



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for
HIGHWAY 11 AND HIGHWAY 17 INTERCHANGE UNDERPASS WIDENING
FUTURE HIGHWAY 11/17
CITY OF NORTH BAY
GWP 5748-04-00, SITE 43-350
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

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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	4
4.1 Fill.....	4
4.2 Silty Sand Till	5
4.3 Bedrock.....	5
4.4 Groundwater	6
5. MISCELLANEOUS	6

PART B - PRELIMINARY FOUNDATION DESIGN REPORT

6. ENGINEERING RECOMMENDATIONS.....	7
6.1 General	7
6.2 Foundations	8
6.2.1 Spread Footings.....	8
6.2.2 Piles	9
6.2.3 Caissons	9
6.3 Approach Embankments	10
6.4 Construction Considerations	10
6.4.1 Excavations.....	10
6.4.2 Groundwater Considerations	11
6.5 Lateral Earth Pressures.....	12
7. ADDITIONAL STUDIES	13
8. DISCUSSION OF FOUNDATION ALTERNATIVES.....	14
9. CLOSURE	14



Table 1 – List of Standard Specifications Referenced in Report

Figures GS-TWL-1 – Results of Grain Size Distribution Analyses

Explanation of Terms Used in Report

Record of Borehole Sheets – TWL1, TWL3 (current investigation) and 1 to 5 (previous)

Drawings TWL-1 and 7038901-A – Borehole Locations and Soil Strata

Appendix A – Site Photographs

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Highway 11 and Highway 17 Interchange Underpass Widening
Future Highway 11/17
GWP 5748-04-00, Site 43-350
City of North Bay
District 54, Sudbury, Ontario

1. INTRODUCTION

This report summarises the results of a preliminary foundation investigation carried out for the widening of the existing underpass at the Highway 11 and Highway 17 interchange in North Bay, Ontario (Site 43-350). The investigation was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

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

A previous investigation was conducted at the site by McClymont & Rak Engineers Inc. in May 1990 (Contract 91-216, WP 703-89-01).

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

2. SITE DESCRIPTION AND GEOLOGY

The site is situated at the south junction of Highway 11 and Highway 17 in North Bay. The widening of the existing underpass will carry an additional four lanes (auxiliary and through lanes) of Highway 17 westbound traffic over Highway 11. The alignment of the underpass is considered to be west-east.



Land use in the vicinity of the site comprises commercial properties to the north and undeveloped lands to the south of the underpass. Photographs are provided in Appendix A.

The topography is gently rolling land with rock outcrops. The ground beyond the right-of-way is covered with grass, trees and bushes. The native soils are generally represented by silty sand till.

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

3. INVESTIGATION PROCEDURES

The current field work for this study was carried out on April 30, 2007, comprising 2 test holes (1 test pit and 1 borehole) designated TWL1 and TWL2 advanced to depths of 0.8 and 4.7 m at the locations shown on Drawing TWL-1. Access limitations due to the existing sloping ground limited the drill rig and excavator access to advance the explorations closer to the abutments than where they were drilled. For uniformity with MTO format, the test pit was reported on a record of borehole sheet with the borehole type indicated as an excavated test pit. The foundation investigation was limited to two boreholes as instructed by an e-mail from MTO dated May 2, 2007.

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



The information from the previous investigation conducted at the site by McClymont & Rak Engineers Inc. in May 1990 (Contract 91-216, WP 703-89-01) has been used in preparation of this report; the locations of boreholes 1 to 5 are shown on Drawing 7038901-A. The previous borehole locations were referred to a different co-ordinate system. Their co-ordinates have been converted by Stantec to the Nad83 co-ordinates and approximate locations indicated on Drawing TWL-1. It should also be noted that significant changes to the previous grades have occurred during the construction of the existing underpass.

The locations of and ground surface elevations at the boreholes for the current study were provided by Del Bosco Surveying Ltd. The test holes were advanced either by means of a track-mounted CAT 312 excavator or using continuous flight hollow stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

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

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

Soils were identified in the field in accordance with the MTO Soil Classification procedures. All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. In addition, two grain size distribution analyses were carried out on selected soil samples, their results being presented in Figure GS-TWL-1 and on the corresponding Record of Borehole sheets.



4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classification, inferred stratigraphy, boundary elevations, standard penetration test data, groundwater observations, the results of grain size distribution analyses and moisture content determination. The relevant Records of Boreholes 1 to 5 from the previous investigation were also included for reference.

The borehole locations and stratigraphic cross-sections prepared from the borehole data are shown on Drawings TWL-1 and 7038901-A. It is noted that the current Highway 11 pavement is at approximate elevation 207.0, the west abutment at elevation 216.5 and the east abutment at elevation 215.0, indicating that grade changes have occurred since 1990. Further comments are provided in the following sections.

The subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial fill overlying silty sand till. Cobbles and boulders were identified in the glacial till. Bedrock was contacted at elevation 200.8 to 209.4. The strata encountered are summarized below.

4.1 Fill

Surficial fill was present in the current test holes. With boulders in borehole TWL1, the fill was composed of sand and had a thickness of 0.8 and 1.4 m in boreholes TWL1 and TWL3 respectively. The fill was compact in relative density (SPT-N value of 19), containing boulders in borehole TWL1, and extended to elevation 205.9 and 205.7.

The fill shown in boreholes 1 to 5 was typically represented by compact sand and gravel, with cobbles/boulders and a moisture content of 7 to 14%. The fill was previously penetrated at elevation 209.6 at the west abutment, 210.2 at the pier and 209.2 to 209.4 at the east abutment. It is worth noting, however, that the topography has likely undergone considerable changes in elevation since the previous field investigation in 1990. Currently, the fill likely extends to elevation 205.7 at the west abutment and near elevation 209 at the east abutment. At the pier, the grade was lowered to approximate elevation 207 and the previous fill unit was removed



locally. Currently, the ground behind the proposed abutments has been partially filled for the embankment platform of the existing Highway 17.

4.2 Silty Sand Till

Underlying the fill in borehole TWL3 at 1.4 m depth (elevation 205.7) in borehole TWL3 was silty sand till. This stratum was very dense and about 12% in moisture content. The silty sand till was 3.3 m thick and penetrated at a depth of 4.7 m (elevation 202.4). The results of grain size distribution analyses conducted on two samples of the stratum are presented in Figure GS-TWL-1.

The glacial till revealed in the previously drilled boreholes was compact to very dense and had a moisture content of 10 to 12%. The glacial till extended to bedrock at the west abutment (elevation 200.8 to 201.3) and the pier (elevation 204.1).

4.3 Bedrock

Bedrock was contacted at elevation 200.8 to 209.4 in boreholes 1 to 5 and inferred by refusal at depths of 0.8 and 4.7 m (elevation 205.9 and 202.4) in test holes TWL1 and TWL3 respectively. Based on the data from boreholes 1 to 5, the bedrock comprises a predominantly strong to very strong grey gneiss (refer to the previous borehole logs for more detailed descriptions).

The measured BXT core recovery was invariably 100%. The RQD determined from the rock cores was in a range of 91 to 100% at the centre pier and east abutment, thus indicating an excellent quality rock. At the west abutment, the rock exhibited poor quality becoming good to excellent quality (RQD of 30% increasing to 76 to 100%).



4.4 Groundwater

In the course of the field work for the current study, groundwater was at a depth of 1.4 m (elevation 205.7) in borehole TWL3. During the previous investigation, groundwater was reported to be at elevation 209.2 to 210.0 in boreholes 1 to 3; it is noted that the existing ground surface at the proposed pier location (borehole 3) does not exceed elevation 207. No water was observed in the remaining boreholes. The groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.

5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. M. Rapsey, Senior Technician, and direction of Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer, 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. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, conducted an independent review of the report.

**PART B
PRELIMINARY FOUNDATION DESIGN REPORT**

for
Highway 11 and Highway 17 Interchange Underpass Widening
Future Highway 11/17
GWP 5748-04-00, Site 43-350
City of North Bay
District 54, Sudbury, Ontario

6. ENGINEERING RECOMMENDATIONS

6.1 General

This report provides preliminary foundation engineering comments and recommendations regarding design and construction of foundations, abutments and approach fill embankments for the proposed underpass widening at the Highway 11 / Highway 17 interchange in North Bay, Ontario.

The underpass widening is envisaged to be a two-span structure with span lengths of 60.0 m each and width of about 18.0 m (ref. Preliminary General Arrangement drawing prepared by Stantec in July 2007).

The road grade on Highway 17 at the underpass widening will be near elevation 216.5 at the west abutment and 215.0 at the east abutment matching the existing grades. The approach embankments to the structure are envisaged to be up to 7 m high (interpreted from the road grade and ground surface elevations shown on the Stantec drawing referred to above). The grade on Highway 11 is assessed to be near elevation 207.0.

In summary, the subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial fill overlying compact to very dense silty sand till containing cobbles and boulders. Bedrock, contacted at elevation 200.8 to 209.4, dips down 8.6 m in the direction of the west abutment. Not observed at the east abutment, groundwater was elsewhere at elevation 205.7 in the course of the field work for the current study and 209.2 to 210.0 during the previous investigation. It is noteworthy that the ground surface elevations have changed since 1990.



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

The presence of very dense silty sand till at shallow depths at the west abutment justifies the use of conventional spread footings founded in the glacial till. Spread footings founded on bedrock is the preferred means of supporting the foundation loads at the pier and east abutment.

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

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

6.2 Foundations

6.2.1 Spread Footings

Placing the structure foundations on spread footings bearing on the glacial till at the west abutment (elevation 208) or on bedrock at the pier (elevation 204) and east abutment (elevation 209) is considered to be feasible at the site. The elevations are approximate and should be reviewed at the detail design stage.

Spread footings bearing on the dense silty sand till or bedrock should be designed using the following resistance at ultimate and serviceability limit states (ULS and SLS):

FOUNDATION ELEMENT	FOUNDATION SOIL TYPE	FACTORED RESISTANCE AT ULS, kPa	RESISTANCE AT SLS, kPa
West abutment	Dense silty sand till	1000	500
Pier	Bedrock	8000	N/A
East abutment	Bedrock	8000	N/A



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. Considering the bedrock to be a non-yielding material, the design of footings on bedrock will not be governed by settlement criteria since the loading required to produce 25 mm deformation would be much larger than the factored resistance at ULS. The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

Mass concrete could be placed to provide a level founding surface for the footings. Alternatively, the rock surface could be “stepped” to follow variations in the bedrock surface elevation thereby creating a level subgrade by a combination of rock excavation and placement of mass concrete.

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 glacial till or bedrock. For preliminary design purposes, unfactored friction factors of 0.5 and 0.7 could be used for footings founded on the glacial till and excavated bedrock, respectively.

6.2.2 Piles

The bedrock surface is within 6 m of the proposed road grade at the east abutment and the soil cover on the bedrock is less than 1.5 m. Consequently, use of steel H-piles to support the foundation loads does not appear to be practical at this site. The integral abutment scheme is not considered to be compatible with the existing spread footing foundations and structural configuration with median retaining walls.

6.2.3 Caissons

Installation of caissons is not recommended due to the high groundwater level, the presence of pervious sandy strata as well as numerous cobbles and boulders at the site.



6.3 Approach Embankments

The condition of the existing approach embankments should be investigated during detail design. It is expected, however, that the materials previously used for the construction of the existing approach embankments will be incorporated in the new approach fills with adequate benching (OPSD-208.010).

The design calls for the embankments to be up to 7 m high. The new portions of the approach embankments are to be constructed with granular materials to minimize the magnitude of post-construction settlements. The subgrade revealed in the boreholes drilled at the abutments consists of compact to very dense silty sand till overlying bedrock.

Any topsoil at the abutment locations and along the alignment of the approach fill within 20 m of the abutments should be stripped prior to placement of the embankment fill.

The embankments should be constructed following conventional MTO procedures and have side slopes inclined no steeper than 2H:1V for granular fill. The maximum settlement of the approach fill and native soils brought about by the embankment loads is anticipated to be in the order of 40 mm. These settlements are expected to be completed shortly after placement of the fill.

6.4 Construction Considerations

6.4.1 Excavations

Excavation for construction of the structure foundations supported on spread footings will extend through the existing embankment fill and compact to very dense silty sand till at the west abutment and to the bedrock at the pier and east abutment. Excavation of these soils is expected to be relatively straightforward. Road protection will be required at the west abutment to excavate into the existing approach embankment fill at the existing median retaining wall.



The in situ materials are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Therefore, temporary cut slopes over the full depth of excavation should be inclined at an angle of 45° to the horizontal. The need to excavate flatter sideslopes if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should not be overlooked.

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

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

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

6.4.2 Groundwater Considerations

In the course of the field work for the current study, groundwater was at elevation 205.7 at the west abutment and not observed at the east abutment. It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavations. Groundwater levels are subject to seasonal fluctuations and precipitation patterns.

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



6.5 Lateral Earth Pressures

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

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

where K = coefficient of lateral earth pressure (dimensionless)
 γ = unit weight of free-draining granular material, kN/m^3
 h = depth below final grade, m
 q = surcharge load, kPa, if present
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
 where \emptyset = angle of internal friction of retained soil (35° for Granular A or Granular B Type II or Type III)
 δ = angle of friction between the soil and wall (23.5° for Granular A or Granular B Type II or Type III)

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

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

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

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 towards 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 minimise the build-up of hydrostatic pressure behind the wall. The subdrain tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.

7. ADDITIONAL STUDIES

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

Considering the existing data comprising boreholes with rock cores at each foundation element, the following items should be considered for detailed design studies:

- At the pier and east abutment, advance 2 boreholes 2.5 m west and east and 5 m north and south of the centreline to 3.0 m beyond refusal. The existing data at the west abutment is considered to be adequate for detail design.
- Advance test pits or boreholes through the existing fill north of the existing structure to investigate the condition of the existing fill and required road protection scheme.
- Advance 1 borehole 20 m west and east of the corresponding abutments for approach embankment recommendations.



8. DISCUSSION OF FOUNDATION ALTERNATIVES

In view of the site conditions and proposed structure widening configuration using median retaining walls, it is considered that spread footings are the only type of foundation to be considered for this site. Consequently, the foundation alternatives exclude the use of driven piles for conventional or integral abutments.

The foundation scheme to be selected also depends on other considerations which are being evaluated separately by Stantec.

9. CLOSURE

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

Yours very truly,

Peto MacCallum Ltd.

Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer

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

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

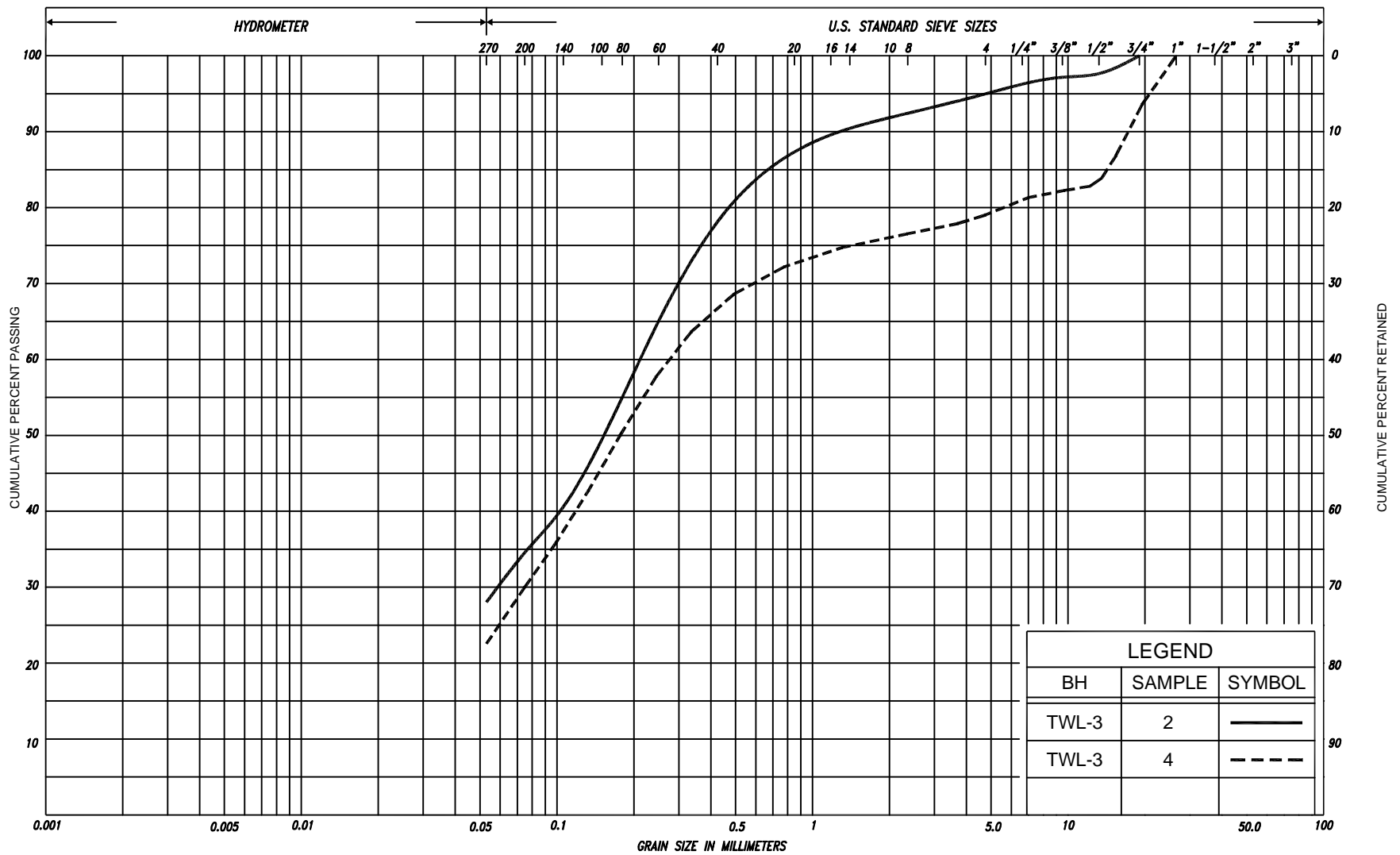
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TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
OPSS 120	General Specification for the Use of Explosives	November 2003
SP 405F03	Construction Specification for Pipe Subdrains	November 2006
OPSD-208.010	Benching of Earth Slopes	November 2003
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 SAND			MEDIUM SAND		COARSE SAND	GRAVEL		COBBLES	UNIFIED
CLAY	FINE	MEDIUM SILT	COARSE	FINE	MEDIUM SAND	COARSE				GRAVEL		COBBLES	M.I.T.
CLAY		SILT		V. FINE	FINE	MED.	COARSE			GRAVEL			U.S. BUREAU
				SAND									

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No TWL1												1 of 1		METRIC			
G.W.P. 5748-04-00		LOCATION Co-ords. 5 129 559 N; 309 042 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
206.7	Ground Surface																
0.0	Sand and boulders trace silt	X															
	Brown (FILL)	X															
205.9	End of borehole																
0.8	Refusal on probable bedrock																
	* Borehole dry																

RECORD OF BOREHOLE No TWL3

1 of 1

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 129 579 N; 309 960 E. ORIGINATED BY M.R.
DIST Sudbury HWY 11/17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY T.X.
DATUM Geodetic DATE April 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										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE		
207.1	Ground Surface						20	40	60	80	100	20	40	60	GR	SA	SI	CL		
0.0	Sand with gravel, some silt Brown Moist to wet (FILL)		1	SS	19															
205.7	Silty sand, trace gravel																			
1.4	Very dense Grey Wet		2	SS	80							○			5	60	(35)			

	with gravel		3	SS	50/15cm															
	(TILL)		4	SS	77/22cm							○			21	49	(30)			
202.4	End of borehole																			
4.7	Refusal on probable bedrock																			
	Samples 3 and 4: sampler bouncing																			
	* 2007 04 19																			
	▽ Water level observed during drilling																			
	▼ Water level measured after drilling																			

RECORD OF BOREHOLE No 1

METRIC

W P 703-89-01 LOCATION Co-ord. 5,129,354 N: 309,936 E. ORIGINATED BY SM
 DIST 13 HWY 11 BOREHOLE TYPE Solid Stem Auger, Tricone, Rock Core COMPILED BY SM
 DATUM Geodetic DATE May 11, 1990 CHECKED BY SB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100				
								SHEAR STRENGTH kPa ○ UNCONFINED * FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L				
								WATER CONTENT (%) 5 10 15				
211.1	Ground surface						211					GR SA SI CL
0.0	Asphalt: 100 mm Fill, sand and gravel, cobbles, boulders, compact		1	AS			210					
209.6			2	SS	22		209					
1.5			3	SS	28		208					5 64 (31)
	Heterogeneous mixture of sand, silt, gravel, cobbles and boulders, compact to very dense (glacial till)		4	SS	43		207					
			5	SS	100/		206					
			6	SS	180/		205					12 52 (36)
							204					
							203					
							202					
201.3							201					
9.8			7	RC BXT	REC 100%		200					RQD 100%
	Gneiss bedrock, coarse crystalline gneissic layering, grey with pink feldspar, strong to very strong, excellent to good quality, below 11.3m some fissure		8	RC BXT	REC 100%		199					RQD 76%
198.3												
12.8	End of Borehole											

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

METRIC

W P 703-89-01 LOCATION Co-ords. 5,129,340 N; 309,932 E. ORIGINATED BY SM
 DIST 13 HWY 11 BOREHOLE TYPE Solid Stem Auger, Tricone, Rock Core COMPILED BY SM
 DATUM Geodetic DATE May 14, 1990 CHECKED BY SR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
210.9	Ground surface															GR SA SI CL
0.0	Asphalt: 90 mm Fill, sand and fine-medium gravel, compact		1	SS	31		210									
209.6																
1.3	Heterogeneous mixture of sand, silt, gravel, cobbles and boulders, dense to very dense (glacial till)		2	SS	49		209									12 58 (30)
			3	SS	76		208									
			4	SS	38		207									
			5	SS	98		206									3 60 (35)
			6	SS	100% 102%		205									
			7	SS	100% 102%		204									
	more boulders below 8.2m						203									
							202									
200.8							201									
10.1	Gneiss bedrock, brown-grey weathered, fractured, poor quality, below 10.8m grey-pink, excellent quality, very strong		8	RC EXT	REC 100%		200									RQD 30%
199.3			9	RC EXT	REC 100%											RQD 92%
11.6	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3

METRIC

W P 703-89-01 LOCATION Co-ords. 5,129,334 N: 309,987 E. ORIGINATED BY SH
DIST 13 HWY 11 BOREHOLE TYPE Solid Stem Auger, Tricone, Rock Core COMPILED BY SH
DATUM Geodetic DATE May 9-10, 1990 CHECKED BY SB

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100		
211.1	Ground surface												
0.0	Asphalt: 100 mm Fill, sand and gravel, compact					211							
210.2			1	SS	100%	210							
0.9	Rock fragments, boulders, silty sand, dense to very dense (Possible fill)		2	RC	100%	209							
			3	SS	100%	208							
206.9						207							
4.2	Heterogeneous mixture of sand, silt, gravel and boulders, very dense (glacial till)		4	SS	100%	206							
						205							
204.1						204							
7.0	Gneiss bedrock, coarse crystalline, gneissic layering, grey-pink, biotite and feldspar phenocryst, strong to very strong, excellent quality		1	RC BXT	REC 100%	203							RQD 92%
			2	RC BXT	REC 100%								RQD 100%
201.1													
10.0	End of Borehole												

RECORD OF BOREHOLE No 4										METRIC							
W P 703-89-01		LOCATION Co-ords. 5,129,333 N: 310,050 E.		ORIGINATED BY SM													
DIST 13 HWY 11		BOREHOLE TYPE Solid Stem Auger, Rock Core		COMPILED BY SM													
DATUM Geodetic		DATE May 17, 1990		CHECKED BY SB													
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80						100
210.6	Ground surface																
0.0	Fill, silty sand, organic stained pockets, compact	X	1	SS	20	Dry	210										
209.2																	
1.4																	
	Biotite gneiss bedrock, grey, below 3.30m pink feldspar phenocryst, strong to very strong, excellent quality	Z	2	RC BKT	REC 100%		209										
206.3	End of Borehole		3	RC BKT	REC 100%		208										
4.3																	

OFFICE REPORT ON SOIL EXPLORATION

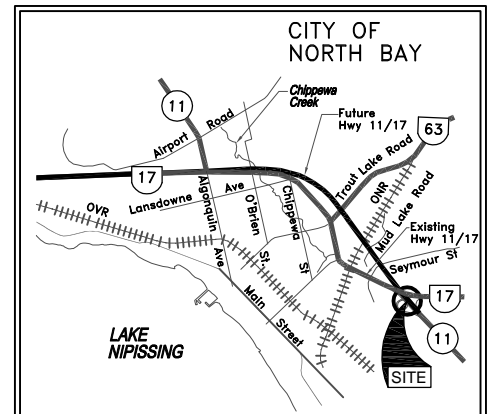
RECORD OF BOREHOLE No 5

METRIC

W P 703-89-01 LOCATION Co-ords. 5,129,314 N: 310,044 E. ORIGINATED BY SM
DIST 13 HWY 11 BOREHOLE TYPE Solid Stem Auger, Rock Core COMPILED BY SM
DATUM Geodetic DATE May 15, 1990 CHECKED BY SB

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
211.0	Ground surface																
0.0	Asphalt: 80 mm					Dry	211										
210.4	Fill, sand and gravel																
0.6	Rock fragments, boulders, some sand (probable fill)		1	SS	100% UCM		210										
209.4																	
1.6	Gneiss bedrock grey, some pink feldspar phenocryst, gneissic layering, strong to very strong, excellent quality		2	RC BXT	REC 100%		209										RQD 95%
							208										
			3	RC BXT	REC 100%		207										RQD 91%
206.3																	
4.7	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION



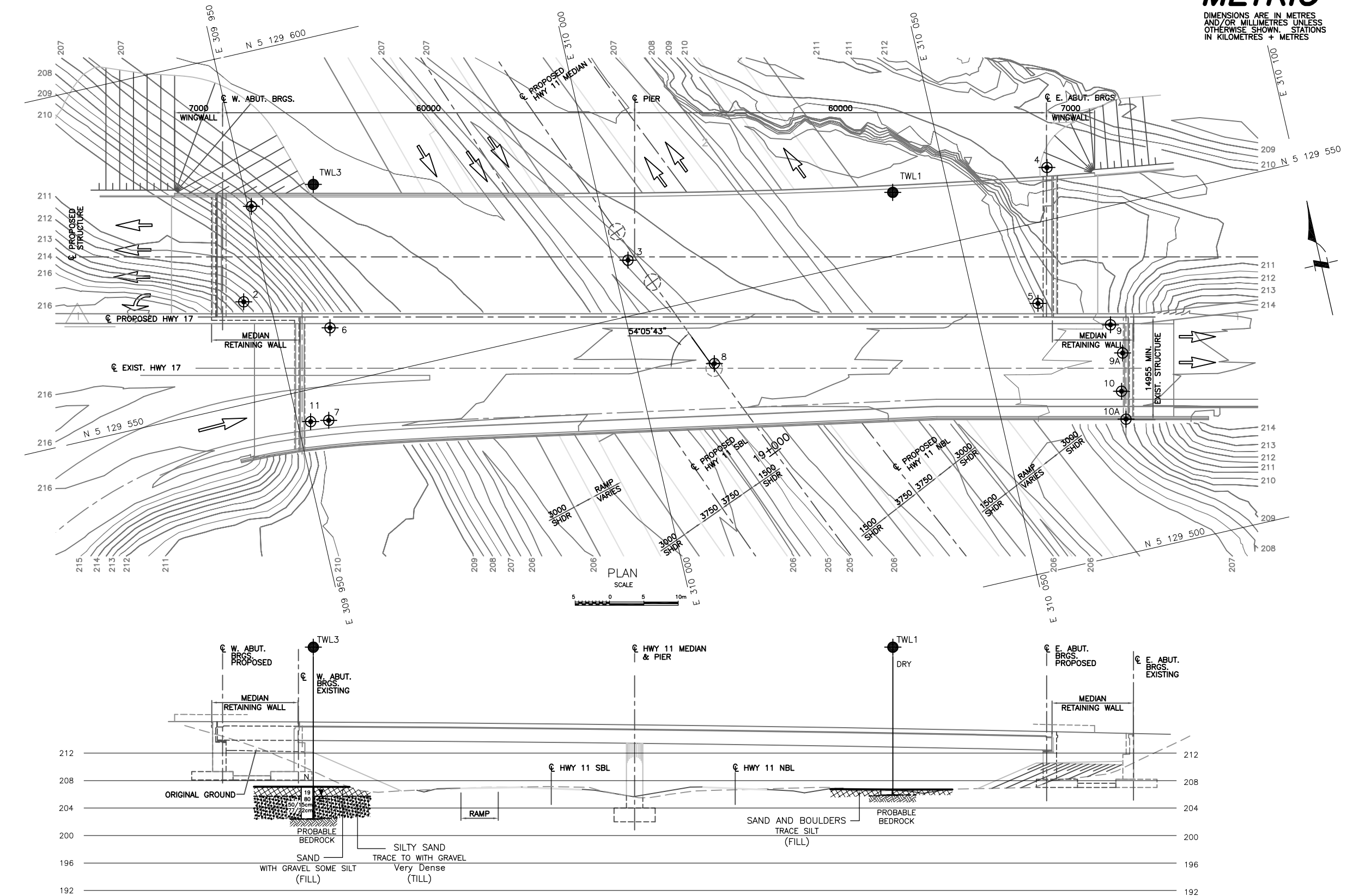
LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
	Borehole from previous investigation		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60 Cone, 475 J / blow)		
	W L at time of investigation April 2007		
	Head		
	ARTESIAN WATER Encountered		
	PIEZOMETER		

BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
TWL-1	206.7	5 129 559	310 042
TWL-3	207.1	5 129 579	309 960
BOREHOLES FROM PREVIOUS INVESTIGATION			
1	211.1	5 129 578	309 951
2	210.9	5 129 565	309 946
3	211.1	5 129 558	310 002
4	210.6	5 129 558	310 065
5	211.0	5 129 539	310 059
6	211.0	5 129 558	309 958
7	210.7	5 129 545	309 955
8	210.9	5 129 541	310 011
9	211.0	5 129 533	310 069
9A	210.8	5 129 529	310 069
10	210.7	5 129 523	310 068
10A	210.7	5 129 519	310 068
11	210.5	5 129 545	309 952

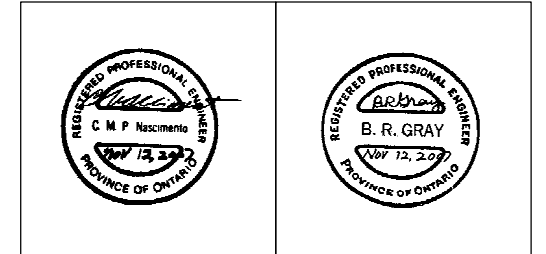
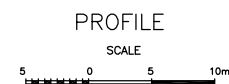
— NOTE —
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

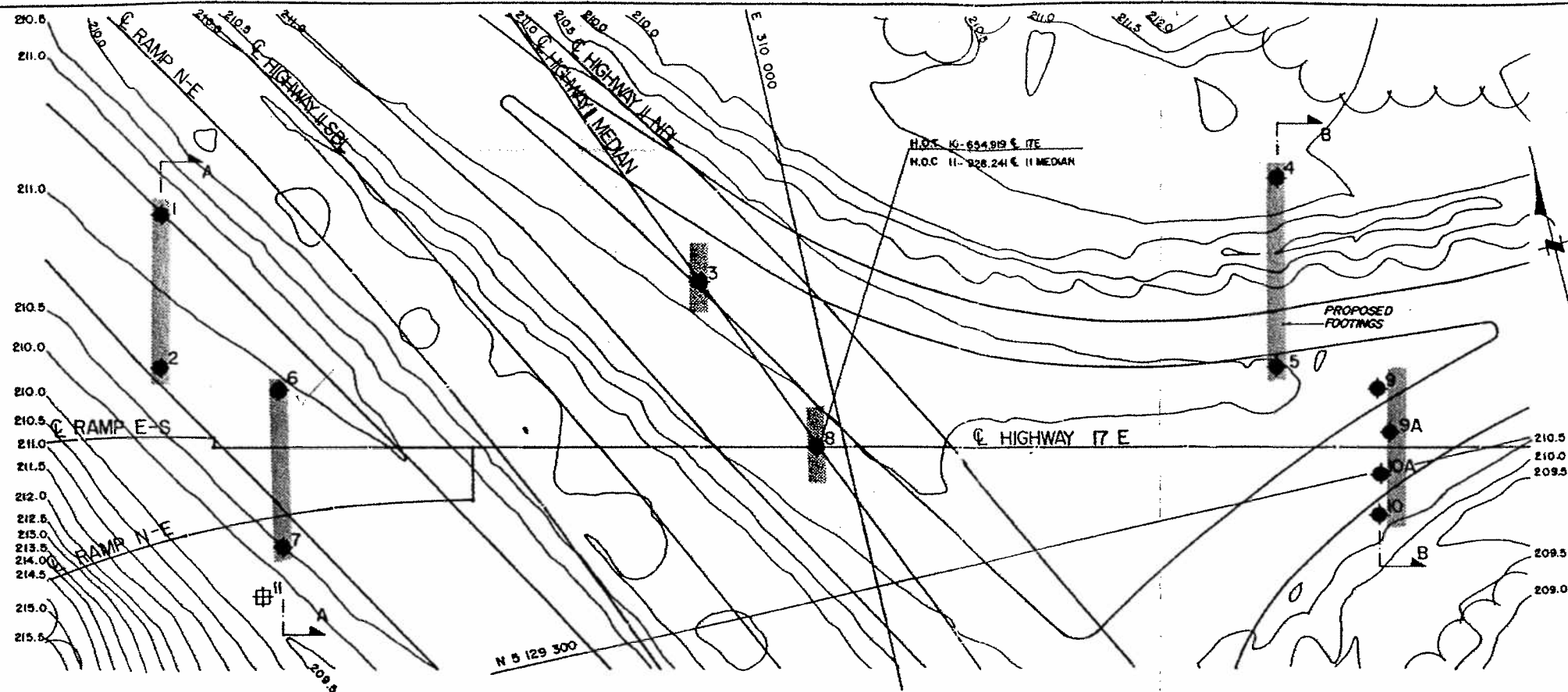
Geocres No. 31L-118			
HWY No	11/17	DIST	SUDBURY
SUBM'D	FP	CHECKED	GD
DRAWN	NA	CHECKED	CN
DATE	NOV. 12, 2007	APPROVED	BRG
SITE	--	DWG	TWL-1



- NOTES:
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - COORDINATES AT BOREHOLE LOCATIONS OF THE PRESENT INVESTIGATION WERE PROVIDED BY DEL BOSCO SURVEYING LTD. COORDINATES FOR THE PREVIOUS INVESTIGATION WERE CONVERTED FROM THE PREVIOUS CO-ORDINATES TO Nad83 MTM Zone 10 BY STANTEC CONSULTING LTD. THE ACCURACY OF THE PREVIOUS SURVEY WAS NOT VERIFIED AND THE BOREHOLE LOCATIONS ARE INDICATED FOR ILLUSTRATION PURPOSES ONLY.
 - GROUND SURFACE ELEVATION AT LOCATIONS OF PREVIOUS BOREHOLES MAY HAVE CHANGED FROM THE TIME OF PREVIOUS INVESTIGATION.
 - REFER TO DRAWING No. 7038901-A DATED JULY 9, 1990 FOR PROFILE AND CROSS-SECTIONS OF PREVIOUS INVESTIGATION.



REF No.: STANTEC DRAWING: 65000580_11-17_B9.dwg
Dated July 2007



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

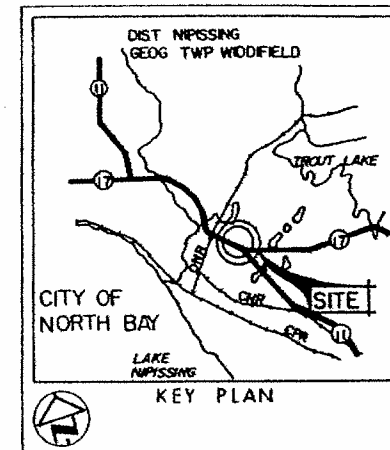
CONT No
WP No 703-89-01



SHEET

CITY OF NORTH BAY
PROPOSED INTERCHANGE AT
HIGHWAY 11 AND HIGHWAY 17
BORE HOLE LOCATIONS & SOIL STRATA

MC CLYMONT & RAK ENGINEERS, INC.



LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◆ Bore Hole & 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 90.05 and 90.06
- ⊕ Test Pit

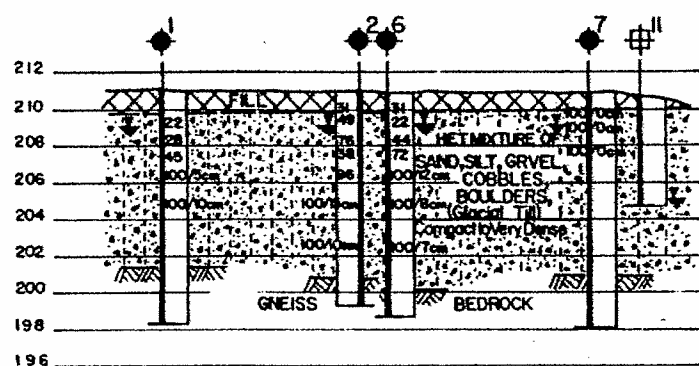
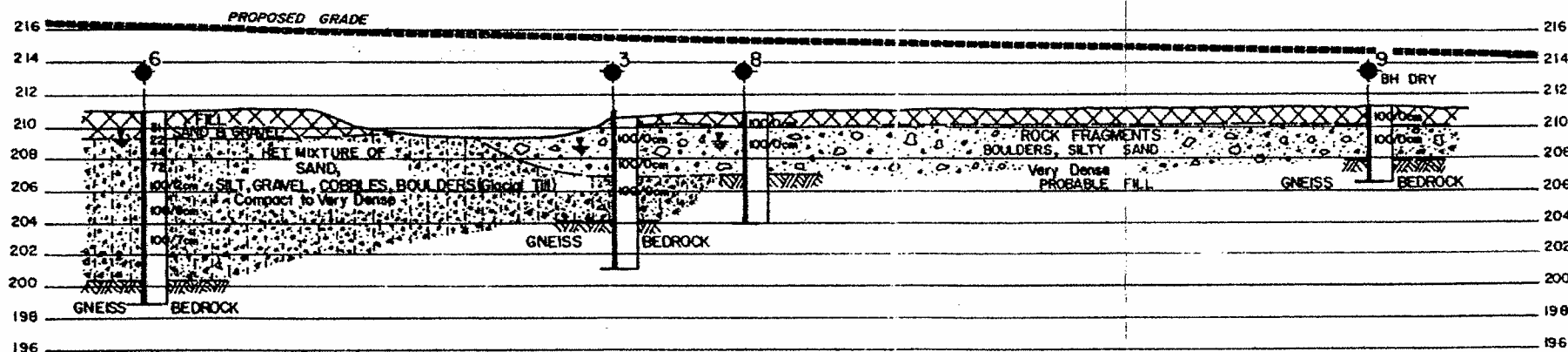
No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	211.1	5 129 354.2	309 936.2
2	210.9	5 129 340.9	309 932.0
3	211.1	5 129 334.3	309 987.9
4	210.6	5 129 333.8	310 050.4
5	211.0	5 129 314.8	310 044.7
6	211.0	5 129 334.4	309 943.4
7	210.7	5 129 321.2	309 940.2
8	210.9	5 129 316.8	309 996.8
9	211.0	5 129 309.4	310 054.3
9A	210.8	5 129 305.0	310 055.1
10	210.7	5 129 295.5	310 053.4
10A	210.7	5 129 299.8	310 053.7
11	210.5	5 129 321.7	309 937.6

NOTE

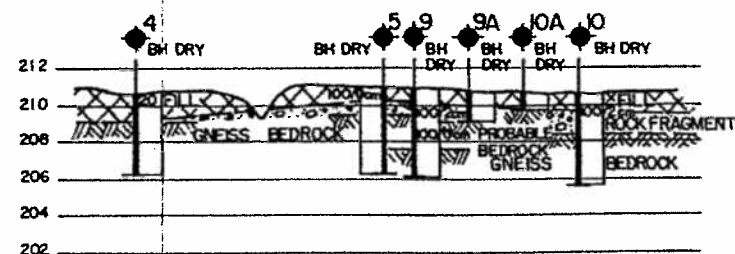
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically included in accordance with the conditions of Section 102-2 of Form 100.

REV	DATE	BY	DESCRIPTION
1			
Geocres No 31L-54			
HWY No 17E		DIST 13	
SUBMD S.B. CHECKED		DATE JULY 9, 1990	
DRAWN E.R. CHECKED		SITE 43-350	
		DWG 7038901-A	



A-A



B-B

SECTIONS

SCALE
5m 0 10m
4m 2 0 4m
HORIZONTAL
VERTICAL



APPENDIX A

Site Photographs



Photograph 1: Looking south at existing Highway 17 and Highway 11 interchange underpass. Note median retaining wall at west abutment. Existing grade was lowered at the underpass (note rock cut in distance on Highway 11 median). (June 19, 2007)



Photograph 2: Zoom view of east abutment showing extensive rock outcrop exposures. (June 19, 2007)