



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT  
for**

**ONTARIO NORTHLAND RAILWAY OVERHEAD  
FUTURE HIGHWAY 11/17  
CITY OF NORTH BAY  
GWP 5748-04-00  
DISTRICT 54, SUDBURY, ONTARIO**

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**PART A**  
**PRELIMINARY FOUNDATION INVESTIGATION REPORT**  
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Future Highway 11/17  
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**1. INTRODUCTION**

This report summarizes the results of the preliminary foundation investigation carried out for the proposed construction of the Ontario Northland Railway (ONR) Overhead 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 overhead will carry the Future Highway 11/17 northbound and southbound lanes over the ONR tracks. Stantec prepared the preliminary General Arrangement (GA) drawing for the proposed 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 ONR Overhead.

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

**2. SITE DESCRIPTION AND GEOLOGY**

The proposed structure is located about 500 m west of the current intersection of the ONR tracks and the existing Highway 11/17 and about 1,000 m north from Highway 11 and Highway 17 South Junction interchange.



Land use in the vicinity of the site comprises the ONR railway station south of the tracks located about 150 m south of the ONR overhead structure, a commercial shopping mall (Northgate Square) north of the tracks and recently developed residential areas. Topographically, the structure site is located on relatively flat terrain north and south sides of the tracks. The existing embankment fill under the ONR tracks is about 3 to 4 m high. The Johnston Creek flows south-north through the ONR embankment south of structure site. In addition, swampy areas were observed along this creek. The ground cover comprises grasses, bushes and scattered trees. Photographs of the crossing are enclosed in Appendix A.

The project site is situated in the Algonquin Highlands physiography region within the Canadian Shield. The typical rock type in the project area is a metasedimentary migmatitic biotite gneiss of the Precambrian. The soil/bedrock interface is at variable depths, generally shallow and the bedrock outcrops are visible in the vicinity of the site.

### **3. INVESTIGATION PROCEDURES**

The subsurface investigation was carried out on April 19, 2007. No boreholes or test pits were carried out at near the abutment locations because clearing of the underground utilities was not provided by ONR. In view of the presence of shallow bedrock depths, the investigation included four sampled test pits, which were identified with the ONR prefix. The test pits were advanced to depths of 0.9 to 1.8 m at the locations shown on Drawing ONR-1. For uniformity purposes, the test pits were reported in MTO records of borehole, and the borehole type referred to an excavated test pit.

In view of the prevalent bedrock outcrop terrain at the site and preliminary phase of the investigation, PML substituted the programmed boreholes with test pits. The direct observation of the bottom of the test pits allows for a better definition of the bedrock surface and better distribution between cobbles and boulders than refusal to auger refusal in boreholes.

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.



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

The test pits were advanced using a CAT 312 track-mounted excavator, supplied and operated by a specialist contractor, working under the full-time supervision of a Field Supervisor from PML engineering staff. All test pits were backfilled in accordance with the MTO guidelines and MOE Reg. 903 for test pit abandonment procedures.

The relative density of the soil deposits was estimated by observation of the test pit walls and manual examination of the soil samples.

The groundwater conditions at the test pits 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. The laboratory test program comprised the following tests:

- Natural moisture content determinations (4)
- Grain Size analyses (4)

The results of the laboratory natural moisture content determinations, grain size analyses are shown on the Record of Borehole sheets. The envelope of grain size distribution charts is presented on Figure GS-ONR-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 pits locations and the preliminary layout of the ONR overhead structure as well as the longitudinal soil profile and cross-sections are presented on the Foundation Drawing ONR-1.

The stratigraphy revealed in the test pits generally comprised surficial topsoil overlying a continuous sand layer extending to inferred bedrock in three test pits. In one test pit, a localized deposit of sandy gravel containing boulders was encountered below topsoil and extended to the observed bedrock. The stratigraphy is consistent in the test pits dug on both sides of the railway embankment. These conditions should be verified during the Detail Design.

##### **4.2 Topsoil**

A surficial topsoil unit is present in all test pits. The unit is about 200 to 300 mm thick, extending to elevations 204.8 to 205.6.

##### **4.3 Sand**

A deposit of cohesionless sand with variable amounts of silt and gravel is present below the surficial topsoil layer and extended to the inferred bedrock at 1.5 to 1.8 m depths (elevations 204.4 to 203.3) in all test pits except ONR4. In addition, cobbles were found in the test pit ONR2. The deposits are judged to be in loose to compact conditions by the observation of test pit walls and soil samples.

The envelope of grain size distribution charts of representative samples of the sand is shown on Figure GS-ONR-1. The water content determinations values were varied widely from 8 to 21%.



#### **4.4 Sandy Gravel**

Below the topsoil in test pit ONR4, a deposit of cohesionless sandy gravel is encountered and extended to 0.9 m depth (elevation 204.2). This local unit contained boulders and was judged to be in a loose condition.

#### **4.5 Bedrock**

The bedrock surface was observed at the bottom of the test pits and by refusal to further advance of the excavations. The depth to the inferred bedrock varies from 0.9 to 1.8 m, elevations 204.2 to 203.3. According to local mapping, the bedrock likely comprises metasedimentary gneiss.

#### **4.6 Groundwater**

Groundwater was only measured in the test pit ONR2 at depth of 0.8 m (elevation 204.2). No water was observed the other test pits. The high water table along the east of the structure site is influenced by the existing Johnston Creek. Seasonal perched water is also anticipated at the site in topographic low areas controlled by the underlying bedrock. 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 Project Engineer. 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.



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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 Ontario Northland Railway (ONR) Overhead for the Future Highway 11/17. The recommendations are preliminary and based on the results of the limited subsurface investigation, as outlined in the Part A of this report.

Based on the preliminary GA drawing, the proposed bridge is approximately 40 m wide and will comprise a single span structure with length of 20 m between abutments. The deck of the proposed overhead is planned at about elevation 216, that is 12 m higher than the existing ground surface. The approach embankments will also be about 12 m high.

In summary, the soil stratigraphy in the test pits comprises units of topsoil overlying cohesionless loose to compact sand and sandy gravel extending to observed bedrock. These cohesionless deposits contain cobbles and boulders locally. The bedrock surface was observed at relatively shallow depths of 1.5 to 1.7 m (elevations 204.4 to 203.3) and 0.9 to 1.8 (elevations 204.2 to 203.8 in the test pit.

No boreholes or test pits were carried out within the footprint of the abutments due to lack of utilities clearances by ONR. The subsurface conditions at the abutments are likely similar to those encountered in the test pits dug for the current preliminary investigation in view of their uniformity. For the purpose of this preliminary report, the levels of the bedrock surface at the abutments were assumed to be consistent with those encountered in the test pits. The actual conditions should be investigated at the Detail Design stage.



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

## **6.2 Foundations**

### **6.2.1 General**

Although boreholes were not carried out in the abutment locations, we anticipate shallow depth to bedrock at these locations. Therefore, placing the foundations of the proposed overhead on spread footings constructed on bedrock is considered feasible. Considering the variable relative density and boulder content of the native soil and shallow soil cover, spread footings founded on native soil are not feasible.

Construction of spread footings founded on structural fill is not considered feasible because structural fill pad need wider area than available beside the existing track embankment.

The alternative scheme with pile foundations at the west and east abutments for conventional or integral abutment design alternatives is also feasible. It is considered, however that the use of conventional abutments founded on piles would not be cost effective in comparison with the shallow foundation alternatives. The pile alternatives would require a fill pad to drive the piles and (for integral abutments) the excavation of a trench in the bedrock where necessary to accommodate the minimum free pile length of 5 m. Restrictions for blasting bedrock trenches will likely apply to the areas near the ONR tracks.

We consider that drilled caissons to support the foundations are not practical for this site due to the presence of cobbles/boulders and the shallow and wet soil cover that would cause construction difficulties.

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



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 purpose of this preliminary report, the levels of the bedrock surface at the abutments were assumed to be consistent with those encountered at the test pits.

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

FOUNDATION ELEMENT	PML BH NO.	FOUNDING DEPTH (m)	FOUNDING ELEV.
North Abutment	ONR1	1.5	204.4
	ONR2	1.7	203.3
South Abutment	ONR 3	1.8	203.8
	ONR4	0.9	204.2

For preliminary design purposes, the bedrock is assumed to be medium to high strength and recommended preliminary geotechnical resistances for footing 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.

The recommended geotechnical resistance values apply to vertical and concentric loads only. The designer should consider the effects of inclined loads and eccentricity, as applicable.



The lateral loads imposed on the foundations will be partly resisted by the friction developed between the underside of the concrete footing and the bedrock. These forces should be calculated in accordance with the Canadian Highway Bridge Design Code (CHBDC) 2006 Edition. 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 normally be employed to support the foundation loads. Based on the GA drawing, however the space between the abutments and the ONR embankment is restricted and will not allow for the practical construction of the structural fill.

#### 6.2.4 Piles

For the preliminary design of piles for integral abutments, steel H-piles driven to refusal of bedrock should be used. The anticipated depths and elevations of the bedrock surfaces are the same as those indicate for spread footings. It is noted that no test pits or boreholes could be advanced at the proposed abutment locations. For the purpose of this preliminary report, the levels of the bedrock surface at the piers were assumed to be consistent with those encountered at the test pits. Consequently, the abutment foundations should be planned as spread footing on bedrock and designed as indicated previously.

For integral abutment design, the pile tips would have to be established at a level which provides a minimum 5 m long free pile length below the abutment stems. Considering the possible use of false abutments and the proposed 12 m high fill, the free pile length may be achievable without having to excavate trenches into the bedrock subject to structural detail design.



Based on the foregoing considerations, the piles would be founded at the same levels as the footings placed on bedrock. Based on assumed bedrock high strength, the preliminary factored axial resistance at ultimate limit states (ULS) for three pile sections noted below is considered to be appropriate:

Pile Section	Factored Axial Resistance at ULS (kN)
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.

For integral abutment design, two concentric CSPs that extend at least 3 m below of the abutments should be placed around the pile to create an annular space for integral abutments. The inner CSP should 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 of 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.



The pile length providing resistance for integral abutment piles should be considered the dimension below the annular space. The assessed lateral resistance provided by the CHBDC for the pile sections noted previously is as follows:

<b>Steel H-Pile, 310 x 79 Steel H-Pile, 310 x 110 Steel H-Pile, 310 x 132</b>	<b>GRANULAR BACKFILL</b>
Factored Lateral Resistance at ULS, kN	120
Lateral Resistance at SLS, kN	50

### **6.3 Approach Embankments**

Boreholes were not carried out for the approach embankments to the ONR overhead. We anticipate, however that the cohesionless soils underlain by shallow bedrock at the site will provide a competent bearing surface for the embankments. Further subsurface investigations should be carried out at these locations for Detail Design.

The approach embankments should be designed and constructed in accordance with OPSPD-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 rock fill, 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 adjacent to the abutments will about to 12 m high and is likely be founded on bedrock or over shallow cohesionless soil cover, subject to the results of Detail Design. The embankments 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 depends on the workmanship employed by the contractor and, if placed in 200 mm thick lifts compacted to 100% of standard Proctor maximum dry density in accordance with the requirements of SP 206S03 and OPSS 501 (Method A), the estimated settlement should be in order of 30 mm. These estimated total settlements of the approach fill surface near the abutments should be essentially complete within 1 to 2 months after placement of the fill.

The approach embankment remote from the abutments may be constructed using earth or rock fill placed on the native soils. Settlements of these materials also placed according to the OPS Standards mentioned above are considered to be within acceptable limits.

## **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 sand encountered in the borehole is considered Type 3 soils. The rock is considered Type 1 soil, according to OHSA. The cohesionless soils below the groundwater (test pit ONR 2) should be classified as Type 4 soil.



#### 6.4.2 Railway Track Protection

According to preliminary GA drawing, the abutments are located on or adjacent to the ONR track embankment fill. It is anticipated that a suitable track protection scheme following SP 105S19 will be required to support the track embankment and walls of the excavation during construction.

Several protection scheme alternatives such as sheet piling, sheeting supported by rakers or bracing, cantilever or anchored soldier piles and lagging may be considered. For preliminary design purposes, the track protection schemes should be designed for performance level 1b to prevent movement of the existing track embankment. The contractor is responsible for the selection, detailed design and performance for the track protection scheme.

#### 6.4.3 Groundwater Control

Due to presence of Johnston Creek and locally associated swampy areas to the south of the structure site, it is expected that a high water table conditions exist at this location although groundwater was only found in one of the test holes. Further investigations should be carried out at these locations for Detail Design.

The groundwater control during construction for excavation of existing soil cover and bedrock trenches should consider temporary earth dams, crib walls or short sheet piling east of the creek and swampy areas. Once the work areas are confined, the groundwater should be readily handled by conventional sump pumping techniques.





## 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  $\emptyset$  = angle of internal friction of retained soil  
 $\delta$  = 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, $\emptyset$ (degrees)	35
Unit weight, $\gamma$ ( $\text{kN/m}^3$ )	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.



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. SCOPE OF ADDITIONAL FOUNDATION INVESTIGATION**

Since no boreholes were carried out within the footprint of abutments, the recommendations in this report are preliminary and based on PML's interpretation of the factual information obtained from a limited number of test pits 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 about 40 m wide structure, the limited number of borehole data and a visual site assessment for the overhead structure, the recommended scope of the foundation investigation for Detailed Design is as follows:

- Boreholes should be carried out at the centre (1), 1.0 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 the abutment footprint of each to define the surface of the bedrock (total of 9 for each abutment). At least three of the boreholes at the corners and the centre of abutment footing should be cored a minimum of 3.0 m into the bedrock.



- Boreholes should be carried out at each ends of the 13 m long retaining walls (total of 4 boreholes) planned adjacent to the wing walls of the structure. These boreholes should be cored a minimum of 3.0 m into the bedrock.
- Two boreholes should be carried out for each approach embankments (total of 4) about 20 m behind each abutment.
- Four boreholes should be carried out between the abutments/retaining walls and the creek to establish the groundwater conditions along the west side of the site.

## **8. DISCUSSION OF FOUNDATION ALTERNATIVES**

### **8.1 Advantages and Disadvantages of Foundation Alternatives**

It is considered that spread footings on structural fill or caissons are not suitable or practical at this site. The following table summarizes the advantages and disadvantages and inferred risks/consequences of each of the feasible foundation alternatives for the proposed ONR overhead at Highway 11/17.

#### **Spread footings 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
- Requires unwatering at foundation locations
- Requires removal of all boulders from foundation footprint



### **Driven Piles**

#### **Advantages**

- Allows integral abutment design and construction
- Lower long-term maintenance costs of expansion joints for with integral abutment design

#### **Disadvantages**

- Requires blasting of rock trenches
- Blasting maybe restricted near the ONR embankment
- More costly than spread footings alternative
- Heavy equipment for pile driving is required.
- Installation may be difficult due to boulders at soil/rock interface

### **8.2 Preferred Foundation Option Considerations**

From the foundation perspective, spread footings founded on bedrock 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 spread footing foundation. 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 selected foundation alternative also depends on other considerations, such as structural design and road grades, which are being evaluated separately by Stantec.

## 9. CLOSURE

The Preliminary Foundation Design part of this 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 positioned above the printed name.

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



A handwritten signature in blue ink, appearing to read "Brian R. Gray", is positioned above the printed name.

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

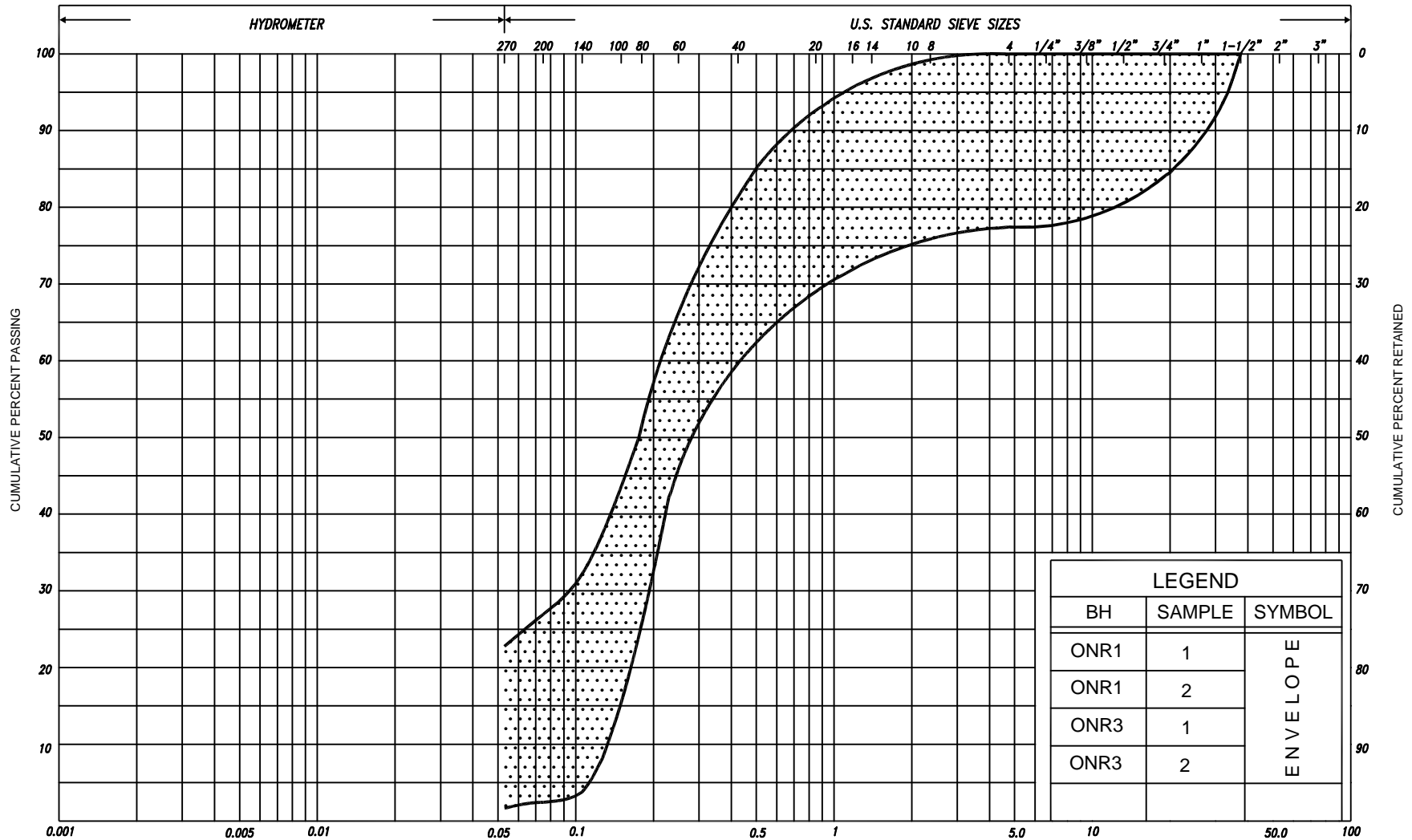


CN/BRG:nb-lnr



**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 105S19	Construction Specification for Protection Systems	November 2006
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			COBBLES	UNIFIED	
CLAY	FINE		MEDIUM	COARSE	FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.
	SILT				SAND									U.S. BUREAU
CLAY		SILT			V. FINE	FINE	MED.	COARSE	GRAVEL					
					SAND									

## GRAIN SIZE DISTRIBUTION

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

FIG No. GS-ONR-1

HWY: 11/17

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



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Transportation  
Ontario

## 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
$\sigma$	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

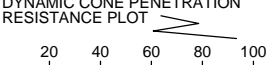




### MECHANICAL PROPERTIES OF SOIL

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$m^2/s$	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

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



<b>RECORD OF BOREHOLE No ONR1</b> <span style="float: right;">1 of 1</span> <b>METRIC</b>																		
G.W.P. 5748-04-00			LOCATION Co-ords. 5 130 609 N; 309 618 E.			ORIGINATED BY M.R.												
DIST Sudbury HWY 11/17			BOREHOLE TYPE Excavator			COMPILED BY T.X.												
DATUM Geodetic			DATE April 19, 2007			CHECKED BY C.N.												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		SHEAR STRENGTH kPa			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE						W <sub>p</sub>	W	W <sub>L</sub>	γ	GR SA SI CL	
205.9	Ground Surface							20	40	60	80	100						
0.0	Topsoil		1	CS	-									o				9 64 (27)
0.3	Sand, with silt trace gravel, trace clay																	
	Compact Grey Wet with gravel		2	CS	-		205							o				23 52 (25)
204.4	Moist																	
1.5	End of borehole																	
	Refusal on probable bedrock																	
	* Borehole dry																	

**METRIC**

20  
15 — 5 (%) STRAIN AT FAILURE  
10

**RECORD OF BOREHOLE No ONR3**

1 of 1

**METRIC**

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 568 N; 309 635 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Excavator 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 <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 (%)		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE													
205.6	Ground Surface							20	40	60	80	100						GR	SA	SI	CL
0.0	Topsoil																				
0.3	Sand, some silt trace gravel, trace clay		1	CS	-		205								o				7	77	(16)
	Loose    Grey    Moist		2	CS	-										o				0	98	(2)
	trace silt																				
203.8	Brown    Wet						204														
1.8	End of borehole Refusal on probable bedrock																				
	*    Borehole dry																				

**RECORD OF BOREHOLE No ONR4**

1 of 1

**METRIC**

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 554 N; 309 622 E. ORIGINATED BY M.R.  
 DIST Sudbury HWY 11/17 BOREHOLE TYPE Excavator 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 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 (%)				
205.1	Ground Surface					*	205	<div><div></div><div></div><div></div><div></div><div></div><div></div></div>					<div><div></div><div></div><div></div><div></div><div></div><div></div></div>				GR SA SI CL
0.0	Topsoil							20	40	60	80	100	20	40	60		
0.2	Sandy gravel, num. boulders																
204.2	Loose Dark Wet brown																
0.9	End of borehole Refusal on probable bedrock																
	* Borehole dry																

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

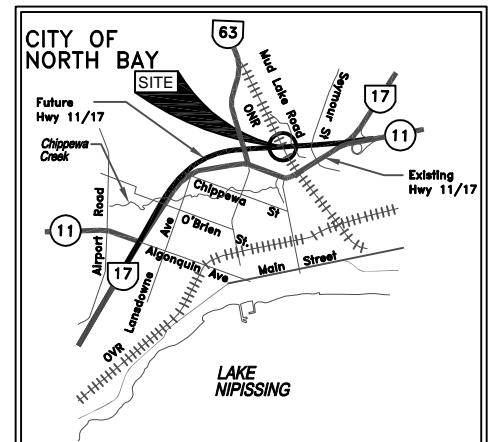
CONT No  
GWP No 5748-04-00

ONTARIO NORTHLAND RAILWAY(ONR)  
OVERHEAD  
FUTURE HIGHWAY 11/17  
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET

**PML Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



KEY PLAN  
SCALE  
1km 0 1 2km

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
ONR1	205.9	5 130 609	309 618
ONR2	205.0	5 130 595	309 605
ONR3	205.6	5 130 568	309 635
ONR4	205.1	5 130 554	309 622

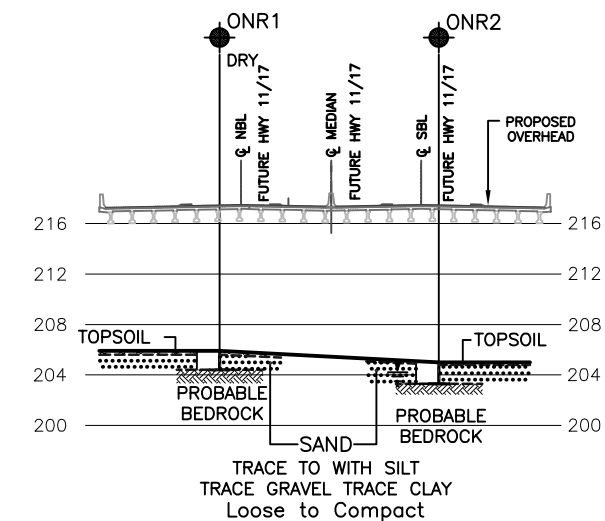
NOTE

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

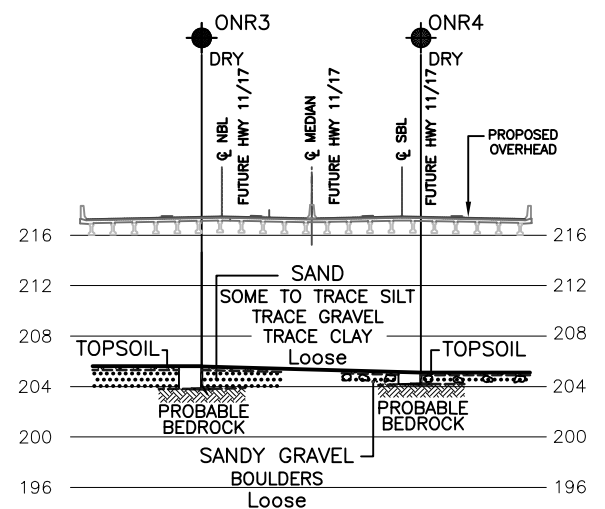
REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-117

HWY No	11/17	DIST	54
SUBM'D	NSB	CHECKED	NSB
DATE	NOV. 02, 2007	SITE	---
DRAWN	NA	CHECKED	CN
APPROVED	BRG	DWG	ONR-1

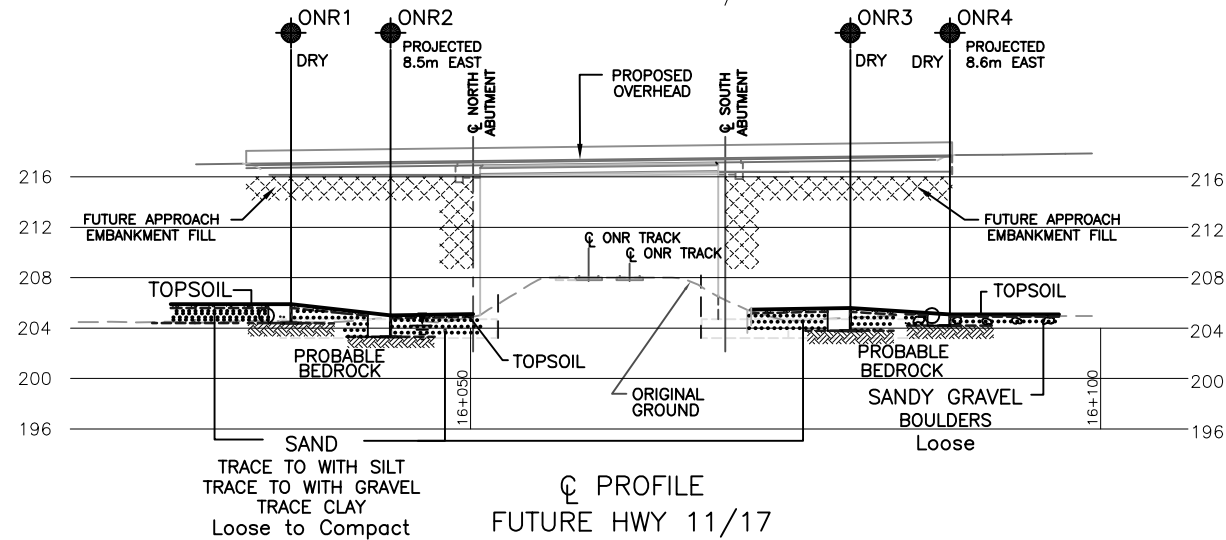
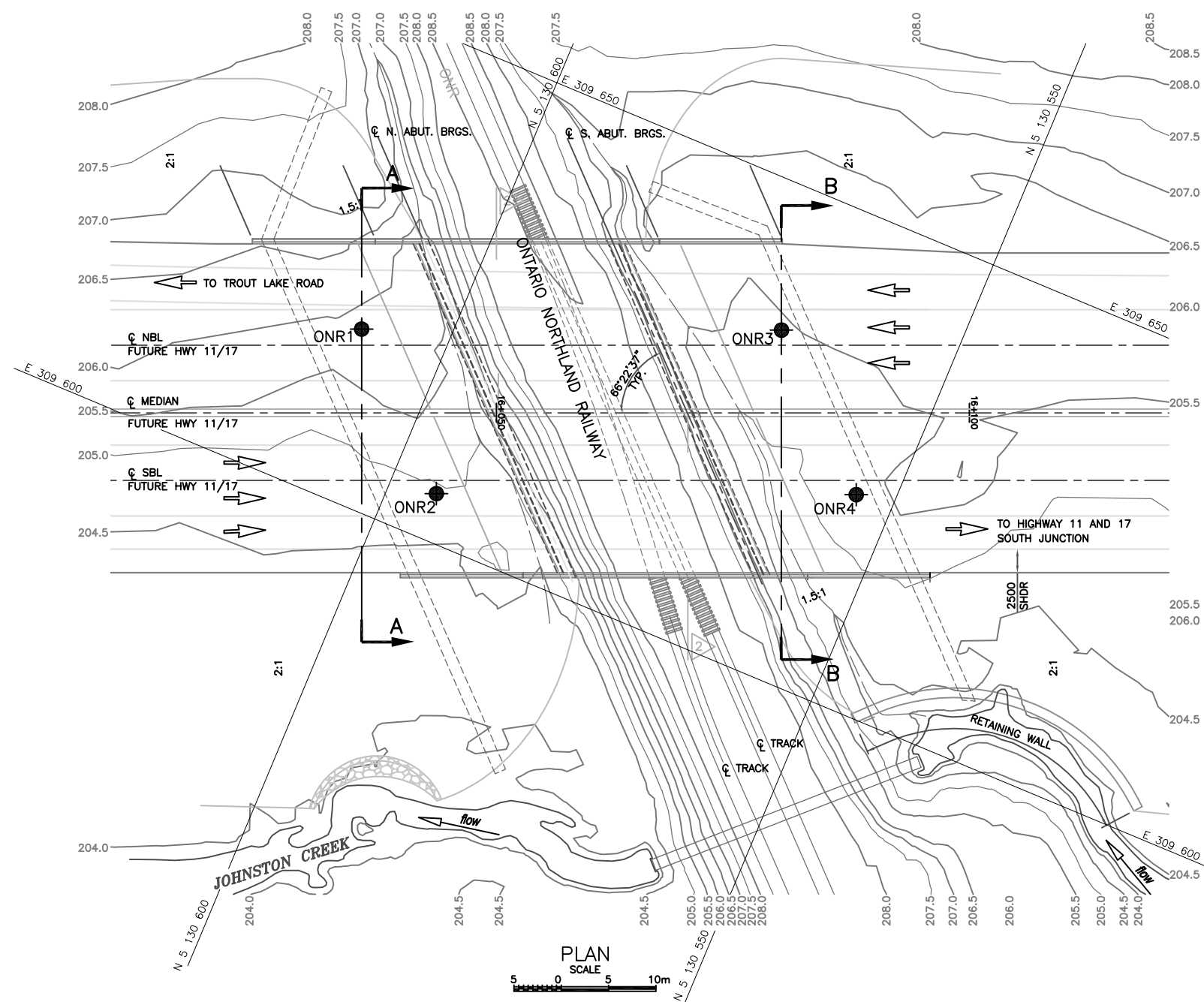


SECTION A-A



SECTION B-B

SCALE  
5 0 5 10m



Q PROFILE  
FUTURE HWY 11/17

SCALE  
5 0 5 10m

NOTES:

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

REF No.: STANTEC DRAWING: 65000580\_ONR\_D1.dwg  
dated JULY, 2007



## **APPENDIX A**

### Site Photographs



Photograph 1: Looking northerly from south of railway tracks near Ontario Northland Railway station. Overhead crossing is proposed to right of hydro pole at right corner of photograph. Note wetland in foreground. (June 19, 2007)



Photograph 2: Looking easterly from Ontario Northland Railway station platform. Overhead location is at the start of the railway siding. (June 19, 2007)





Photograph 3: Looking southeasterly from edge of Northgate Square Mall parking lot, north of ONR railway tracks. Overhead crossing is proposed to left of Hydro pole on left of photograph. Note Johnston Creek, wetland and boulders in foreground. (June 19, 2007)