X.  
 DATUM Geodetic DATE April 30, 2007 CHECKED BY C.N.

SOIL PROFILE				SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)							GR	SA	SI	CL
207.5	Ground Surface																				
0.0	Bedrock at 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  
DELANEY LAKE/MUD LAKE ROAD OVERPASS  
FUTURE HIGHWAY 11/17  
BOREHOLE LOCATIONS AND SOIL STRATA

**PML Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

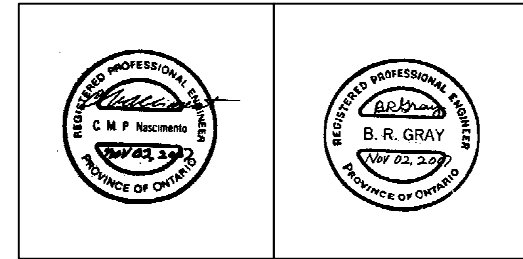


LEGEND

- 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

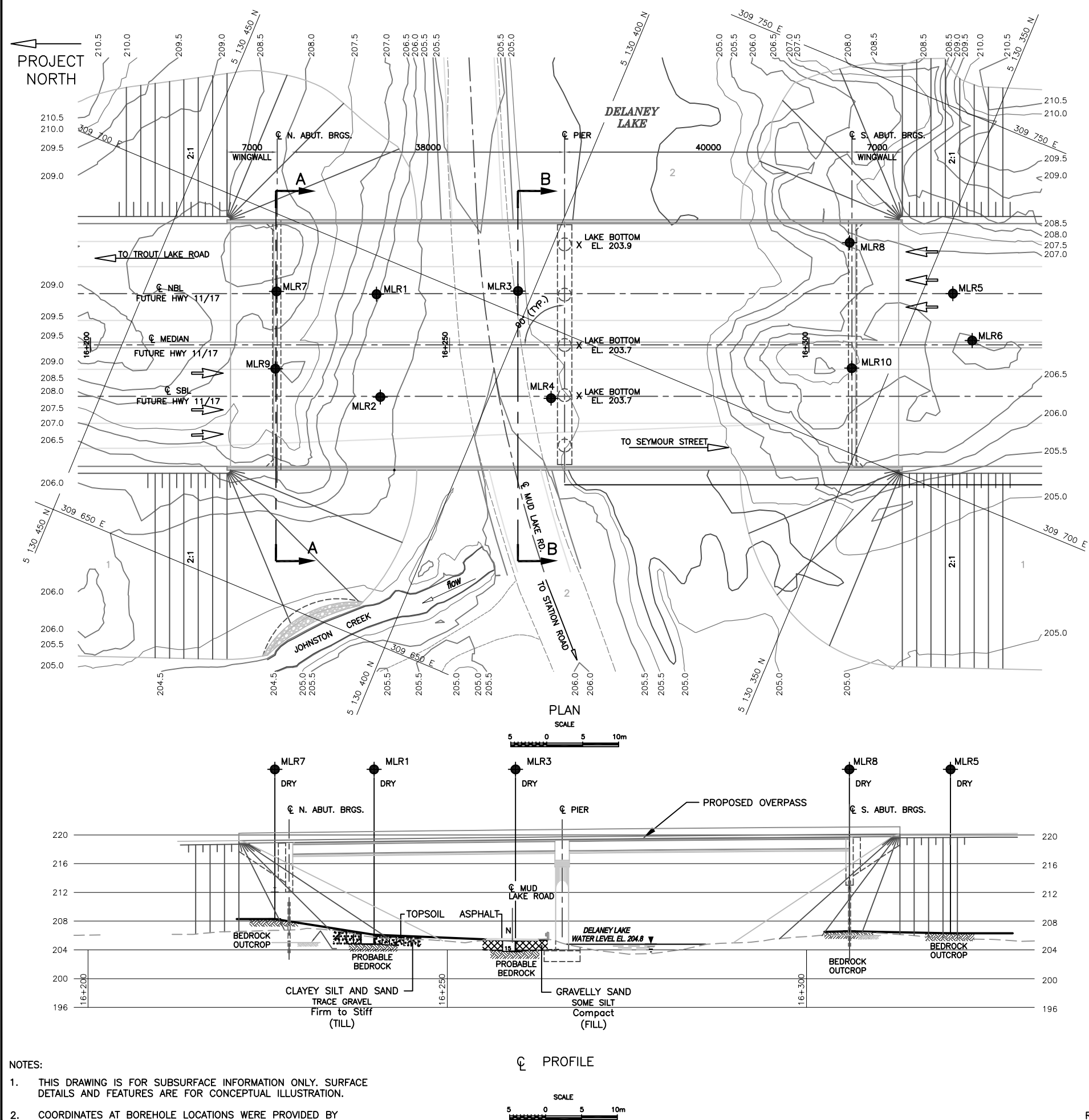
BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
MLR1	206.1	5 130 420	309 695
MLR2	205.7	5 130 414	309 682
MLR3	205.3	5 130 402	309 703
MLR4	205.1	5 130 392	309 691
MLR5	206.3	5 130 346	309 726
MLR6	206.8	5 130 341	309 723
MLR7	208.3	5 130 433	309 690
MLR8	206.6	5 130 362	309 727
MLR9	208.0	5 130 429	309 680
MLR10	207.5	5 130 355	309 711

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-115  
HWY No 11/17  
SUBM'D NSB [CHECKED NSB] DATE NOV. 02, 2007 SITE --  
DRAWN NA [CHECKED CN] APPROVED BRG DWG MLR-1



NOTES:  
1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.  
2. COORDINATES AT BOREHOLE LOCATIONS WERE PROVIDED BY DELBOSCO SURVEYING LTD.

REF No.: STANTEC DRAWING: 65000580\_MUD\_LAKE\_BRIDGE\_B7.dwg dated JULY 2007

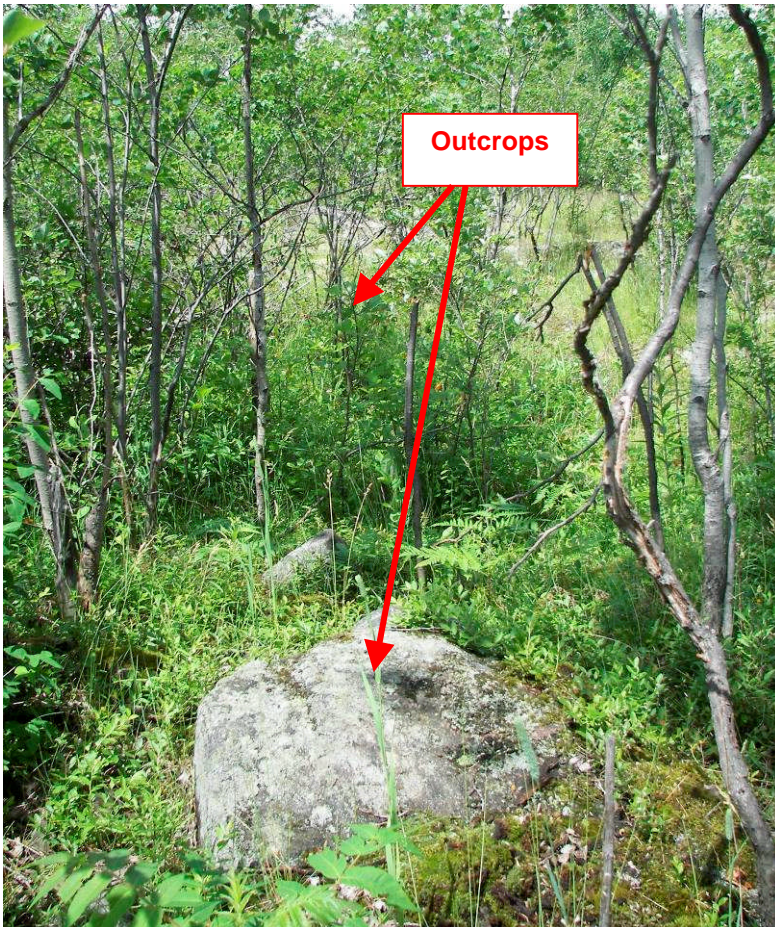


## **APPENDIX A**

### Site Photographs



Photograph 1: Looking south across Mud Lake Road and Delaney Lake at prominent rock outcrop where the south abutment is proposed. Scrap tires at right side of photograph mark south end of proposed pier. (June 19, 2007)



Photograph 2: Looking north along rock outcrops located on proposed north abutment location. Note typical shallow earth cover between outcrops. (June 19, 2007)



Photograph 3: Looking westerly along Mud Lake Road from edge of Station Road. Front end of pick-up truck is at approximate location of proposed Highway 11/17 centreline median; measuring tape extends along proposed SBL centreline. (April 30, 2007)



Photograph 4: Looking northerly from outcrop south of Delaney Lake. Approximate proposed SBL centreline marked by Hydro pole. Note fill near location of twin pipe culverts leading to Johnston Creek at left of photograph. (April 30, 2007)