



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

**SEYMOUR STREET OVERPASS
FUTURE HIGHWAY 11/17
CITY OF NORTH BAY
GWP 5748-04-00
DISTRICT 54, SUDBURY, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

- 3 cc: Stantec Consulting Ltd. for distribution to MTO,
Project Manager + one digital copy (PDF format)
- 1 cc: Stantec Consulting Ltd. for distribution to MTO,
Pavements and Foundations Section + one digital copy
(PDF format), and Drawing (AutoCAD format)
- 2 cc: Stantec Consulting Ltd. + one digital copy (PDF format)
- 1 cc: PML Toronto

PML Ref.: 05TF058B
Index No.: 078FIDR
Geocres No.: 31L-114
November 2, 2007



TABLE OF CONTENTS

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

| | |
|---|---|
| 1. INTRODUCTION | 1 |
| 2. SITE DESCRIPTION AND GEOLOGY | 1 |
| 3. INVESTIGATION PROCEDURES | 2 |
| 4. SUMMARIZED SUBSURFACE CONDITIONS | 3 |
| 4.1 General | 3 |
| 4.2 Fill | 4 |
| 4.3 Topsoil/Peat | 4 |
| 4.4 Sandy Gravel to Sand | 4 |
| 4.5 Bedrock | 4 |
| 4.6 Groundwater | 5 |
| 5. MISCELLANEOUS | 5 |

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

| | |
|--|----|
| 6. ENGINEERING RECOMMENDATIONS | 6 |
| 6.1 General | 6 |
| 6.2 Foundations | 7 |
| 6.2.1 General | 7 |
| 6.2.2 Spread Footings on Bedrock | 7 |
| 6.2.3 Spread Footings on Structural Fill | 8 |
| 6.2.4 Piles | 9 |
| 6.3 Approach Embankments | 11 |
| 6.4 Excavation Considerations | 12 |
| 6.5 Lateral Earth Pressures | 13 |



| | |
|---|----|
| 7. SCOPE OF ADDITIONAL FOUNDATION INVESTIGATION | 14 |
| 8. DISCUSSION OF FOUNDATION ALTERNATIVES | 15 |
| 8.1 Advantages and Disadvantages of Foundation Alternatives | 15 |
| 8.2 Preferred Foundation Option Considerations | 16 |
| 9. CLOSURE | 16 |

Table 1 – List of Standard Specifications Referenced in Report

Figure GS-SS-1 – Results of Grain Size Distribution Analyses

Explanation of Terms Used in Report

Record of Borehole Sheets

Drawing SS-1 – Borehole Locations and Soil Strata

Appendix A – Site Photographs

**PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT**

for
Seymour Street 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 the preliminary foundation investigation carried out for the proposed construction of the Seymour Street Overpass at the Future Highway 11/17, in the City of North Bay, 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 single span overpass will carry the Future Highway 11/17 northbound and southbound lanes over Seymour Street. Stantec prepared the preliminary General Arrangement (GA) drawing for the structure that was dated July 2007.

According to preliminary GA drawing, the Future Highway 11/17 will be a four-lane highway running north-south at the Seymour Street 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 Future Highway 11/17 contemplated structure is located about 300 m east of the existing Highway 11/17 and Seymour Street intersection. The site is about 600 m north from the existing Highway 11 and Highway 17 South Junction interchange.



Land use in the vicinity of the site comprises transportation facilities, which include a railway station, Greyhound Bus terminal and a gas service station. Industrial and commercial buildings are also present. Topographically, the structure site is located on a relatively flat terrain between high rock outcrops to the north and swamp ground to the south. Fill was placed previously across the site to construct the Seymour Street embankment and for adjacent parking lots and driveways of existing businesses. The ground cover beyond the Seymour Street embankment comprises grasses, typical swamp vegetation and isolated stands of trees and bush.

Photographs of the crossing site are enclosed in Appendix A.

The project 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 bedrock outcrops are visible at the site.

3. INVESTIGATION PROCEDURES

The subsurface investigation was carried out on April 30, 2007. In view of the prevalent wet ground at the site and obvious presence of bedrock at shallow depths, the investigation comprised four sampled test pits which were identified with the prefix SS. The test pits were advanced to depths of 0.3 to 2.0 m at the locations shown on Drawing SS-1. For uniformity purposes, the test pits were reported in MTO Records of Boreholes, having the borehole type referred to an excavated test pit.

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 the test hole locations. PML cleared the locations of the test holes for the presence of underground services and utilities. The elevations in this report are expressed in metres.



The test pit were dug 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 the PML engineering staff. All test holes were backfilled in accordance with the MTO guidelines and MOE Reg. 903 for test pit abandonment procedures.

The relative density of the encountered soils were assessed based on the inspection of the test pit walls and recovered chunk samples.

The groundwater conditions at the test pit 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 Borehole sheets.

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 results of the laboratory natural moisture content determinations and one grain size analysis are shown on the Record of Borehole sheets. The grain size distribution chart is presented on Figure GS-SS-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 borehole locations and the preliminary layout of the Seymour Street Overpass as well as the longitudinal soil profile and cross-sections are presented on the Foundation Drawing SS-1.

The soil stratigraphy in the test pits comprises units of surficial topsoil underlain by cohesionless sandy gravel and sand or a localized fill unit overlaying peat. The sand or peat/topsoil mantle probable bedrock at three locations. A bedrock outcrop exists in one test pit location.



4.2 Fill

A 1.2 m thick fill layer is present in test pit SS4 (dug at the south abutment) and extended to 1.2 m depth, elevation 209.1. This unit is composed of boulders and cobbles mixed with gravel and judged to be in a loose condition by observation of the test pit walls.

4.3 Topsoil/Peat

The surficial units of topsoil 300 mm thick are present in test pits SS1 and SS2 extending to probable bedrock in test pit SS1 at 0.3 m depth, elevation 210.0. These deposits were excavated at the north abutment location.

Peat is identified below the fill layer in test pit SS4 at depth 1.2 m (elevation 209.1) and extending to probable bedrock at a depth of 2.0 m (elevation 208.3). The water content determination on one peat sample was 329%.

4.4 Sandy Gravel to Sand

A deposit of cohesionless sandy gravel trace silt containing boulders was found at a depth 0.3 m (elevation 210.0) and extended to a depth of 1.0 m (elevation 209.3). This sandy gravel is underlain by a sand unit extending to probable bedrock at a depth of 1.2 m (elevation 209.1).

The grain size distribution chart of a representative sample of the sand is shown on Figure GS-SS-1. The water content determination on this sand sample was 18%.

4.5 Bedrock

Bedrock is present at the surface at the location of test pit SS3. Bedrock was observed at the bottom of test pits and inferred by refusal to further advance of the excavations. The depth to the bedrock/inferred bedrock surface varies from 0.0 to 2.0 m, elevations 208.3 to 212.3. According to local mapping, the bedrock likely comprises metasedimentary gneiss.



Local variations in the bedrock surface within the immediate vicinity of the test pits were up to 300 mm, characteristic of the knobby nature of the bedrock.

4.6 Groundwater

Groundwater was measured only in test pits SS1 and SS4 at ground surface (elevation 210.3). No water was observed in boreholes SS2 and SS3. It is believed that the surface water is a perched condition 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 Foundation Engineer. Bruman Leasing Ltd. supplied the excavation equipment. 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
Seymour Street Overpass
Future Highway 11/17
City of North Bay
GWP 5748-04-00
District 54, Sudbury, Ontario

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 Seymour Street 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 Seymour Street Overpass will be a single span structure with a length of about 41.0 m between abutments. The proposed 32 m wide single bridge deck will carry five lanes of traffic (four lanes and one ramp) for the Future Highway 11/17 and will be 8 m high above Seymour Street at about elevation 220.0. The approach embankments will be also about 8 m high at the abutments.

In summary, the soil stratigraphy is comprised of localized layers of surficial fill or topsoil overlying cohesionless sandy gravel/sand or peat over inferred bedrock. Cobbles and boulders were encountered in the test pits. The bedrock/inferred bedrock was found at relatively shallow depths of 0.3 to 1.2 m (elevations 210.0 to 209.1) and 0.0 to 2.0 m (212.3 to 208.3) at the north and south abutments, respectively.

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, it is considered that placing the structure foundations on spread footings bearing on bedrock or structural fill is 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. Normally this alternative will require the excavation of a trench in the bedrock where necessary to accommodate the minimum free pile length of 5 m below the abutment stem.

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

| FOUNDATION ELEMENT | PML BH NO. | FOUNDING DEPTH (m) | FOUNDING ELEV. |
|--------------------|------------|--------------------|----------------|
| North Abutment | SS1 (east) | 0.3 | 210.0 |
| | SS2 (west) | 1.2 (sloping) | 209.1 |
| South Abutment | SS3 (east) | 0.0 | 212.3 |
| | SS4 (west) | 2.0 | 208.3 |



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 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 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 rock fill. However, rock fill 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 on structural fill.

6.2.4 Piles

For the preliminary design of piles for integral abutments, steel H-piles driven to refusal on the bedrock underlying the site should be used. The anticipated depths and elevations of the bedrock surface are the same as indicate for spread footings. Based on the preliminary GA drawing indicating the abutment stems extending to about elevation 212.0, the pile tips will have to be established at about elevation 207.0, subject to structural detail design. Trenches 1.3 to 5.3 m deep (at the test pit locations) will have to be excavated into the bedrock to provide the minimum 5.0 m pile length required below the abutment stem.



Based on high bedrock strength assumed at the base of the excavation rock trenches, 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.

The compacted granular fill pad used for the installation of the abutment piles should comprise OPSS Granular A or Granular B Type II or Type III materials to allow installation of the piles without damage. In addition, 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 be filled with sand meeting the gradation requirements of Granular B Type 1. 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:

| Steel H-Pile, 310 x 79 Steel H-Pile, 310 x 110 Steel H-Pile, 310 x 132 | GRANULAR BACKFILL |
|---|------------------------------|
| 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 Seymour Street Overpass. We anticipate, however that construction of the approach embankments is likely to be founded on bedrocks, in view of the shallow bedrock conditions encountered at the north and south abutments. Further subsurface investigations should be carried out at these locations for final 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 or rock fill, respectively, construction of 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 be about 8 m high. 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 settlements should be in the order of 20 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.

Remote from the abutments, where the embankments are constructed with rockfill placed in accordance with the same standards, estimated settlements are expected to be in the 40 mm range. About 50% of the settlements are expected to occur within the first 6 to 12 months after placement and the remaining during the following 5 to 10 years.

6.4 Excavation Considerations

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 boreholes are considered Type 3 soils. The rock is considered Type 1 soil according to OHSA.

Groundwater observed in the test pit during the course of field work was at ground surface essentially perched on the underlying bedrock surface. Groundwater control during construction for excavation of existing soil cores and bedrock trenches should consider temporary earth dams in the generally low lying swampy ground. 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)
 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 ϕ = angle of internal friction of retained soil
 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, ϕ (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. 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 interpretation and recommendations are only provided for planning purposes and feasibility studies.

Based on the 32 m wide structure, the limited number of borehole data and a visual assessment for 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 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 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 Seymour Street Overpass at Highway 11/17.

Spread 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 founding level subgrade on bedrock

Footings on Structural Fill

Advantages

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

Disadvantages

- More costly than spread footings founded 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 cost expansion joints with integral abutment design

Disadvantages

- More costly than spread footings alternative
- Heavy equipment for pile driving is required
- Requires bedrock excavation to achieve minimum required pile length



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 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 abutment alternative on spread footings foundation for construction. 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

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 printed name.

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 printed name.

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

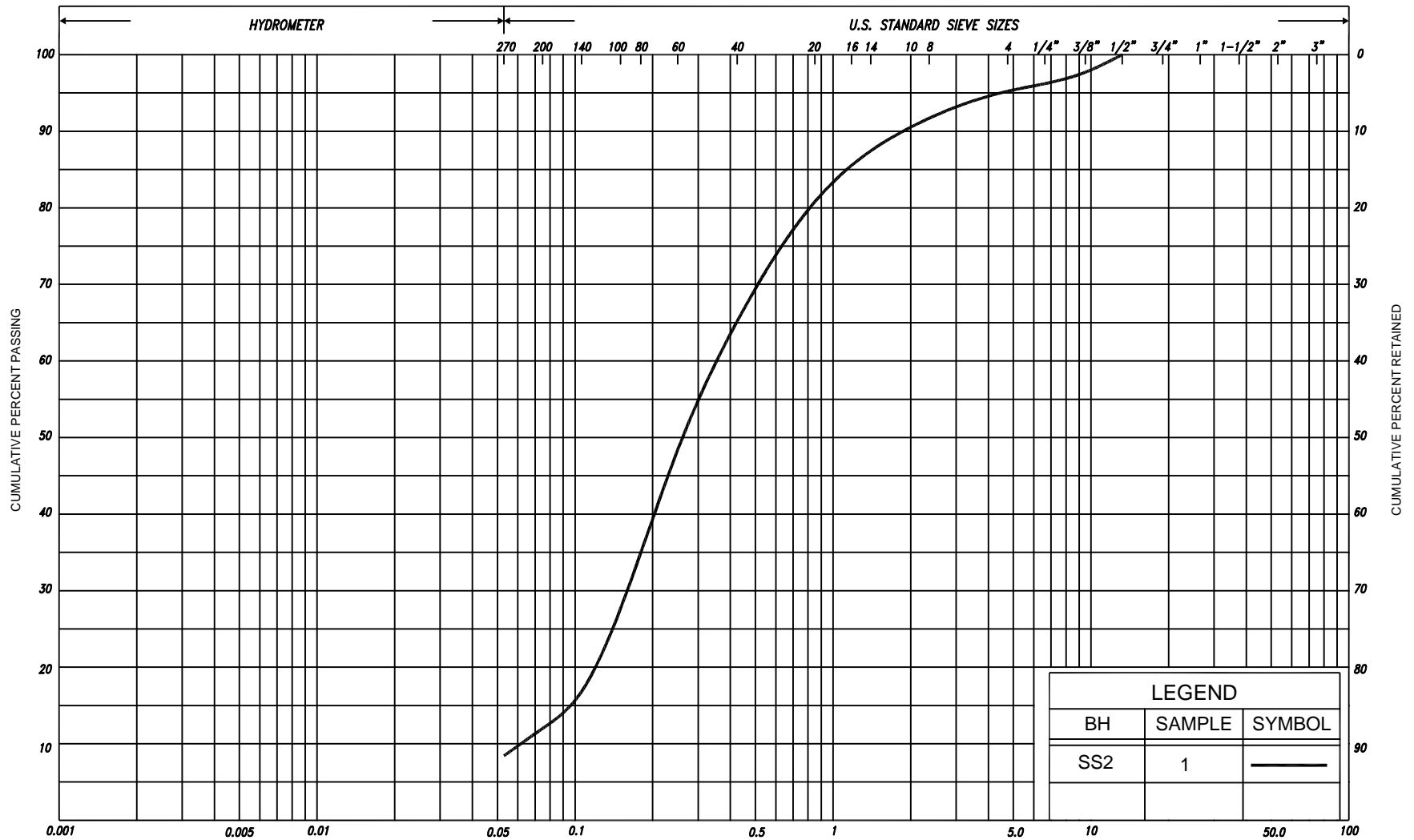
CN/BRG:nb-mi





TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

| DOCUMENT | TITLE | DATE |
|-----------------|--|---------------|
| 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 | | | |
| | | | | SAND | | | | | | GRAVEL | | | | | | | |
| CLAY | FINE | | MEDIUM | | COARSE | | FINE | | MEDIUM | | | | COARSE | | COBBLES | M.I.T. | |
| | SILT | | | | | | | | | | | | | | | | |
| CLAY | | SILT | | | V. FINE | | FINE | | MED. | | | | COARSE | | GRAVEL | | |
| | | | SAND | | | | | | | | | | | | | | |

GRAIN SIZE DISTRIBUTION

SAND, some silt, trace gravel

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 |
| 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 | | | | | | |

METRIC

20
15 — 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No SS2

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 156 N; 309 788 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 _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|--|----|----|-----|--|------------------------------------|-------------------------------------|-----------------------------------|--|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | | | | |
| 210.3 | Ground Surface | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | |
| 0.0 | Topsoil | | | | | | | | | | | | | | | | |
| 0.3 | Sandy gravel num. boulders, trace silt | | | | | | | | | | | | | | | | |
| 209.3 | Dark Wet | | | | | | | | | | | | | | | | |
| 1.0 | brown | | 1 | CS | - | | | | | | | | | | | 5 83 (12) | |
| 209.1 | Sand | | | | | | | | | | | | | | | | |
| 1.2 | some silt, trace gravel | | | | | | | | | | | | | | | | |
| | Compact Grey Wet | | | | | | | | | | | | | | | | |
| | End of borehole | | | | | | | | | | | | | | | | |
| | Refusal on probable bedrock | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No SS3

1 of 1

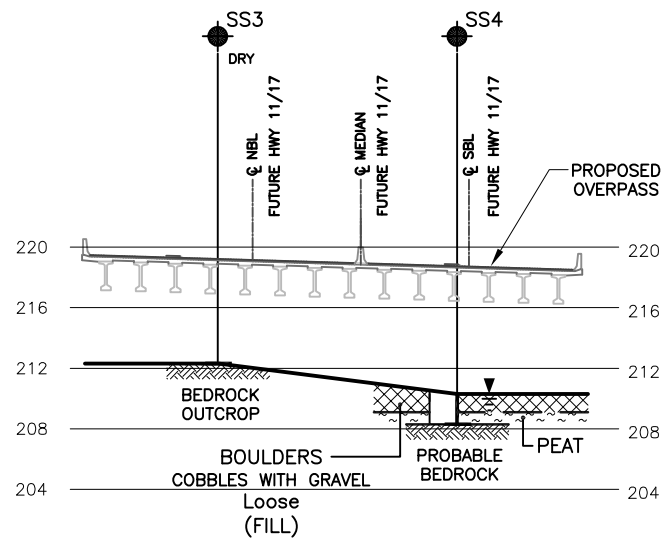
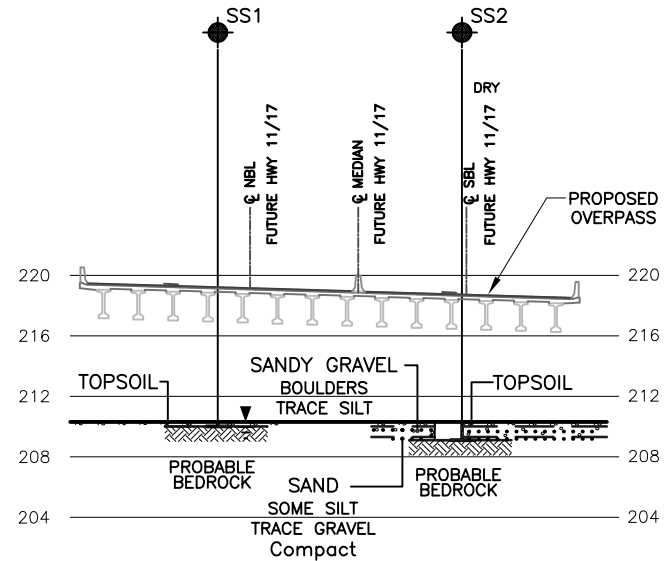
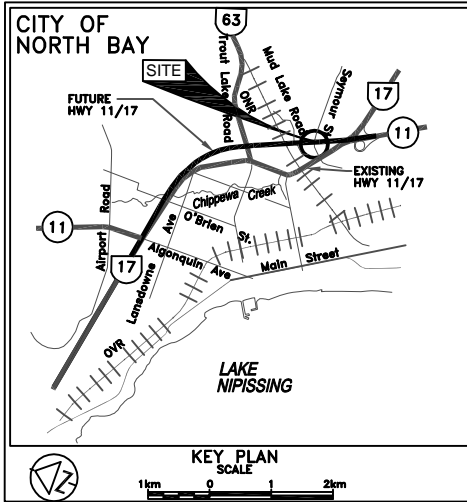
METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 130 120 N; 309 818 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 _p | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | |
|---------------|--------------------|------------|--------|---------|------------|--|---------------------------------|-----------------|---|----|-------------------|----|-----|------------------------------------|-------------------------------------|-----------------------------------|--|--|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | | | | | | | |
| | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE | | | | | | | | | | | | | | |
| 212.3 | Ground Surface | | | | | | | | 20 | 40 | 60 | 80 | 100 | | 20 | 40 | 60 | | | |
| 0.0 | Bedrock at surface | | | | | | | | | | | | | | | | | | | |
| | * Borehole dry | | | | | | | | | | | | | | | | | | | |

METRIC

20
15 — 5 (%) STRAIN AT FAILURE
10



LEGEND

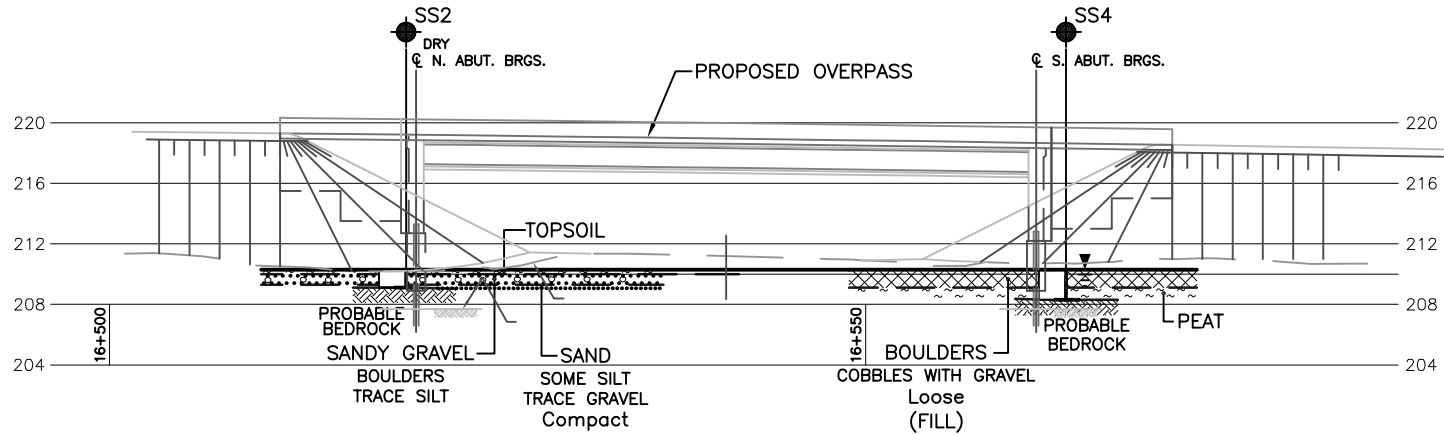
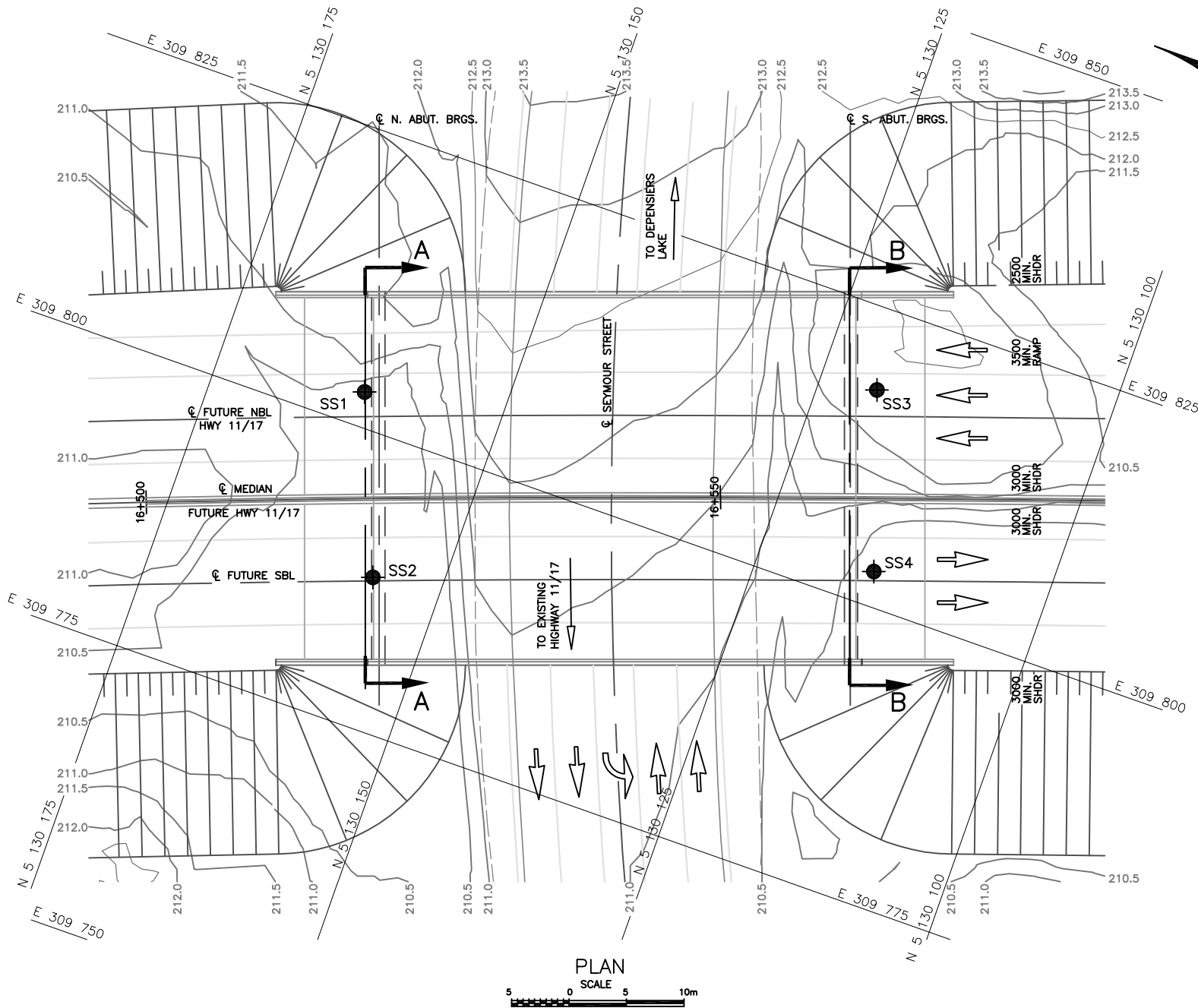
- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60° Cone, 475 J / blow)
- W L at time of investigation April 2007
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

| BH No | ELEVATION | COORDINATES | |
|-------|-----------|-------------|----------|
| | | NORTHINGS | EASTINGS |
| SS1 | 210.3 | 5 130 162 | 309 803 |
| SS2 | 210.3 | 5 130 156 | 309 788 |
| SS3 | 212.3 | 5 130 120 | 309 818 |
| SS4 | 210.3 | 5 130 115 | 309 803 |

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-114 |
| HWY No | 11/17 |
| SUBM'D | NSB |
| CHECKED | NSB |
| DATE | NOV. 02, 2007 |
| SITE | --- |
| DRAWN | NA |
| CHECKED | CN |
| APPROVED | BRG |
| DWG | SS-1 |



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.



APPENDIX A

Site Photographs



Photograph 1: Looking west along Seymour Street at location of proposed overpass. South abutment is located between the two nearest hydro poles. Note embankment through low-lying area for existing Seymour Street platform. (June 19, 2007)



Photograph 2: Looking northeast across Seymour Street from northeast corner of ESSO Gas Station property. North abutment is partially located in shallow swamp at the upper left of photograph. Note large boulders in foreground amongst weeds. (June 19, 2007)