PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

CHAPAIS STREET / CHIPPEWA STREET UNDERPASS
FUTURE HIGHWAY 11/17
CITY OF NORTH BAY
GWP 5748-04-00
DISTRICT 54, SUDBURY, ONTARIO

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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)
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PML Ref.: 05TF058F
Index No.: 076FIDR
Geocres No.: 31L-111
November 2, 2007

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Chapais Street / Chippewa Street Underpass
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 Chapais Street / Chippewa Street Underpass 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 underpass will carry Chapais Street over the Future Highway 11/17. Stantec prepared the preliminary General Arrangement (GA) drawing for the structure that was dated July 2007.

According to the preliminary GA drawing, the Future Highway 11/17 will be a four-lane highway running north-south at the Chapais Street / Chippewa Street Underpass.

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

2. SITE DESCRIPTION AND GEOLOGY

The contemplated structure is located about 200 m north of the existing Highway 11/17. The site is about 2,400 m north from the existing Highway 11 and Highway 17 South Junction.

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Land use in the vicinity of the site is industrial to the west and residential to the east. Topographically, the structure site is located on a relatively flat rock knob surrounded by low-lying terrain on the south and east sides. The ground cover beyond the rock outcrop comprises grasses, scattered stands of trees and bush. Photographs of the crossing 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 bedrock is at the ground surface at this site.

3. INVESTIGATION PROCEDURES

The field subsurface investigation was carried out on April 30, 2007. Three locations were investigated at the site and were identified as test holes with the prefix CS. The holes were advanced to depths of 0.0 to 1.1 m at the locations shown on Drawing CS-1.

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

Two of the test holes encountered bedrock at surface. The third test hole was advanced using continuous flight hollow stem augers powered by track mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a Field Supervisor from the PML engineering staff. The borehole was backfilled in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures.

The groundwater conditions at the test hole locations were assessed by visual examination of the soil and, where appropriate, by measurement of the water level in the open test 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. Only one soil sample was recovered and returned to our laboratory for detailed visual examination and soil classification. Lab testing comprised the determination of the water content of the recovered soil sample.

4. SUMMARIZED SUBSURFACE CONDITIONS

4.1 General

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

The test hole locations and the preliminary layout of the Future Highway 11/17 and Chapais Street/Chippewa Street Underpass structure as well as the longitudinal soil profile are presented on the Foundation Drawing CS-1.

The soil stratigraphy comprises shallow surficial fill extending to probable bedrock in the borehole. The extensive bedrock outcrop is present in the other two test holes.

4.2 Fill

A 1.1 m thick fill layer is encountered in borehole CS3 (west abutment) extending to probable bedrock (elevation 220.9). The layer is composed of sand, with silt, with gravel typical of OPSS Granular B Type II material and found to be in dense condition. It is inferred that the fill was placed to provide a flat surface for the existing Hydro One outdoor storage yard. The standard penetration N value for this unit is 32 for 150 mm sampler penetration, with the sampler bouncing after 150 mm penetration on inferred bedrock. The water content of the fill was 12%.

4.3 Bedrock

Bedrock is present at the surface in the test holes CS1 and CS2 and was inferred by refusal to further advance in the borehole CS3. The depth to the bedrock/inferred bedrock surface varies from 0.0 to 1.1 m, elevations 220.9 to 221.9.

4.4 Groundwater

No water was observed in the borehole during field investigation. The groundwater levels are subjected to fluctuations due to seasonal and rainfall patterns.

Seasonal perched pond water is anticipated at the site in topographic low areas controlled by the underlying bedrock.

5. MISCELLANEOUS

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

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
Chapais Street / Chippewa Street Underpass
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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 Chapais Street / Chippewa Street Underpass 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.

According to the preliminary GA drawing, the proposed bridge will be a two span structure with a total length of 52 m between abutments. The Highway 11/17 platform is planned within an about 4 m deep rock cut to about elevation 218.0 through the Chapais Street/Chippewa Street alignment and the underpass approach embankments will be about 4 m high.

In summary, the soil stratigraphy revealed in the test holes generally indicates that east abutment and pier are located on bedrock outcrops at elevations 221.8 and 221.9, respectively. The west abutment location is underlain by surficial fill over inferred bedrock at 1.1 m depth (elevation 220.9). It is noted that the bedrock was inferred by refusal to augering in one borehole. Bedrock outcrops dominate the entire terrain at the structure location.

Construction of the underpass structure foundations is expected to be entirely in bedrock cut. Standard rock excavation and construction procedures should be suitable. It is imperative that the rock blasting be controlled to prevent disturbance to the existing industrial buildings and utilities and to the condition of bedrock founding level. A precondition survey of existing structures should be conducted prior to construction. Restrictions may be required to limit the effects of blasting (noise and vibration). Also, there may be requirements to limit access to the structure site and hours of work when blasting can be carried for public safety considerations.

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 is feasible. Footings may be used for conventional or semi-integral abutment design.

Pile foundations for integral abutments founded on bedrock may not be practical at this site since a 4 m deep bedrock cut is contemplated. An additional rock trench excavation about 5 m deep would be required to accommodate minimum pile length of 5 m below the abutment stem.

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

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

FOUNDATION ELEMENT	PML BH NO.	FOUNDING DEPTH (m)	FOUNDING ELEV.
East Abutment	CS1	0.0	221.8
Pier	CS2	4.9 *	217.0
West Abutment	CS3	1.1	220.9

* Approximate depth of cut for Future Highway 11/17 platform. This level did not allow for the footing thickness, if required.

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

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

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

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

6.2.3 Piles

For the preliminary design of piles for integral abutments, the steel H-piles will need to be driven to refusal in trenches cut into the bedrock to obtain the minimum 5 m free pile length below the abutment stem. The anticipated depths and elevations of the bedrock surfaces are the same as indicate for spread footings.

The design of integral abutments should be evaluated from the economic viewpoint since the pile tips will have to be established in 5.5 m deep trenches blasted into the bedrock to about elevation 217.5 for west and east abutments, subject to structural design. The pier should be founded on a spread footing placed on the bedrock at about elevation 217.0 as indicated previously.

Based on bedrock high strength assumed at the base of the excavated 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 rock excavation width should be at least 1 m wider than the plan area of the piles, sideslopes could be excavated near vertical. The excavation should be backfilled with Granular “B” Type II 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 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 centre pier footing founded on bedrock will not require frost protection.

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 Chapais Street / Chippewa Street Underpass. We anticipate, however that construction of the approach embankments is likely be founded on bedrock, in view of the bedrock outcrop and shallow bedrock conditions encountered in the west and east 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, 3101.200 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.

The earth fill slopes, if employed, should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation.

The backfill placed adjacent to the abutments will be about 4 m high and it is considered feasible to backfill the structure using granular materials or rock 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), should be in the order of 10 to 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.

The approach embankments remote from the abutments may be constructed using earth or rockfill placed on the encountered bedrock or shallow fill. Settlements of these materials also placed as recommended above are considered to be within acceptable limits. Some low-lying wet ground was noted east of the bridge site beyond the rock outcrop may require special swamp treatment to be determined during detail design.

6.4 Excavation Considerations and Groundwater Control

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 cohesionless sand fill encountered in the borehole is considered a Type 3 soil. The rock is considered Type 1 soil, according to OHSA.

Excavation of the rock should follow the conventional methods of rock excavation 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. This will be primarily dependent on detailed design and should be investigated further during detailed design investigation.

No water was observed during course of the field work. It is considered that seepage from soil and rock fissures or surface that enters the excavation should be readily handled by conventional sump pumping techniques. Groundwater conditions should be further assessed during detail design by drilling boreholes to the full depth contemplated for the proposed foundation construction.

6.5 Lateral Earth Pressures

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

$p = K(\gamma h + q) + Cp + Cs$

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

Free-draining granular material or rock backfill are recommended for backfill material behind the wall. The following parameters are recommended for design:

PARAMETERS	GRANULAR A OR GRANULAR B TYPE II OR TYPE III	ROCK BACKFILL
Internal Friction Angle, Ø (degrees)	35	42
Unit weight, γ (kN/m^3)	22.8	18.0
Coefficient of Active Earth Pressure, K_a	0.27	0.20
Coefficient of Earth Pressure At Rest, K_o	0.43	0.33
Coefficient of Passive Earth Pressure, K_p	3.69	5.04

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

- Boreholes should be carried out at the centre (1), 2.5 m in front (2) and 2.5 m behind (2) the centre pier, the west and east abutments centreline and six 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 centre of each foundation element) should be cored 3.0 m into the bedrock.
- One borehole should be carried out for each of the approach embankments.
- The impact of rock excavation method on the existing buildings, utilities and foundation level bedrock should be assessed.

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 Chapais Street / Chippewa Street Underpass at Highway 11/17.

ADVANTAGES AND DISADVANTAGES

SPREAD FOOTINGS ON BEDROCK		DRIVEN PILES	
ADVANTAGES	DISADVANTAGES	ADVANTAGES	DISADVANTAGES
• Less costly than deep foundation alternative • Conventional design and construction of foundations • Allows semi-integral abutment design	• Long-term maintenance costs of expansion joints for conventional abutment and deck design.	• Allows integral abutment design and construction • Lower long-term maintenance costs of expansion joints for with integral abutment design	• More costly than shallow foundation alternative • Require significant bedrock excavation to achieve minimum required pile length • Heavy equipment for pile driving is required.

8.2 Preferred Foundation Option Considerations

From the foundation perspective, both spread footings and driven pile foundations for integral abutments 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 costs.

Consequently, the most economical alternative in the long term is the semi-integral abutment alternative on spread footings foundations. This foundation scheme is considered to be the preferred foundation system from the foundation engineering 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 was independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.

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

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 120	General Specification for the Use of Explosives	November 2003
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-3101.200	Walls Abutment, Backfill Rock	November 2005
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail	November 2005

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
r_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
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO

n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

Peto MacCallum Ltd.

Foundation Design

RECORD OF BOREHOLE No CS1

METRIC

G.W.P.	5748-04-00	LOCATION	Co-ords. 5 131 753 N; 308 979 E.	ORIGINATED BY	M.R.
DIST	Sudbury HWY 11/17	BOREHOLE TYPE	Manual Sampling	COMPILED BY	T.X.
DATUM	Geodetic	DATE	April 30, 2007	CHECKED BY	C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	*					
221.8	Ground Surface										
0.0	Bedrock at surface										
	* Borehole dry										

ON_MOT VER3 05TF058-ST.GPJ ON_MOT.GDT 11/6/2007 7:49:49 AM

$+^7$, $\times^5$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No CS2

METRIC

G.W.P. 5748-04-00 LOCATION Co-ords. 5 131 734 N; 308 966 E. ORIGINATED BY M.R.

DIST Sudbury HWY 11/17 BOREHOLE TYPE Manual Sampling COMPILED BY T.X.

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	*					
221.9	Ground Surface										
0.0	Bedrock at surface										
	* Borehole dry										

ON_MOT VER3 05TF058-ST.GPJ ON_MOT.GDT 11/6/2007 7:49:51 AM

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No CS3

METRIC

G.W.P. 5748-04-00	LOCATION Co-ords. 5 131 713 N; 308 952 E.	ORIGINATED BY M.R.
DIST Sudbury HWY 11/17	BOREHOLE TYPE Continous Flight Hollow Stem Augers	COMPILED BY T.X.
DATUM Geodetic	DATE April 30, 2007	CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER (SAMPLES)	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
222.0	Ground Surface										
0.0	Sand with silt, with gravel Dense Brown Moist (FILL)		1	SS	32/15cm		221				
220.9 / 1.1	End of borehole										
	Refusal on probable bedrock										
	Sample 1: sampler bouncing										
	* Borehole dry										

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

METRIC

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

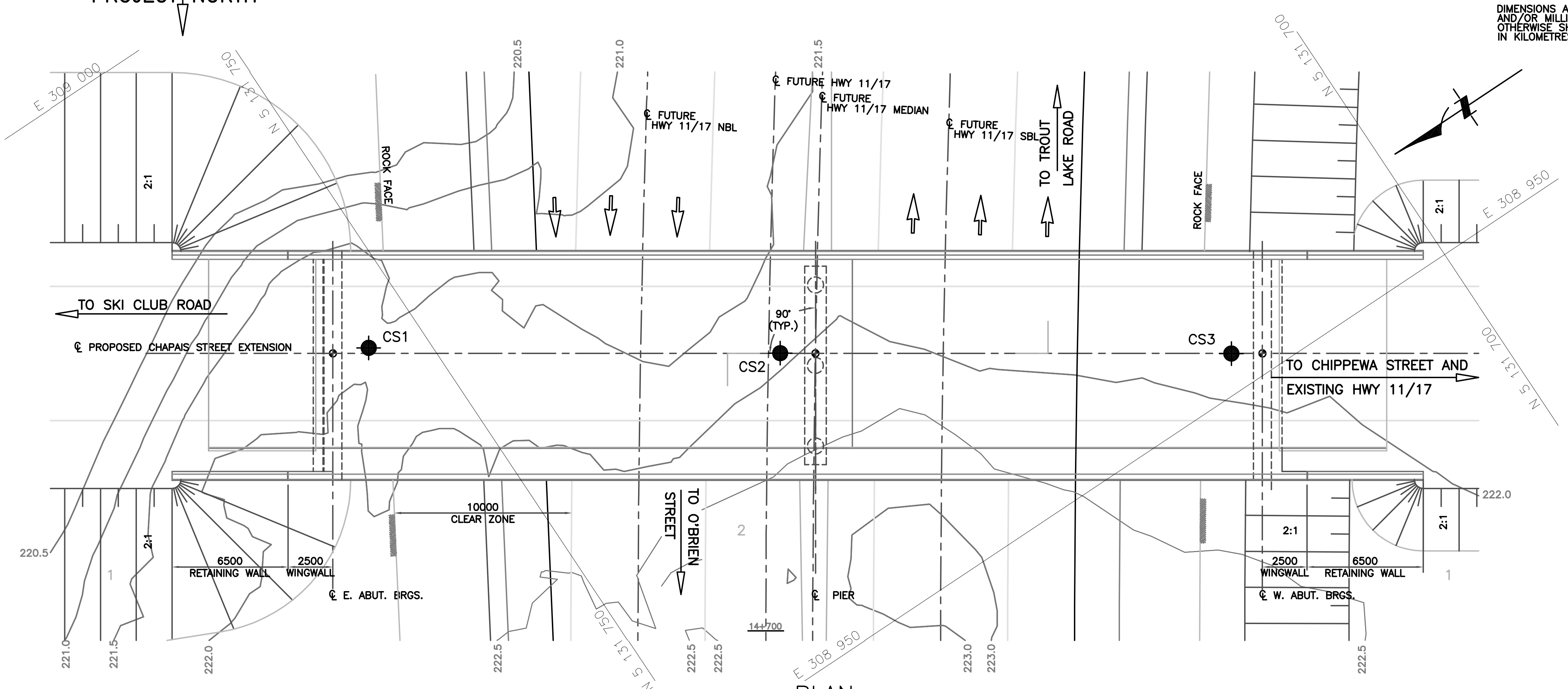
CONT No

GWP No 5748-04-00

CHAPAIS ST. / CHIPPEWA ST. UNDERPASS
FUTURE HIGHWAY 11/17
BOREHOLE LOCATIONS AND SOIL STRATA

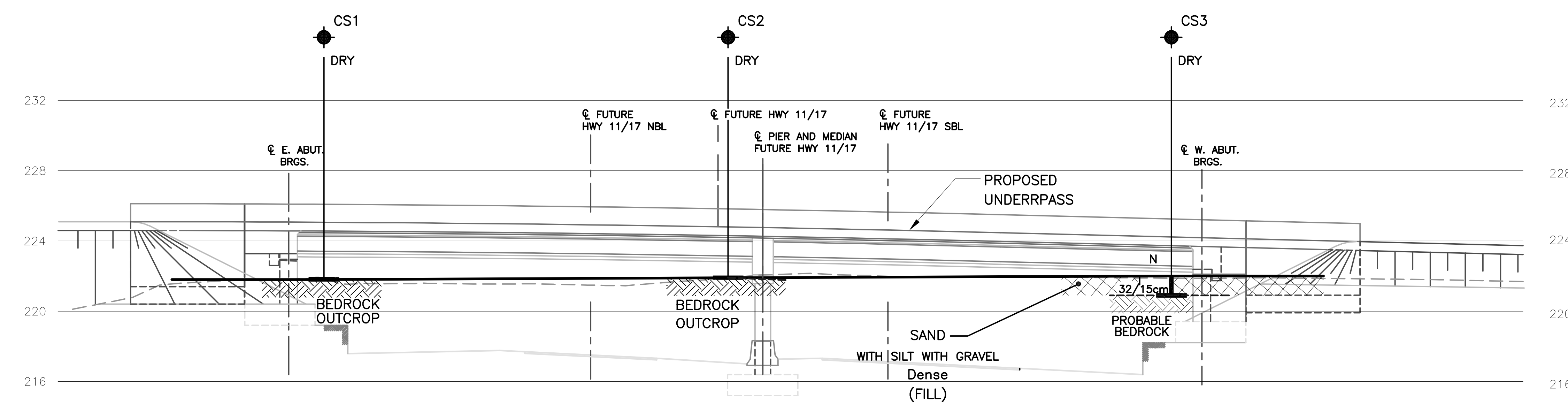
SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



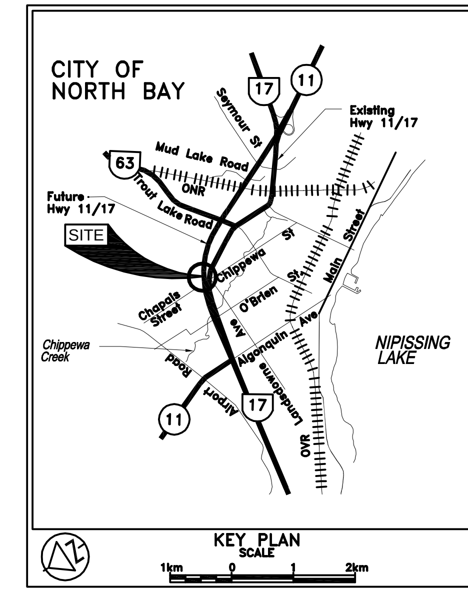
PLAN

SCALE 3 0 3 6m



℄ PROFILE CHAPAIS STREET EXTENSION

SCALE 3 0 3 6m



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
CS1	221.8	5 131 753	308 979
CS2	221.9	5 131 734	308 966
CS3	222.0	5 131 713	308 952

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

REGISTERED PROFESSIONAL ENGINEER C M P Nascimento NOV 02, 2007 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER B. R. GRAY NOV 02, 2007 PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-111

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

NOTES:

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

REF No.:
STANTEC DRAWING: 65000580_CHAPAIS_CHIPPEWA_G1.dwg dated July 2007

APPENDIX A

Site Photographs

Photograph 1: Looking west from east of proposed east abutment. Location of centre pier is at fence line, proposed west abutment is located between the nearest hydro pole and Hydro One building. Note extensive bedrock outcrop. (April 17, 2007)

Photograph 2: Looking north from approximately 25 m south of pier location. Trees and shrubs cover low-lying areas surrounding the bedrock outcrop seen at middle of photograph. (April 17, 2007)

Photograph 3: Looking south easterly from rock outcrop under proposed east abutment location. Note bush and scattered tree cover in low-lying area south of bedrock outcrop. (April 17, 2007)