Foundation Investigation and Design Report
Moon River Rest Areas
GWP 5725-04-00
District 52, Huntsville, ON
Geocres No. 31E-269

Prepared for:
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Project No: SUGE00010479A
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APPENDICES

Part 1 Foundation Investigation

1.1 Introduction

Trow Associates Inc. (Trow) was retained by Northland Engineering Ltd to prepare the Foundation Investigation and Design for WP 5725-04-00. The subject site is located within the geographic Township of Georgian Bay in the District of Muskoka.

The project involves the construction of two restroom facilities within proposed rest areas located approximately 1.56 km and 1.35 km north of the Moon River, on the east and west sides of Highway 69 respectively. The purpose of this geotechnical investigation was to determine the existing site topography, vegetation, drainage, land use and existing structures within the two proposed rest areas. In addition, subsurface soil and groundwater conditions were to be determined within the proposed construction limits for each restroom by field investigation and laboratory testing.

For the purpose of this report all references to Highway Station Chainage and Existing Structure Location and Numbering have been based on site drawings originated for MTO Contract No. 2004-5001.

1.2 Site Description and Geological Setting

1.2.1 General Site Description

The proposed rest areas are located in the Township of Georgian Bay Muskoka on the east and west sides of Highway 69 between Station 10+062.713 to the south and Station 10+578.827 to the north.

The proposed rest area locations are shown on the attached Borehole Location Plan in Appendix A.

The overall terrain is dominated by exposed migmatitic and gneissic rock outcrops and steep rocky slopes with wetlands systems within the bedrock depressions. The uplands areas are treed with sugar maple and hemlock forests growing on shallow organic silt soils or on bare rock.

1.2.2 Geological Setting

Reference to the Ontario Geological Survey (OGS) Maps 2544 and 2556 indicates that the bedrock geology at the site is located in the Mesoproterzoic era comprising migmatitic rocks and gneisses of undetermined protolith and granitic origin. The predominant rock types are layered biotite gneisses and migmatites which locally include quartzofeldspathisc gneisses, orthogneisses and paragneisses. The topography in the area consists of undulating bedrock outcrops separated by intervening marshy zones and wooded areas. The surface soils in the area comprise intervening recent deposits of shallow organic peats and muck combined with a thin, discontinuous layer of drift.

1.2.3 West Side Rest Area

The proposed footprint for the restroom facility on the east side of Highway 69 was centered at approximately Station 10+210 with an approximate offset west of the Center Line (CL) median of 58 m. The proposed restroom facility was located within an area that had been artificially raised and leveled using an imported silty sand and cobble fill material. This built up area was subtended by a pool of standing water along the west side and a water bearing ditch along the south side. The water bearing ditch was fed by culvert # 29 and by run off from within the built up fill.

Existing structures on the west side of Highway 69 within the proposed rest area included a "truck rest area", four box culverts, a steel guard rail located along the edge of gravel and a post and wire fence located along the existing limit of the roadway. It is understood that these structures were constructed as a part of MTO Contract Number 2004-5001.

The "truck rest area" was located between Stations 10+300 to 10+600, it was surfaced with a layer of Hot Mix Asphalt (HMA) and granular base over an undetermined depth of blast rock fill Select Subgrade Material (SSM). Based on the proposed design for the rest area (as shown on the attached Borehole Location Plan in Appendix A), this "truck rest area" will be incorporated into the construction phase as an exit ramp and deceleration lane.

1.2.4 East Side Rest Area

The proposed footprint for the restroom facility on the east side of Highway 69 was located on a rock outcrop, centered at approximately Station 10+435 with an approximate offset east of the Center Line (CL) median of 58 m. The proposed restroom facility location was approximately 3.5 m above the existing edge of pavement at an elevation of ~237 m above MSL. The rock outcrop sloped gently down towards the east and steeply down towards the north and south, with a vertical rock cut along the west face adjacent to the highway.

Existing structures on the east side of Highway 69 within the proposed rest area included two box culverts, a gravel road leading to an active quarry and a section of road subbase roughed in using blast rock fill. The gravel road intersected the proposed rest area at approximately Station 10+390 and the section of roughed in road base was located adjacent to the existing roadway from approximately Station 10+170 to Station 10+380 within the approximate location of the proposed exit ramp and deceleration lane for the rest area.

1.2.5 Drainage

As a result of the undulating nature of the topography at this site, the general trend of surface runoff was to work its way down to a low lying stream located at the north end of the site or to be retained within the wetland areas in localized depressions. The stream passed underneath the roadway through culverts 32 and 34 at CL Median Station 10+569 (as shown on the attached Borehole Location Plan in Appendix A) and flowed towards the west.

Within the proposed rest area on the west side of Highway 69, surface runoff was retained in a pool of standing water along the west side of the site in the approximate location as shown on the attached Borehole Location Plan in Appendix A.

Along the east side of Highway 69 the rock outcrop upon which the proposed restroom facility is to be located was the high point within the proposed rest area. Surface runoff was directed to towards a wetland area to the south of the outcrop and to a low lying stream to the north of the outcrop.

Along the roadway surface, runoff was directed towards a ditch within the Center Line (CL) median and dispersed through existing culverts primarily to the west, although a small portion of the runoff is directed to the wetland area located to the south of the proposed rest area on the east side of Highway 69.

Table 1 below indicates the CL origin of the existing culverts and the direction of flow.

Table 1: Existing Culvert Station, Numbering and Direction of Flow

Culvert Number	Center Line Station	Direction of Flow
27	10+293	West
29	10+182	East
30	10+328	East
31	10+489	East
32	10+569	East
34	10+569	East

1.3 Investigative Procedures

1.3.1 General

The fieldwork for this project was carried out on November 25th and 26th, 2006. The Investigation consisted of two boreholes (BH-1 and BH-2). Borehole BH-1 was located within the proposed footprint for the restroom facility on the west side of Highway 69. Borehole BH-2 was located within the proposed footprint for the restroom facility on the east side of Highway 69.

Both boreholes were advanced to bedrock between 0.6 m to 1.65 m depth. A 1.5 m rock core was retrieved from within each borehole. The boreholes were advanced with a Mobile CME-55 track mounted drill rig owned and operated by Landcore Drilling, a specialist drilling contractor. The drill rig was equipped with standard soil and rock sampling equipment, including continuous flight hollow stem augers, diamond toothed core barrels and steel casing.

From the drilling program, soil samples were obtained using a 51 mm (2 inch) outside diameter split spoon sampler in conjunction with Standard Penetration Tests (ASTM D 1586), at 0.75 m intervals for the upper 3.0 m and at 1.5 m intervals thereafter. The Standard Penetration Test "N" values were recorded and used to provide an assessment of the in-situ relative density of the overburden soils. All boreholes were backfilled with auger cuttings and sealed with Bentonite pellets upon completion of drilling.

All fieldwork was supervised by a member of Trow's engineering staff who directed the drilling and sampling operations, logged the factual borehole data, and retrieved soil samples for subsequent laboratory testing and identification. All borehole elevations were determined in the field by Trow. The geodetic borehole elevations were referenced to the north-west corner of the proposed restroom facilities on both sides of Highway 69 as staked in the field by Sutcliff Rody Quesnel Inc. (SRQ).

The locations of the boreholes and the elevations are shown on the attached Borehole Location Plan in Appendix A.

1.4 Laboratory

The soil samples obtained in the field were carefully transported to our Sudbury laboratory and examined for further verification and classification. A laboratory testing program for selected soil samples consisted of Natural Moisture Content Determination (LS 701) and Particle Size Analyses (LS 702).

The laboratory test results are summarized on the attached borehole logs in Appendix B as well as in Appendix C.

1.5 Subsurface Conditions

1.5.1 General

The subsurface conditions encountered during the field investigation are summarized on the attached borehole logs in Appendix B. The following is a brief description of the subsurface conditions encountered during the field investigation.

1.5.2 Stratigraphy of Proposed Restroom Facility West of Highway 69

The stratigraphy within the footprint of the proposed restroom facility west of Highway 69 as determined from borehole BH-1 comprised a 50 mm thick layer of topsoil over a 1.6 m thick layer of sand with cobble, trace fibrous organic fill overlying bedrock.

The fill material was brown in colour, moist to wet and loose to compact. Uncorrected Standard Penetration Test (SPT) "N" values ranged from 8 to 18 blows per 300 mm. Bedrock was encountered in borehole BH-1 at an elevation of 230.0 m or 1.7 m below grade.

The bedrock comprised biotite gneiss formed during the Mesoproterzoic Era within the Central Gneiss Belt Region of Ontario. The biotite gneiss was fresh to slightly weathered, light grey to pink with frequent dark grey bands, large grained with occasional fractures. The core recovery within the bedrock was excellent at 100% with a Rock Quality Designation (RQD) of 30% indicating poor rock quality. The unconfined compressive strength of the rock at an elevation of ~229.7 m was found to be 110 MPa, making it very strong in standard engineering terms.

1.5.3 Stratigraphy of the Restroom Facility East of Highway 69

The stratigraphy within the footprint of the proposed restroom facility east of Highway 69 as determined from borehole BH-2 comprised a 0.6 m thick layer of organic silt with fine sand, trace gravel overlying bedrock.

The organic silt with fine sand was brown in colour, damp and loose and of very low plasticity. Bedrock was encountered in borehole BH-2 at an elevation of 236.5 m, or 0.6 m below grade.

The bedrock comprised migmatite formed during the Mesoproterzoic Era within the Central Gneiss Belt Region of Ontario. The migmatite contained occasional bands of light coloured quartzofeldspathic gneiss, was fresh, medium to dark gray, medium grained and strong. The core recovery within the bedrock was excellent at 100% with a Rock Quality Designation (RQD) of 97% indicating excellent rock quality. Unconfined compressive strength of the rock at an elevation of ~236.3 m was found to be 73 MPa, making it strong in standard engineering terms.

1.6 Groundwater Conditions

Groundwater was encountered in boreholes BH-1 at an elevation 230.4 m, or 1.3 m below existing grade. Groundwater was not encountered in borehole BH-2.

Seasonal variations in the water table should be anticipated, with higher levels occurring during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods.

Part 2 Engineering Discussion and Recommendations

2.1 Introduction

The following subsections address the geotechnical design and construction considerations for the proposed restroom foundations within the proposed rest areas located on the east and west sides of Highway 69 north of the Moon River within the Township of Georgian Bay Muskoka.

2.2 Foundation Recommendations

Given the relatively shallow depth to bedrock within the proposed restroom locations on both sides of Highway 69, Trow recommends placing the building on shallow strip or spread footing on bedrock.

2.2.1 Strip or Spread Footing Bearing Capacity

For the proposed footings founded on fresh or slightly weathered bedrock, a Factored Bearing Resistance at ULS of 1.5 MPa is recommended in accordance with the C.H.B.D.C. and subject to inspection by a qualified geotechnical engineer. To place the footings directly on bedrock the overburden material along with any loose debris and rock shatter must be removed, exposing sound bedrock. Prior to the placement of the footings, the exposed bedrock is to be visually inspected by a qualified geotechnical engineer to verify the integrity of the rock. Strip or spread footings widths must comply with the Ontario Building Code (OBC) minimum requirements and standard MTO practices.

Footings should preferably be established on a relatively level rock surface (i.e. generally sloping at an angle of less than 10° from the horizontal). However, footings may be placed on bedrock sloping up to 30° from the horizontal, provided that steel dowels are provided to resist the shear. Where rock slopes are greater than 30° from the horizontal, the rock surface must be leveled to provide a step-like footing base.

The above Factored Bearing Resistance at ULS applies to footings placed directly on sound bedrock. The bearing capacity at SLS will not govern for footings founded on bedrock as the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored resistance at ULS.

2.2.2 Strip or Spread Footing Anchors

Design parameters for anchors into the underlying bedrock have been provided in the event that additional resistance is required for overturning, etc.

The structural engineer normally designs the length and diameter of the steel dowels for the footings. From previous bedrock investigations in the area and the current investigations, it was found that the rock mass was considered to be competent with an estimated unit weight of approximately 25.0 kN/m^3 to 30.0 kN/m^3. The unconfined compressive strength of the rock material was found to be in the order of 73 MPa to 110 MPa making it strong to very strong in standard engineering terms. For this type of bedrock, failure will occur between the dowel and the grout, or between the grout and the rock, and not from a quasi-conical rock mass failure, provided sufficient dowel bond lengths have been designed. Empirical methods of analysis, such as pull out tests have shown that the bond developed between the grout and the dowel are typically twice that of the bond developed between the grout and the bedrock. Therefore, the design analysis for this rock mass should be based on failure occurring between the grout and the bedrock interface. For straight-shafted dowels, the anchor force which can be developed is dependent on the ultimate bond stress of the bedrock or the grout material.

The ultimate bond stress is typically taken as 10% of the compressive strength of the bedrock (8 MPa) or the compressive strength of the grout material whichever is less. The allowable bond stress, "τ_b" taken between the rock and the grout is normally 50% or less of the ultimate bond stress, (i.e. Safety Factor of 2.0).

The required bond length (L) for the anchor is a function of the core hole diameter (d), and can be calculated as follows:

$$L\ (m) = P/(\pi \times d \times \tau_b)$$

Where: P = working capacity of anchor (kN)

τ_b = working bond stress (kPa)

d = core hole diameter (m)

The upper 300 mm of the bedrock is not normally considered part of the bond length, since this area is usually weathered/fractured, and as a result does not usually develop the ultimate bond stress assumed in the above calculations. As the bedrock surface at this site will be cleared of all weathered or fractured material, the upper 300 mm of the anchor may be considered, but to be conservative Trow still recommends ignoring this portion of the anchor.

Therefore, since the rock mass at this site is competent, it would be advantageous to use a good quality, high strength, high shear grout to increase the performance of the anchor.

During construction, pull-out tests equal to the design loads should be performed to confirm the strength of the anchors. This work can be performed on a representative number of anchors by this office.

2.2.3 Floor Slab

Conventional floor slab-on-grade construction will be possible at this site, provided that all surface topsoil, organics, and fill material are removed down to bedrock. The floor slab elevation(s) should be established to provide a minimum of 300 mm of Granular "B" Type II (OPSS 1010) followed by 150 mm of Granular A (OPSS 1010), combined with an appropriate moisture barrier such as a polyethylene membrane. Fill material required below the Granular "B" Type II may consist of a Granular "B" Type I (OPSS 1010) material placed in 150 mm thick lifts and compacted to 98% of the Standard Proctor Maximum Dry Density. The Granular "B" Type II material should be placed below the floor slab in lifts not exceeding 200 mm and compacted to 100% Standard Proctor Maximum Dry Density (SPMDD). The 150 mm thick layer of Granular "A" should be placed directly below the floor slab and compacted to 100% Standard Proctor Maximum Dry Density (SPMDD).

A representative of Trow should be on-site during for any fill material placement, to verify the design assumptions, and to verify the design recommendations.

2.2.4 Backfill Material

All backfill material used for the foundation walls should comprise Granular "B" Type II material (OPSS 1010), with a maximum aggregate size not exceeding 100 mm. The Granular "B" material should placed in uniform lifts, not exceeding 200 mm during backfilling operations and be compacted to 100% Standard Proctor Maximum Dry Density (SPMDD).

2.2.5 Settlements

Provided the recommendations as outlined in this report are followed, the total and differential settlements of the structure are anticipated to be negligible.

2.2.6 Drainage

The exterior grade around the buildings should be sloped away from the walls to prevent surface runoff from entering the building. Permanent perimeter weeping tile should be installed where any floor is less than 150 mm above final grade and is required to be dry. Perforated drainage tile must be placed at the base of the footings to drain the foundation wall backfill. The drainage tile should have a minimum diameter of 100 mm, and be surrounded by well draining filter material (i.e. 20 mm clearstone gravel). The filter material should be surrounded with a non-woven geotextile. The perforated drainage tile should drain to an interior sump, or other frost free area. All subsurface walls should be adequately damp-proofed above the water table and waterproofed below the water table. The roof drains should discharge away from the building to appropriate drainage areas.

2.2.7 Earthquake Parameters

The Ontario Building Code specifies that new structures be designed to withstand lateral force at the base of a structure equivalent to a potential seismic event. An equivalent lateral force at the base of a structure, V_e, is calculated using the following:

$$V_e = v \cdot S \cdot I \cdot F \cdot W$$

The terms relevant to the geotechnical conditions are the zonal velocity ratio "v" and the foundation factor "F". The zonal velocity ratio for the Parry Sound area is 0.05. The foundation factor, "F", which should be applied at this site, is 1.0. These parameters should, however, be reviewed in detail by the structural designer.

2.2.8 Unwatering, Drainage and Construction Concerns

Groundwater was encountered within borehole BH-1 on the west side of Highway 69. This groundwater is likely associated with the pool of standing water located adjacent to the area of built up fill (as mentioned previously in section 1.2.5 of this report). The overburden fill material encountered at this borehole location comprised sand with cobble fill and may be considered relatively free draining. Although the inflow of groundwater can likely be controlled with the use of conventional pumps, Trow recommends that the pool of standing water be permanently drained to permit construction in the dry and to avoid future complications.

2.2.9 Lateral Earth Pressure

Foundation walls should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure is given by

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

where p = Lateral earth pressure (kPa).

K = Coefficient of earth pressure.

γ = Unit weight of backfill.

h = Depth to point of interest (m).

q = Surcharge load acting adjacent to the wall at the ground surface (kPa).

The above expression does not take into account hydrostatic pressure.

The Table below lists various earth pressure properties for given materials.

Table 1 - Material Types and Earth Pressure Properties

Material	Friction Angle ø′ (unfactored)	Coefficient of Active Earth Pressure (k_a)	Coefficient of Passive Earth Pressure (k_p)	Coefficient of Earth Pressure at Rest (k_o)	Unit Weight γ (kN/m^3)
Granular A	38°	0.24	4.17	0.38	22
Granular B Type I	30°	0.33	3	0.50	21
Granular B Type II	35°	0.27	3.7	0.43	21
Rock Fill	42°	0.2	5	0.33	20

Note: Values given for horizontal earth pressures are for horizontal backfill. For sloping backfill, the design requirements outlined in Sec C6.9.1(c) of the C.H.B.D.C. should be used. A unit weight of γ*=20 kN/m*^{3} *is based on well graded rockfill.*

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effects of compaction surcharge should be taken into account in the calculations of active and at rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

2.2.10 Frost Protection

For foundations on bedrock frost protection is not required, provided the footings are established on sound bedrock that has adequate drainage provisions and all shatter has been cleaned from the bearing surface.

2.2.11 Excavations

All excavations must be conducted in accordance with the Occupational Health and Safety Act and Regulations for Construction. The underlying non-cohesive soils such as the silty sand and cobble fill may be classified as "Type 3" soil, and the organic soils such as the organic silt with fine sand may be classified as a "Type 4" soil, in conformance with the Ontario Health and Safety Act and Regulations.

In accordance with the Occupational Health and Safety Act Regulations for Construction Projects, excavation procedures of the existing non-cohesive material for Type 3 soils and organic materials for Type 4 soils will be adequate. Excavation side slopes in the rock fill should remain stable at a slope of 1H:1V above the groundwater table. We do not anticipate that groundwater will be encountered within the excavations for foundations.

Bedrock excavation will require drilling and blasting procedures. Based on our experience with bedrock in this area, the bedrock is known to be "brittle" and contains fractures and joints. It is often difficult to blast and excavate to defined geometry using conventional drilling and blasting procedures. Problems with "overbreak" are common. This may affect quantities claimed by the rock excavation contractor as well as the amount of backfill, which may be required. The contractor should make an allowance for potential "overbreak" conditions.

2.2.12 Blasting and Vibration Monitoring/Control

Due consideration should also be given to controlled blasting procedures to prevent damage to adjacent structures. A pre-blast survey is recommended prior to initiating blasting operations. Pre-blast surveys should be completed on all existing structures in close proximity to the subject site, to obtain background, pre-blast condition data in the event of any damage claims. The size and design of the blast should be limited such that peak particle velocities are limited to 50 mm/sec. Blast monitoring devices (seismographs) should be set up at strategic locations during blasting to ensure that vibrations from blasting are within the acceptable limits. Appropriate blasting mats must be provided to avoid any fly rock, which could damage nearby property and seriously harm the public. A recognized, competent specialist contractor experienced in this type of work should carry out any blasting. Prior to conducting the blasting, the blast design should be reviewed by a competent professional engineer familiar with blasting techniques. The blasting contractor must also follow OPSS 120, General Specification for the Use of Explosives.

3.0 Miscellaneous

The field investigation was supervised by Mr. J. Jackson of Trow's Sudbury office and Mr. S. McAuliffe of Landcore Drilling Limited using equipment owned and operated by Landcore Drilling Limited.

The laboratory testing was performed at Trow's Sudbury Laboratory.

This report has been prepared by Mr. J. Jackson, B.Eng., and reviewed by Mr. T. Crilly M.Sc., P.Eng. and Mr. S. Gonsalves, M.Eng., P.Eng. Designated MTO Foundation Contact.

4.0 Closure

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

Trow Associates Inc.

JAN 31/07

TM

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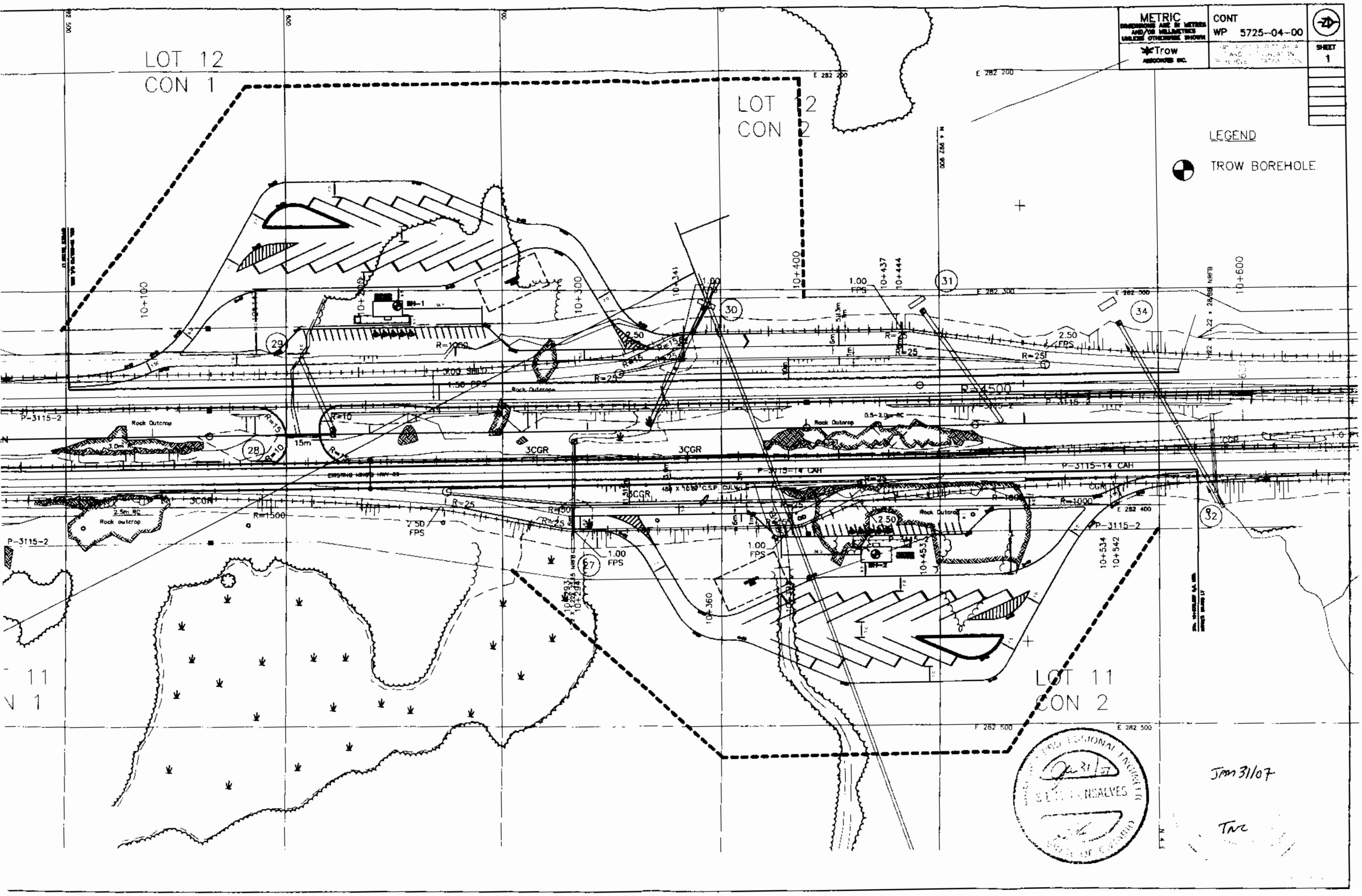
APPENDIX A

Drawings



METRIC
CONT
WP 5725-04-00
SHEET 1
Trow
LEGEND
TROW BOREHOLE
LOT 12
CON 1
LOT 12
CON 2
LOT 11
CON 2
10+100
10+200
10+300
10+341
10+400
10+437
10+444
10+453
10+534
10+542
10+600
10+360
Rock Outcrop
3CGR
P-3115-2
P-3115-14 CAH
R=4500
R=1000
R=1500
BH-1
BH-2
Jan 31/07

APPENDIX B

Borehole Logs

EXPLANATION OF TERMS USED IN REPORT

'N' VALUE: AN INDICATOR OF SUBSOIL QUALITY. IT IS OBTAINED FROM THE STANDARD PENETRATION TEST (CSA STD. A119.1). SPT 'N' VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 2 INCH O.D. SPLIT-BARREL SAMPLER TO PENETRATE 12 INCHES INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WEIGHING 140 POUNDS, FALLING FREELY A DISTANCE OF 30 INCHES. FOR PENETRATIONS OF LESS THAN 12 INCHES 'N' VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. 'N' VALUES CORRECTED FOR OVERBURDEN PRESSURE ARE DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST (CSA STD. A119.3): CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (2" O.D. 60 CONE ANGLE) DRIVEN BY 350 FT-LB IMPACTS ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 12 INCH ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOIL QUALITY: SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSITY.

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

S_u (PSF)	0 - 250	250 - 500	500 - 1000	1000 - 2000	2000 - 4000	> 4000
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

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

'N' (BLOW/FT)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCK QUALITY: 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 DRILLED IN THAT CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE NATURALLY FRACTURED CORE PIECES, 4"+ 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

FIG 22-4: EXPLANATORY SHEET FOR SOIL MECHANICS TERMINOLOGY USED IN FOUNDATION INVESTIGATIONS

ABBREVIATIONS & SYMBOLS

LABORATORY TESTING

TRIAXIAL TESTS ARE DESCRIBED IN TERMS OF WHETHER THEY ARE CONSOLIDATED (C) OR NOT (U) ISOTROPICALLY (I) OR NOT (A) AND SHEARED DRAINED (D) OR UNDRAINED (U) WITH PORE PRESSURE MEASUREMENTS (BAR OVER SYMBOLS)

EG. $\overline{CIU}$ = CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENT

UNLESS OTHERWISE SPECIFIED IN REPORT ALL TESTS ARE IN COMPRESSION

FIELD SAMPLING

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

EARTH PRESSURE TERMS

μ	COEFFICIENT OF FRICTION
δ	ANGLE OF WALL FRICTION
K_o	COEFFICIENT OF EARTH PRESSURE AT REST
K_A	COEFFICIENT OF ACTIVE EARTH PRESSURE
K_p	COEFFICIENT OF PASSIVE EARTH PRESSURE
i	ANGLE OF INCLINATION OF SURCHARGE
ω	SLOPE ANGLE-BACKFACE OF WALL
β	ANGLE OF SLOPE
N_γ, N_q, N_c	BEARING CAPACITY FACTORS
D_f	DEPTH OF FOOTING
B, L	FOOTING DIMENSIONS

INDEX PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G_s	SPECIFIC GRAVITY OF SOLIDS
e	VOIDS RATIO
e_o	INITIAL VOIDS RATIO
e_{max}	e IN LOOSEST STATE
e_{min}	e IN DENSEST STATE
D_r	RELATIVE DENSITY = $\frac{e_{max} - e}{e_{max} - e_{min}}$
n	POROSITY
w	WATER CONTENT
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
w_s	SHRINKAGE LIMIT
I_p	PLASTICITY INDEX = $w_l - w_p$
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
A_c	ACTIVITY = $\frac{I_p \text{ of soil}}{\% < 2\mu m \text{ Soil Fraction}}$
Om	ORGANIC MATTER CONTENT
S_r	DEGREE OF SATURATION
S	SENSITIVITY = $\frac{S_u \text{ (undisturbed)}}{S_u \text{ (remoulded)}}$

STRENGTH PARAMETERS

ϕ	ANGLE OF SHEARING RESISTANCE
τ_f	PEAK SHEAR STRENGTH
τ_R	RESIDUAL SHEAR STRENGTH
c	COHESION INTERCEPT
$\sigma_1, \sigma_2, \sigma_3$	NORMAL PRINCIPAL STRESSES
u	PORE WATER PRESSURE
u_e	EXCESS u
r_u	PORE PRESSURE RATIO
q_u	UNCONFINED COMPRESSIVE STRENGTH
s_u	UNDRAINED SHEAR STRENGTH
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO
E	MODULUS OF ELASTICITY
G	MODULUS OF SHEAR DEFORMATION
k_s	MODULUS OF SUBGRADE REACTION
m, n	STABILITY COEFFICIENTS
A, B	PORE PRESSURE COEFFICIENTS

NOTE: EFFECTIVE STRESS PARAMETERS ARE DENOTED BY USE OF APOSTROPHE ABOVE THE SYMBOL, THUS: ϕ' = EFFECTIVE ANGLE OF SHEARING RESISTANCE; σ' = EFFECTIVE NORMAL STRESS

HYDRAULIC TERMS

h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
j	SEEPAGE FORCE PER UNIT VOLUME
η	COEFFICIENT OF VISCOSITY
k	COEFFICIENT OF HYDRAULIC CONDUCTIVITY
k_h	k IN HORIZONTAL DIRECTION
k_v	k IN VERTICAL DIRECTION
m_v	COEFFICIENT OF VOLUME CHANGE
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX
C_r	RECOMPRESSION INDEX
d	DRAINAGE PATH DISTANCE
T_v	TIME FACTOR
U	DEGREE OF CONSOLIDATION
O_r	OVERCONSOLIDATION RATIO (OCR)

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Unified Soil Classification System (USCS) as outlined by the Ministry of Transportation. Different classification systems may be used by others; one such system is the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), as outlined in the Canadian Foundations Engineering Manual. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

UNIFIED SOIL CLASSIFICATION

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

0.002	0.006	0.02	0.06	0.2	0.6	2.0	6.0	20	60	200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Notes On Soil Descriptions

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay and Silt	<0.075 mm		
Sand	0.075 to 4.75 mm	"trace" (e.g. Trace sand)	0% to 10%
Gravel	4.75 to 75 mm	"some" (e.g. Some sand)	10% to 20%
Cobbles	75 to 200 mm	with (e.g. with sand)	20% to 35%
Boulders	>200 mm	and (e.g. and sand)	35% to 50%

For a given material listed as an adjective (e.g. silty sand) means the predominant grain size is sand sized with 30 to 40% silt sized particles.

The compactness of Cohesionless soils and the consistency of the cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	**Standard Penetration Resistance "N" Blows/ 0.3 m**	**Consistency**	**Undrained Shear Strength (kPa)**
Very Loose	0 to 5	Very soft	<12
Loose	5 to 10	Soft	12 to 25
Compact	10 to 30	Firm	25 to 50
Dense	30 to 50	Stiff	50 to 100
Very Dense	Over 50	Very Stiff	100 to 200
		Hard	>200

5. **ROCK CORING**

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundless of the rock mass. It is obtained from the rock cores by summing the length of the core covered, counting only those pieces of sound core that are 100 mm or more length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

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Brampton, Ontario L6T 4V1

RECORD OF BOREHOLE No BH-1

SHEET 1 OF 1 **METRIC**

PROJECT NO. GWP 5725-04-00	LOCATION South Bound Lane STA 10+385.7 @38.72 m RT CL Median	ORIGINATED BY JJ
DIST Muskoka HWY 400-69	BOREHOLE TYPE CME 200 mm Ø HSA	COMPILED BY TA
DATUM Geodetic, Ground Surface	DATE 11/25/2006 -	CHECKED BY TC

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / SPT TEST (N-Value) ● DYNAMIC CONE PENETRATION 20 40 60 80 100 / SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ⊗ QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100 / PLASTIC LIMIT PL, NATURAL WATER CONTENT w, LIQUID LIMIT LL; WATER CONTENT (%) 10 20 30 / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
231.7									
231.6 0.1	**TOPSOIL**, (50mm) **SAND FILL**, brown, moist to wet, loose to compact, with cobbles, trace fine fibrous organics, trace to some silt.	1	SPT	18		14	78	6	2
					231				
		2	SPT	8					
		3	SPT	60/100mm		36	55	9	
230.0 1.7	**BIOTITE GNEISS**, light grey to pink with frequent dark grey bands, coarse grained, occasional fractures, thin, very stong.				230				
		4	NQ		229				
228.5 3.2	END OF BOREHOLE AT ~ 3.20 m DEPTH ROCK CORE: AT ~ 1.65-3.20 m depth REC=100% RQD=30%								

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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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RECORD OF BOREHOLE No BH-2

SHEET 1 OF 1 **METRIC**

PROJECT NO. GWP 5725-04-00 LOCATION North Bound Lane STA10+205.7 @37.19 m LT CL Median ORIGINATED BY JJ

DIST Muskoka HWY 400-69 BOREHOLE TYPE CME 200 mm Ø HSA COMPILED BY TA

DATUM Geodetic, Ground Surface DATE 11/25/2006 - CHECKED BY TC

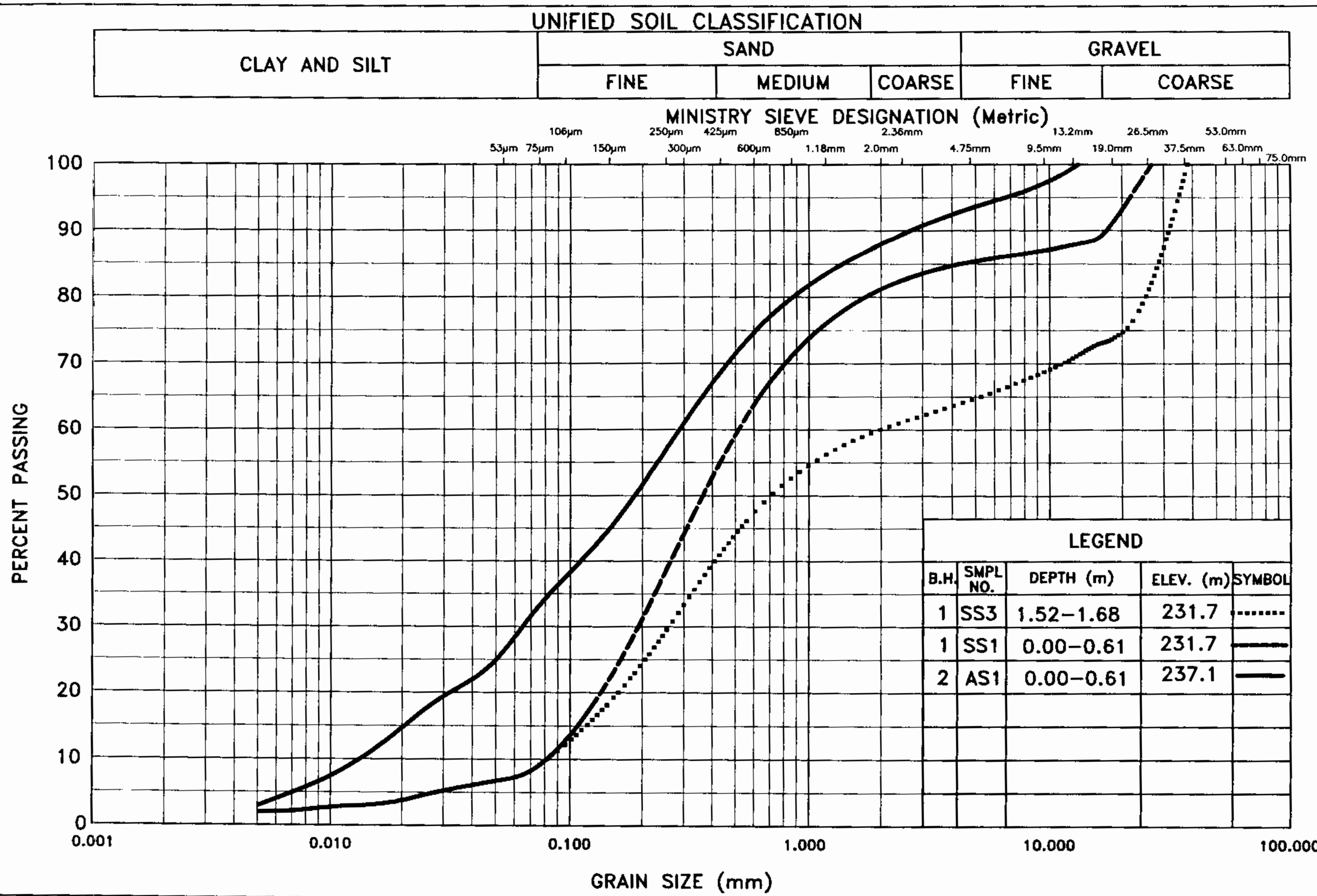
ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SPT TEST (N-Value) / DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL WATER CONTENT / LIQUID LIMIT (WATER CONTENT %)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.1 / 0.0	**ORGANIC SILT,** brown, damp, loose, with fine grained sand.	1	BAG			237				6 60 31 3
236.5 / 0.6	**MIGMATITE,** with occasional bands of quartsfeldspathic gneiss, fresh, strong, medium grained.	2	NQ			236				
235.0 / 2.1	END OF BOREHOLE AT ~ 2.1 m DEPTH ROCK CORE: AT ~ 1.61-2.1 m depth REC=100% RQD=97%									

Legend: SPT TEST (N-Value) ● ; DYNAMIC CONE PENETRATION (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PL — w — LL; WATER CONTENT (%) 10 20 30

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

APPENDIX C

Laboratory Data



Ministry of Transportation

Ontario

METRIC

GRAIN SIZE DISTRIBUTION

FIGURE No. D2

G.W.P. 478–98–00

MOON RIVER FOUNDATION

REF. S010479G