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**FOUNDATION INVESTIGATION AND DESIGN REPORT
VARIABLE MESSAGE SIGN #13
HIGHWAY 17 WESTBOUND, APPROXIMATELY
1.4 KM WEST OF GORMANVILLE ROAD
NORTH BAY, ONTARIO
G.W.P. 5762-04-00, W.P. 5765-04-01**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by IBI Group (IBI) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design for a variable message sign (VMS #13) on Highway 17 Westbound, in North Bay, Ontario. The general location of the site is shown on the Key Plan on Drawing 1.

The terms of reference for the scope of work were outlined in Golder's Proposal P7-1191-0039, dated June 25, 2007, that formed part of the Consultant's Agreement (Number 5006-E-0083) for this project. The work was carried out in accordance with Golder's Quality Control Plan for this project dated October 2007. The site plan showing the proposed sign location was provided to Golder by IBI in January 2008.

We understand the proposed structure will be a sign mounted on a single pole and supported by a spread footing.

2.0 SITE DESCRIPTION

The site of the proposed VMS #13 is situated on Highway 17 approximately 1.4 km west of Gormanville Road at Station 10+076 located in North Bay, Widdifield Township, Ontario. This section of Highway 17 consists of two eastbound lanes and one westbound lane; the westbound gravel shoulder is approximately 4.3 m wide and a shallow drainage ditch parallels the north side of the westbound lane shoulder. The ground surface at the proposed structure location is approximately Elevation 214.0 m.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The subsurface investigation work for the VMS #13 structure was carried out on November 13, 2007, at which time one sampled borehole numbered BH07-1 and two Dynamic Cone Penetration Tests (DCPTs) numbered DCPT07-1 and DCPT07-2 were advanced at the locations shown on Drawing 1. Borehole BH07-1 was drilled on the shoulder of the westbound lane, approximately 0.5 m from the centre of the proposed sign location; Dynamic Cone Penetration Tests DCPT07-1 and DCPT07-2 were located about 5.0 m east and 5.0 m west of Borehole BH07-1, respectively.

The foundation investigation was carried out using a truck-mounted CME-55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The borehole was advanced using hollow stem augers. Soil samples were obtained at intervals of depth ranging from 0.75 m to 1.5 m, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586-99). Borehole BH07-1 was advanced to a depth of 9.9 m below existing ground surface and the two DCPTs were advanced to a depth of 9.1 m below existing ground surface. Details of the subsurface conditions encountered at the borehole and DCPT locations are shown on the Record of Borehole and Penetration Test sheets, respectively, following the text of this report. The borehole was backfilled with a bentonite holeplug to the ground surface in accordance with Ontario Regulation 128 (Amendment to O. Reg. 903); the DCPT holes were allowed to collapse as the drill rod string was extracted.

The fieldwork was supervised throughout by members of Golder's engineering and technical staff, who located the borehole and DCPTs, arranged for the clearance of underground services and traffic protection, supervised the drilling, sampling and in situ testing operations, logged the borehole and DCPTs, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing.

The borehole and DCPTs were located using a measuring tape relative to the stake positioned in the field by IBI. The as-drilled borehole and DCPT locations (relative to MTM NAD83 system) and the ground surface elevations (referenced to Geodetic datum) were subsequently surveyed by IBI and forwarded to Golder. The borehole and DCPT locations are depicted on Drawing 1 and the borehole and DCPT coordinates and ground surface elevations are presented below.

Borehole / DCPT Number	MTM NAD83 Zone 17 Northing (m)	MTM NAD 83 Zone 17 Easting (m)	Ground Surface Elevation (m)
BH07-1	5132733.7	304423.5	214.0
DCPT07-1	5132734.0	304428.6	214.0
DCPT07-2	5132733.0	304418.2	214.0

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

Based on terrain mapping by the Ontario Geological Survey¹, the subsurface soils in the vicinity of the site consist of glaciolacustrine deposits comprising sand and silt and/or silt and clay.

4.2 Site Stratigraphy

Detailed descriptions of the subsurface soil and groundwater conditions as encountered in Borehole BH07-1 and Dynamic Cone Penetration Tests DCPT07-1 and DCPT07-2 advanced during this investigation, together with the results of the laboratory tests carried out on selected samples, are given on the Record of Borehole sheet and Record of Penetration Test sheets following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheet are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary beyond the borehole location.

As indicated previously, Borehole BH07-1 was drilled on the shoulder of the westbound lane, approximately 0.5 m southwest of the proposed sign location. Dynamic Cone Penetration Tests DCPT07-1 and DCPT07-2 were advanced 5.0 m east and 5.0 m west of Borehole BH07-1, respectively, to confirm the relative density of the subsurface deposits. The results of the two DCPTs indicate that the subsurface conditions at the DCPT and borehole locations were relatively uniform.

4.2.1 Fill

Sandy gravel to sand fill was encountered to a depth of 1.4 m below the existing ground surface. The fill had a compact relative density based on an SPT N value of 23 blows per 0.3 m of penetration.

The natural water content of a sample of the fill material was about 6 percent. A grain size distribution test on a sample of the fill is shown on Figure 1.

¹ Northern Ontario Engineering Geology Terrain Study, OGS Map 5041

4.2.2 Sand

Two deposits of sand were encountered within the borehole, separated by a silt deposit as described in Section 4.2.3. The upper sand deposit contained trace gravel and trace silt while the lower sand deposit contained some silt and trace clay. The upper deposit was encountered below the sand fill at a depth of 1.4 m, corresponding to Elevation 212.6 m, and was approximately 2.6 m thick; the lower deposit was encountered at a depth of 6.9 m, corresponding to Elevation 207.1m, and was approximately 1.6 m thick.

The measured SPT 'N' values within the sand deposits ranged from 2 to 30 blows per 0.3 m of penetration, indicating that the sand has a very loose to compact relative density. The lower 'N' value of 2 blows per 0.3 m of penetration is attributed to base heaving of the sand into the hollow stem augers due to groundwater pressure.

The natural water content of the sand deposits ranged from 16 percent to 22 percent. Grain size distribution tests on two samples of the sand are shown on Figure 2.

4.2.3 Silt

A silt deposit was encountered between the two sand deposits at a depth of approximately 4.0 m below existing ground surface, corresponding to Elevation 210.0 m. The thickness of the silt deposit was approximately 2.9 m.

The measured SPT 'N' values within the silt deposit were 2 and 5 blows per 0.3 m of penetration, indicating that the deposit has a very loose to loose relative density.

The natural water contents of two samples of the silt deposit were between 19 percent and 31 percent. The results of a grain size distribution test carried out on a sample of the silt are shown on Figure 3.

4.2.4 Clayey Silt

Beneath the lower sand deposit, the borehole encountered a grey clayey silt deposit at a depth of approximately 8.5 m below existing ground surface, corresponding to Elevation 205.5 m. The clayey silt layer extended to the bottom of the borehole at a depth of 9.9 m (Elevation 204.1 m).

The measured SPT 'N' values within the clayey silt were 2 blows per 0.3 m of penetration, indicating the clay stratum had a very soft to soft consistency. An attempt was made to push an N vane to determine undrained shear strengths of the deposit at a depth of 9.9 m below ground surface; however, the vane could not be advanced into the deposit.

An Atterberg limits test carried out on the sample of the clayey silt provided a liquid limit of 31 percent, a plastic limit of 20 percent, corresponding to a plasticity index of 11 percent. As shown on Figure 4, this soil is classified as clayey silt of low plasticity.

The measured natural water content of the clayey silt deposit was about 36 percent.



4.2.5 Groundwater Conditions

Details of the groundwater conditions and water level observed in the open borehole at the time of drilling are summarized on the Record of Borehole sheet following the text of this report. In general, the samples taken in the borehole were noted to be wet and the sand and silt deposits are water-bearing. The groundwater level observed in the open borehole was recorded at a depth of 1.7 m below the existing ground surface upon completion of drilling, corresponding to Elevation 212.3 m. It should be noted that this water level does not represent the stabilized water level and that the groundwater elevation will fluctuate seasonally depending on precipitation and local soil permeability and should be expected to rise during wet periods of the year.


5.0 CLOSURE

The fieldwork for this project was carried out by a technician from our Sudbury office under the coordination of Mr. André Bom, P.Eng. This report was prepared by Mr. André Bom, P.Eng., a Geotechnical Engineer, and the technical aspects were reviewed by Mr. André Zerwer, P.Eng., Associate and Senior Geotechnical Engineer and Mr. Jorge M.A. Costa, P.Eng. a Principal with Golder. Mr. Costa, also a Designated MTO Contact for Golder, conducted a quality control review of the report.

GOLDER ASSOCIATES LTD.



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PART B

**FOUNDATION DESIGN REPORT
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed variable message sign (VMS #13). The recommendations are based on interpretation of the factual data obtained from the borehole and DCPTs advanced during the subsurface investigation at this site and from site observations. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation design alternatives and to design the proposed sign foundation. As such, where comments are made on construction, they are provided only in order to highlight those aspects which could affect the planning of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Sign Foundation

We understand the proposed sign will be located on the north side of Highway 17 with the centre of the footing located approximately 4.5 m from the edge of pavement at Station 10+076 facing the westbound traffic. Borehole BH07-1 was advanced approximately 0.5 m from the centre of the proposed sign location (i.e. approximately 4.0 m from the edge of the pavement); DCPT07-1 and DCPT07-2 were advanced 5.0 m to the east and to the west of Borehole BH07-1 along the shoulder of Highway 17. Borehole BH07-1 encountered sandy gravel to sand fill to a depth of 1.4 m below the existing ground surface. A very loose to compact sand deposit was encountered from 1.4 m to 4.0 m depth below existing ground surface and was underlain by a loose to very loose silt to a depth of 6.9 m below ground surface. Below the silt deposit, loose to very loose sand was encountered to a depth of 8.5 m. A 0.6 m to 1.4 m thick layer of very soft clayey silt was observed beneath the lower sand deposit that extended to the bottom of the borehole. The unstabilized groundwater level in the open borehole upon completion of drilling was 1.7 m below the existing ground surface.

Overhead sign supports are typically designed with a standard caisson foundation in accordance with the requirements in MTO's *Sign Support Manual*. However, the standard foundation design provided in MTO's *Sign Support Manual* does not apply to sites where there are extensive poor fill materials or materials softer or looser than outlined in the standard cases. Based on a review of the subsurface information, the soils at this sign location have lower friction angles than required for the standard caisson design. For the observed subsurface conditions, a site-specific caisson design is required.

The following sections provide site-specific geotechnical design parameters for a caisson foundation. In addition, geotechnical design parameters are provided for a spread footing, which is the option preferred by IBI.

6.2.1 Caisson Foundations

Caisson foundations for overhead sign supports should be designed in accordance with the requirements in MTO's *Sign Support Manual*. The *Sign Support Manual* includes a standard caisson foundation design (Section 4 and Standard Drawings SS118-3, SS118-4 and SS118-5), in which the caissons are extended 5 m below the design frost depth except where bedrock is encountered within this depth. Based on the depth of frost penetration isopleths for Northern Ontario², the depth of frost penetration for the North Bay area is approximately 1.9 m; the typical caisson founding level would therefore be 6.9 m below the ground surface. At this site, bedrock was not encountered at the borehole or DCPT locations; therefore, the foundation should be designed as caissons in soil. The standard design is based on the following minimum soil conditions:

- **Case 1 (Cohesionless Soils):** Sand with a friction angle of 28 degrees surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and sand with a friction angle of 30 degrees surrounding the lower third of the portion of the caisson below the design frost depth.
- **Case 2 (Cohesive Soils):** Soft clay with an undrained shear strength of 25 kPa surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and "soft" clay with an undrained shear strength of 50 kPa surrounding the lower third of the portion of the caisson below the design frost depth.

As indicated in Section 6.2, for the existing subsurface conditions, a site-specific design is required. The stratigraphy and design parameters for the subsurface conditions encountered in the borehole at the sign support location are given in Table 1.

For cohesionless soils, the unfactored passive lateral earth pressure, P_p (kPa), distributed along the depth of the caisson foundation, may be calculated using the following equations:

$$\begin{aligned} P_p &= K_p \gamma d_w && \text{above the groundwater table; and} \\ P_p &= K_p \gamma d_w + K_p \gamma' (d - d_w) && \text{below the groundwater table} \end{aligned}$$

where K_p is the passive earth pressure coefficient;
 γ is the bulk unit weight (kN/m³);
 γ' is the effective unit weight below the groundwater level (kN/m³);
 d is the depth below the ground surface (m); and
 d_w is the depth to the groundwater level (m).

For cohesive soils, P_p may be calculated using the following equation:

$$P_p = \sigma_z + 2 c_u$$

where σ_z is the total vertical stress (kPa);
 $2 c_u$ is the undrained shear strength (kPa).

The unfactored lateral earth pressure may be assumed to act over an equivalent pile width equal to three times the caisson diameter. A resistance factor of 0.5 should be applied to this calculated lateral resistance in order to obtain the factored lateral geotechnical resistance at the Ultimate Limit States (ULS). In the design of the foundations, the passive resistance within the upper 1.9 m below ground surface should be neglected to account for frost action.

6.2.2 Spread Footing

Alternatively, the sign could be founded on a spread footing. As noted in Section 6.2, the unstabilized groundwater level in Borehole BH07-1 was encountered at a depth of 1.7 m below the existing ground surface and, as indicated in Section 6.2.1, the depth of frost penetration for the North Bay area is approximately 1.9 m; therefore, the founding depth of the spread footing should be at a minimum of 1.9 m below final ground surface.

Based on the observed water level and moisture contents, the groundwater level is anticipated to be at or above the proposed founding depth. Open cut excavations may not be practical for the proposed footing construction due to the anticipated shallow groundwater level and the proximity of Highway 17 adjacent to the proposed excavation. Should open cut excavation be proposed at this site, the excavations should be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OSHA) for Construction Projects. When referencing to OSHA, the fill materials above the groundwater level may be classified as "Type 3 Soil". Fill below the water table and the sand and underlying silt deposits are classified as "Type 4 Soil". Therefore, the entire excavation should be sloped no steeper than 3 horizontal to 1 vertical, and may have to be flatter due to running sand and silt, even if dewatering is carried out ahead of the excavation process.

Shoring and possibly controlled dewatering will likely be required at this site to excavate the subsoils to the proposed founding depth. Basal instability due to groundwater pressures may occur during construction and should be considered in the shoring design. If controlled dewatering is carried out, the impacted area should be carefully monitored to mitigate settlement of the existing highway pavement structure. Should shoring and dewatering be used during construction, the contractor should design the shoring in accordance with MTO's Protection Systems Special Provision No. 105S19, to meet Performance Level 2.

² Ontario Provincial Standard Drawing (OPSD) 3090.100

Relevant shoring design parameters are provided with respect to depth for the subsoils listed in Table 1.

Design Parameter	Fill, Native Sand, Silt
Unit Weight above Groundwater Level γ (kN/m ³)	18
Unit Weight below Groundwater Level γ' (kN/m ³)	8
Friction Angle ϕ (°)	27
K_a^*	0.37
K_p^*	2.66
K_o^*	0.45

* Earth pressure coefficients for horizontal backfill.

During construction, stockpiles should be placed well away from the edge of the excavation, and their height should be controlled so they do not surcharge the sides of the excavation and/or overall slope. Generally speaking, for this site, the distance between the crest of the excavation and the toe of the stockpile should generally be greater than the diameter of the base of the stockpile.

As noted above, the subsoils, including fill materials, should be considered as Type 3 soil above the water table in accordance with the most recent guidelines of the Occupational Health and Safety Act. Below the water table, the subsoils should be considered Type 4 soils.

Disturbance of the underlying materials during construction of the spread footing could influence the settlement of the structure. MTO's Special Provision No. 902S01 should be included in the Contract Documents, requiring inspection and approval of the foundation area by the Quality Verification Engineer (QVE) prior to footing construction, to ensure adequate preparation of the foundation areas.

The base of the excavation should be free of water and loose soil prior to placing concrete. Should the materials at bearing level become saturated or disturbed, we recommend that the affected material be removed immediately prior to placing concrete. We recommend that the prepared subgrade be protected using a 150 mm thick mud mat comprised of a minimum 5 MPa concrete or working mat composed of a minimum 300 mm thick layer of compacted Granular 'B' Type II or Granular 'A' meeting MTO's Special Provision SP110S13. The mud mat or working mat should be placed across the bottom of the excavation immediately upon completion of the excavation and review by the QVE, to limit disturbing the native soils and to provide a platform for constructing the spread footing.

Upon completion of foundation construction and excavation backfilling, we understand that the existing ditch will be realigned around the proposed sign. The final grade at the sign location should be regraded to incorporate the realigned ditch with side slopes of 2 horizontal to 1 vertical or flatter.

6.2.3 Geotechnical Resistance

Spread footings constructed on the properly prepared subgrade at or below the depth given in Section 6.2.2 (1.9 m or Elevation 212.1 m) may be designed based on a factored geotechnical axial resistance of 200 kPa at ULS for a spread footing, rectangular in shape up to 5.0 m long by 2.5 m wide. For the same spread footing dimension indicated above, a geotechnical axial resistance value of 75 kPa for Serviceability Limit States (SLS; for 25 mm settlement) design may be assumed. Design of the proposed sign foundation should also be checked for and provisions made to resist buoyant forces.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing dimensions or founding depth differs from those given above.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be considered in accordance with Clause 6.7.4 and C6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its commentary.

6.2.4 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the prepared subgrade should be calculated in accordance with Clause 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the generally loose to compact sand or compacted granular pad, the coefficient of friction, $\tan \phi'$, can be taken as 0.45. This value is unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3 Construction Considerations

The excavation around and above the spread footing may be backfilled using an approved granular material meeting MTO's Special Provision SP110S13 such as Granular 'A' or 'B' Type II placed in 300 mm loose lifts and uniformly compacted to 95 percent of the standard Proctor maximum dry density of the respective material. The use of native excavated materials as backfill material is not recommended. The final grade surrounding the sign should be sloped to promote surface water drainage and pavement structure drainage away from the pavement and sign, to the adjacent ditch.

We recommend that a Non-Standard Special Provision (NSSP) be included in the Contract Documents to warn the contractor of the following item which is expected to affect the installation of the variable message sign foundation:

- **Control of overburden soils for spread footings:** Excavations for the sign foundation will be advanced through generally cohesionless soils to potentially below the groundwater level, and should be expected to be unstable above and below the groundwater level at this site. It should be anticipated that the excavations will have to be advanced using shoring, possibly in conjunction with controlled dewatering or with fluid support, in order to minimize ground loss during excavation and concrete placement. The contractor is responsible for ensuring that appropriate construction procedures and equipment are used for spread footing construction. An example NSSP to warn the contractor of such conditions is presented in Appendix A.

7.0 CLOSURE

This report was prepared by Mr. André Bom, P.Eng., a Geotechnical Engineer, and the technical aspects were reviewed by Mr. André Zerwer, P.Eng., Associate and Senior Geotechnical Engineer and Mr. Jorge M.A. Costa, P.Eng. a Principal with Golder. Mr. Costa, also a Designated MTO Contact for Golder, conducted a quality control review of the report.

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TABLE 1
SOIL PARAMETERS FOR SIGN FOUNDATION DESIGN
VMS #13, HIGHWAY 17 WESTBOUND
W.P. 5765-04-01

<i>Borehole No.</i>	<i>Stratum</i>	<i>Depth ¹ (m)</i>	<i>Depth to Groundwater (m)</i>	<i>Design Parameters ²</i>				
				ϕ'	C_u	γ	γ'	K_p
BH07-1	Compact sandy gravel to sand fill	0 to 1.4	1.7	27	-	18	8	2.7
	Compact to very loose sand	1.4 to 4.0		27	-	18	8	2.7
	Loose to very loose silt	4.0 to 6.0		27	-	18	8	2.7
	Loose sand	6.0 to 8.5		27	-	18	8	2.7
	Very soft to soft clayey silt	8.5 to 9.9		-	15	17	7	n/a

- NOTES:**
1. Depths are given below the ground surface at the borehole location.
 2. Design parameters: ϕ' = effective friction angle (degrees);
 C_u = undrained shear strength (kPa);
 γ = bulk unit weight (kN/m³);
 γ' = effective unit weight below the groundwater level (kN/m³); and
 K_p = passive earth pressure coefficient.

Compiled by: **AB**
Checked by: **JMAC**

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Cone Penetration Resistance, N_d :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezcone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency

	C_u, S_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. stress: $\Delta\sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity).

(a) Index Properties (continued)

w	water content
w_L	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_L - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p)/I_p$
I_c	consistency index $= (w_L - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

PROJECT <u>07-1191-0039-13</u>			RECORD OF BOREHOLE No BH07-1			1 OF 1 METRIC		
W.P. <u>5765-04-01</u>			LOCATION <u>N 5132733.7 ;E 304423.5</u>			ORIGINATED BY <u>ID</u>		
DIST <u> </u> HWY <u>17</u>			BOREHOLE TYPE <u>Power Auger, 108mm ID, Hollow stem Augers</u>			COMPILED BY <u>BB</u>		
DATUM <u>Geodetic</u>			DATE <u>November 13, 2007</u>			CHECKED BY <u>AB</u>		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p — W — W _L WATER CONTENT (%) 12.5 25.0 37.5
214.0	GROUND SURFACE							
0.0	Sandy gravel to sand, trace gravel, trace silt (FILL) Compact Brown Moist		1	SS	23		213	○
212.6							212	○
1.4	SAND, trace gravel, trace silt Compact to very loose Brown Wet		2	SS	30			○
			3	SS	7		211	○
	Heaving sands encountered in hollow stem augers at 3.0m and 3.6m depths.		4	SS	2			○
210.0							210	
4.0	SILT, some sand, trace clay Loose to very loose Grey Wet		5	SS	5		209	○
							208	
			6	SS	2			○
207.1							207	
6.9	SAND, some silt, trace clay Loose Grey Wet		7	SS	7		206	○
205.5							205	
8.5	CLAYEY SILT Very soft to soft Grey Wet		8	SS	2			○
204.1								
9.9	End of borehole							
	Note: 1. Water level encountered at a depth of 1.7m (Elev. 212.3m) upon completion of drilling. 2. Unable to push N vane below a depth of 9.9m.							

MIS-MTO 001 NORTH BAY SOIL.GPJ GAL-MISS.GDT 4/17/08 ACM



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

MIS-MTO 001 NORTH BAY SOIL.GPJ GAL-MISS.GDT 4/17/08 ACM



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

MIS-MTO 001 NORTH BAY SOIL.GPJ GAL-MISS.GDT 4/17/08 ACM

CONT No.
WP No. 5765-04-01

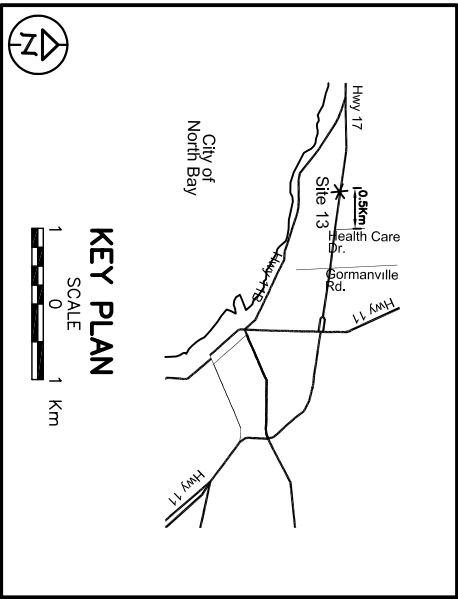


VARIABLE MESSAGE SIGN #13
HIGHWAY 17 WESTBOUND, NORTH BAY
BOREHOLE AND DCPT LOCATIONS

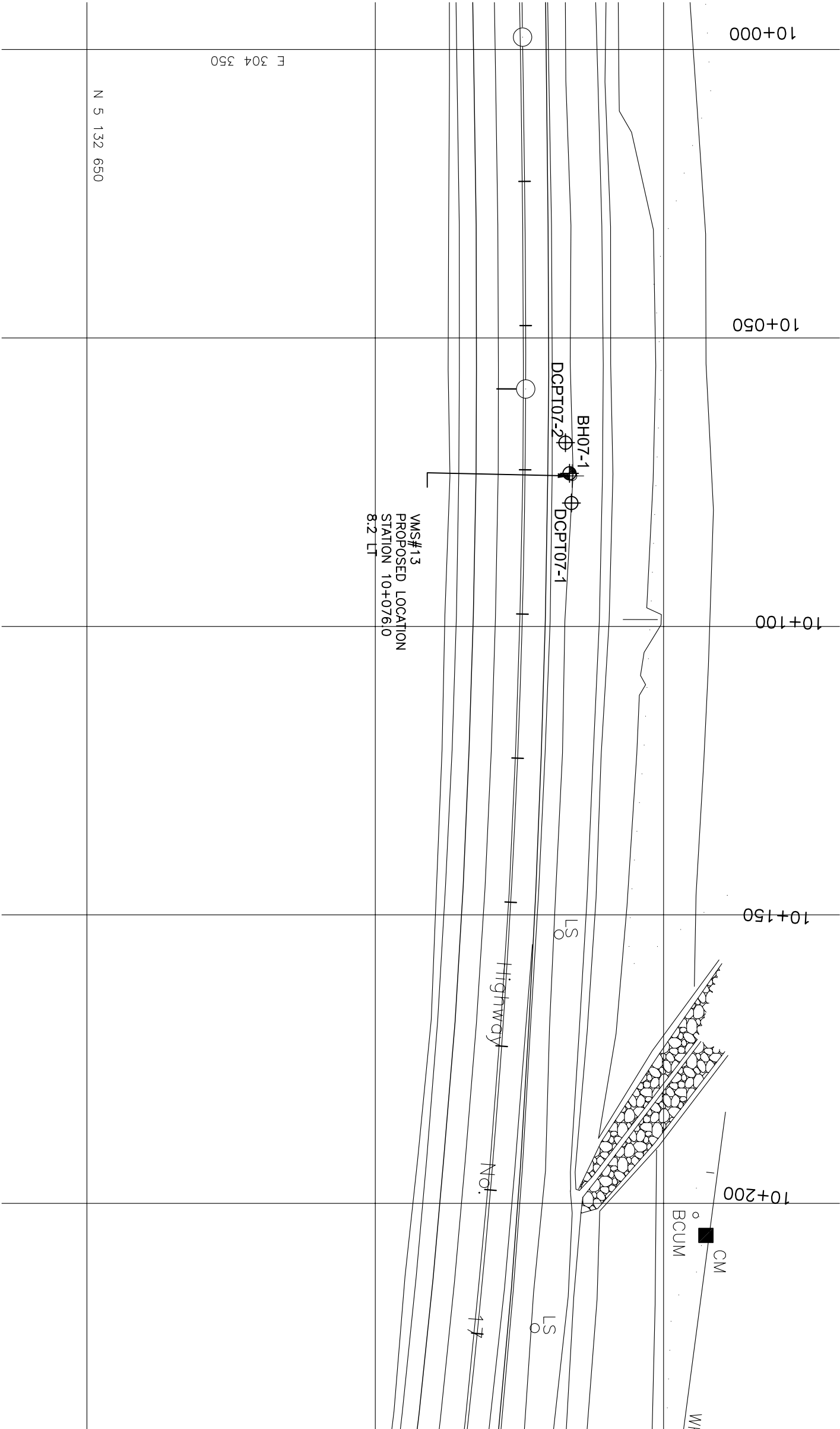
SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND			
	Borehole Location		
	Dynamic Cone Penetration Test Location		
No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
BH07-1	214.0	5132733.7	304423.5
DCPT07-1	214.0	5132734.0	304428.6
DCPT07-2	214.0	5132733.0	304418.2



NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview, information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OF-S General Conditions.

REFERENCE

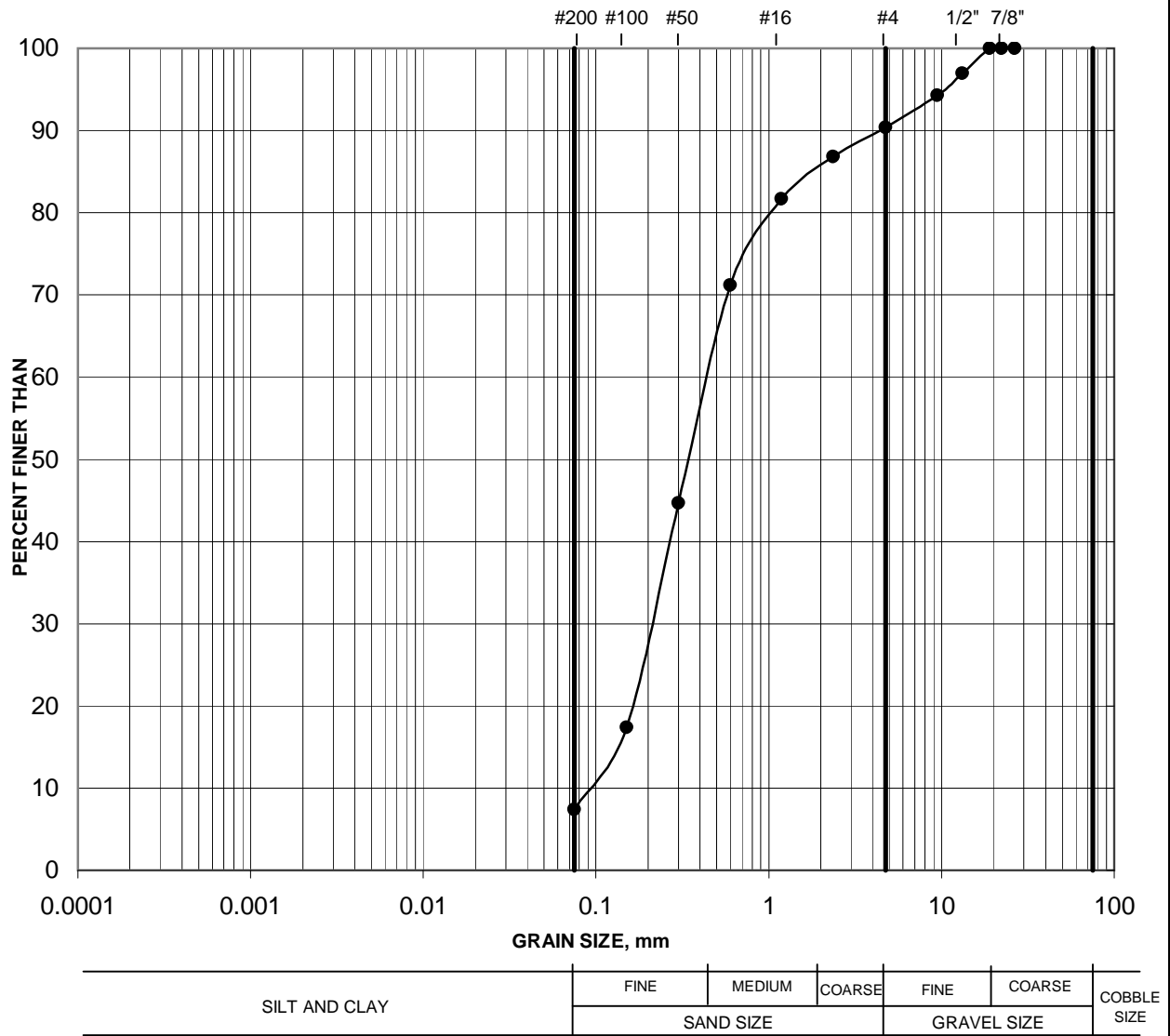
Base plans provided in digital format by IBI, drawing file no. kplnns.dwg and VSM#13.dwg dated January, 2008.

NO.	DATE	BY	REVISION	
Geocres No. 31L-121				
HWY. 17		PROJECT NO. 07-1191-0039-13		DIST.
SUBWD. AB	CHKD. JMAC	DATE: APR. 2008		SITE:
DRAWN: BB	CHKD. AB	APPD. JMAC	DWG. 1	

GRAIN SIZE DISTRIBUTION

Sand (FILL)

FIGURE 1



Borehole	Sample	Elevation (m)
07-1	1	212.9

Project: 07-1191-0039-13

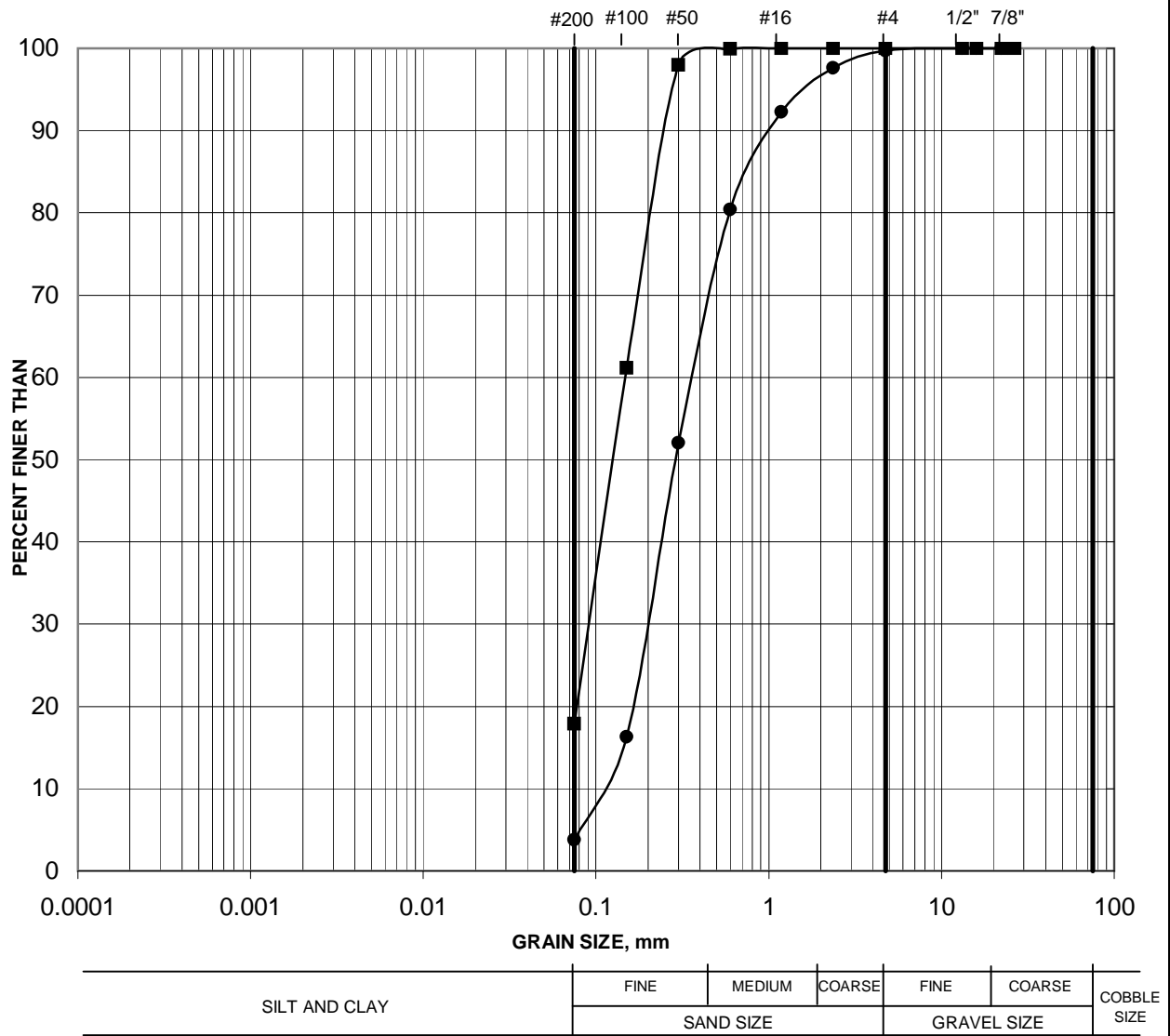
Golder Associates

April 2008

Created by:	SL
Checked by:	AB

GRAIN SIZE DISTRIBUTION SAND

FIGURE 2



	Borehole	Sample	Elevation (m)
●	07-1	3	211.4
■	07-1	7	206.1

Project: 07-1191-0039-13

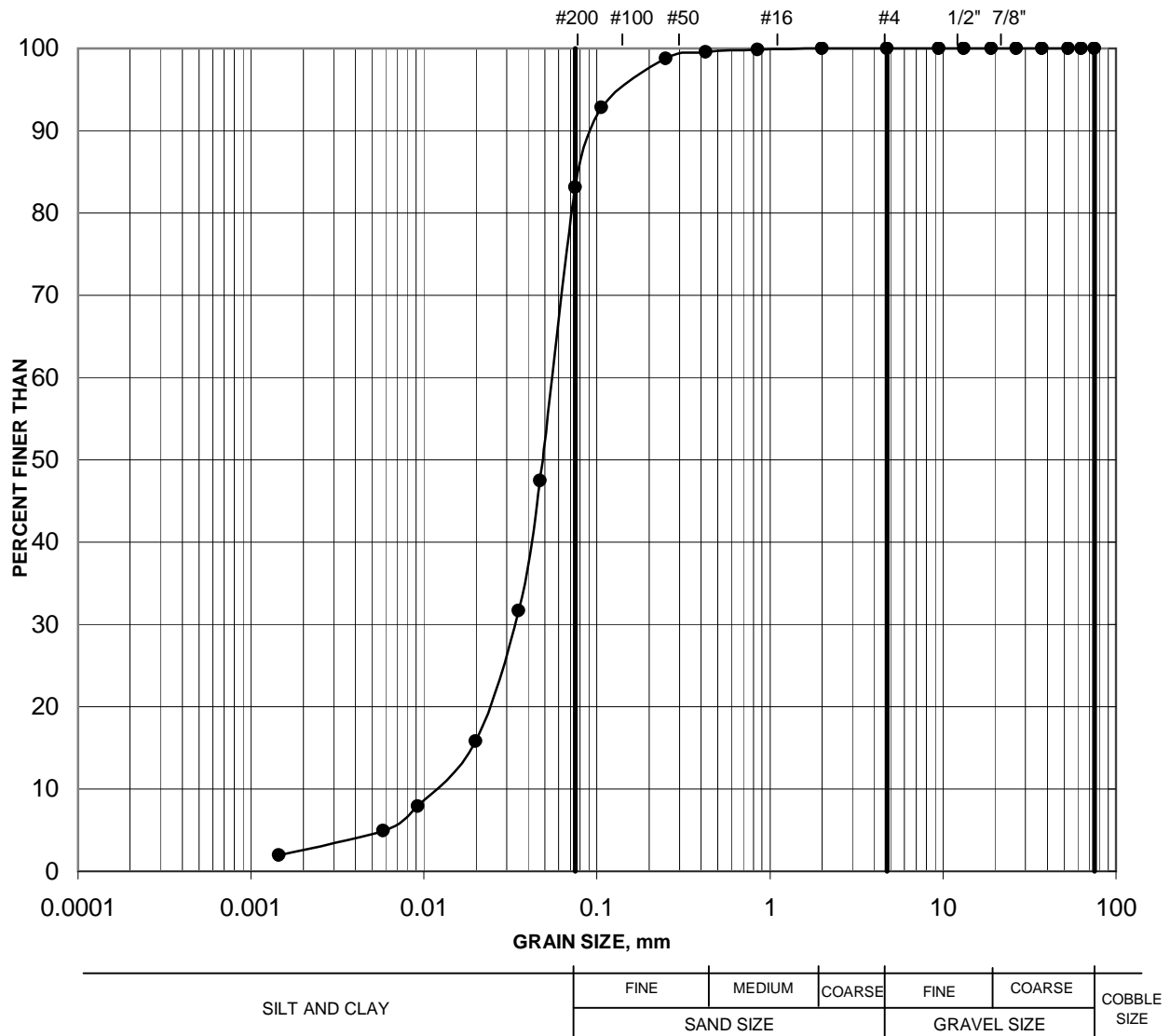
Golder Associates

April 2008

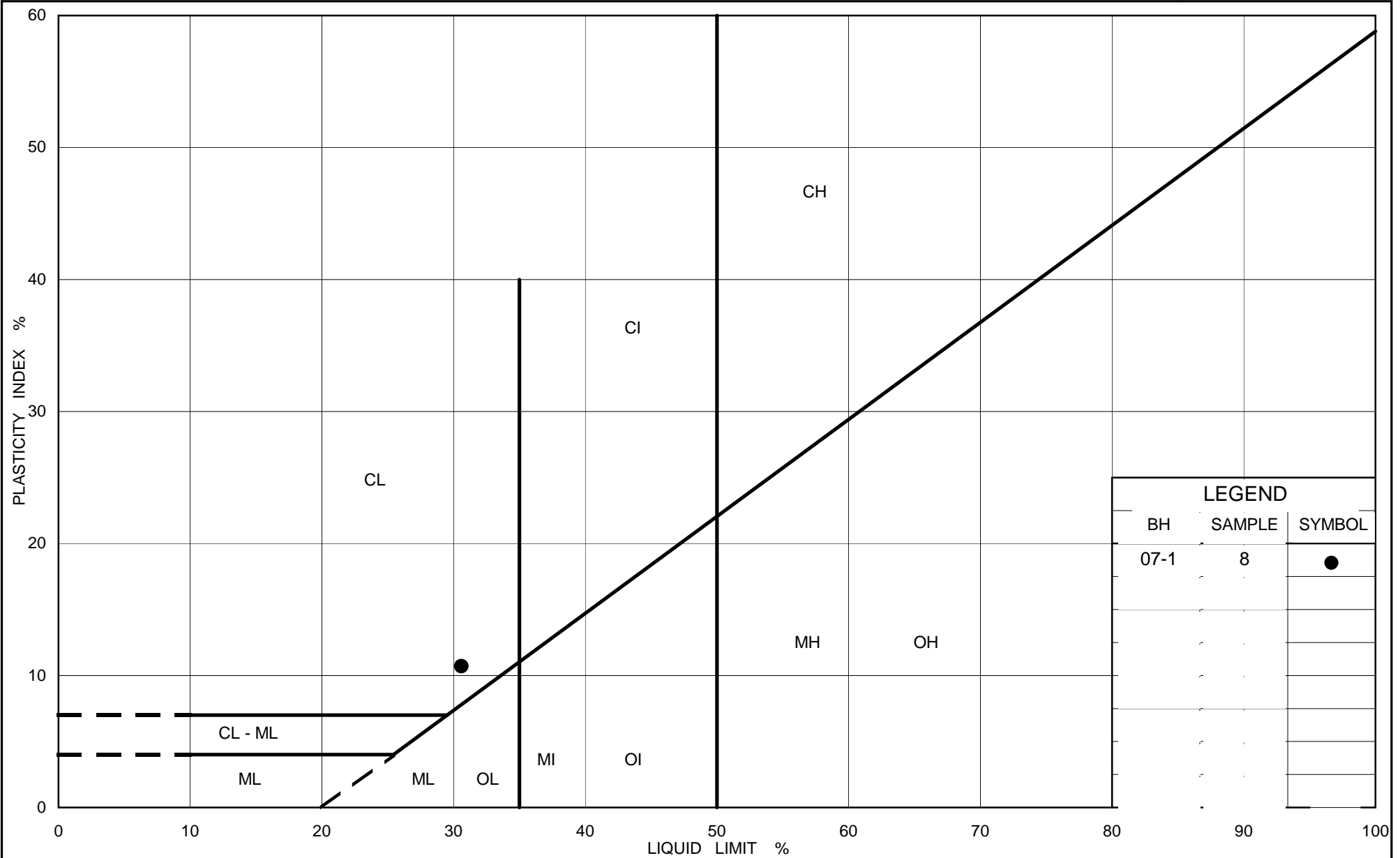
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Checked by:	AB

GRAIN SIZE DISTRIBUTION SILT

FIGURE 3



Borehole	Sample	Elevation (m)
07-1	5	209.1



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

FIGURE No. 4

Project No. 07-1191-0039-13

Checked By: JMAC

APPENDIX A
NON-STANDARD SPECIAL PROVISION

CONTROL OF OVERBURDEN SOILS FOR FOUNDATION EXCAVATIONS -
Item No.

Non-Standard Special Provision

The contactor is hereby notified that the overburden soils at the VMS#13 sign location include cohesionless and water-bearing sand and silt, which are susceptible to soil cave-in, sloughing and boiling. The contractor shall ensure that appropriate construction procedures and equipment are employed.

END OF SECTION