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**FOUNDATION INVESTIGATION AND DESIGN REPORT
VARIABLE MESSAGE SIGN #12
HIGHWAY 11 WESTBOUND, APPROXIMATELY
0.9 KM EAST OF HIGHWAY 101 WEST JUNCTION
MATHESON, ONTARIO
G.W.P. 5762-04-00, W.P. 5674-04-01**

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 #12) on Highway 11 westbound near Matheson, 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, as well as Change Request 1 – Credit, dated November 2, 2007. The work was carried out in accordance with Golder's Quality Control Plan for this project dated October 2007. The site plan detailing the proposed sign location was provided to Golder by IBI in January 2008.

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

2.0 SITE DESCRIPTION

The site of the proposed VMS #12 is located in Bowman Township on Highway 11, approximately 0.9 km east of the intersection with Highway 101 west junction at Station 10+827, near Matheson, Ontario. This section of Highway 11 consists of one eastbound lane and one westbound lane. The westbound gravel shoulder is approximately 3 m wide and a shallow drainage ditch parallel to the road alignment is present at the toe of the gravel shoulder. On the north side of the ditch, the ground surface slopes upwards at an approximate gradient of 3 horizontal to 1 vertical (3H:1V). The height of this backslope, between the bottom of the ditch and the crest was approximately 3.0 m. The existing ground surface at the proposed structure location (4.5 m from the north edge of pavement) is at approximately Elevation 256.5 m.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The subsurface investigation work for the VMS #12 structure was carried out on November 15, 2007, at which time one sampled borehole, numbered BH07-3, was drilled on the shoulder of the westbound lane, approximately 1.3 m from the centre of the proposed sign location, as shown on Drawing 1.

The foundations 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 108 mm I.D. hollow stem augers and 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 BH07-3 was advanced to a depth of 9.8 m below existing ground surface. Details of the subsurface conditions encountered at the borehole location are shown on the Record of Borehole sheet following the text of this report. The borehole was backfilled with bentonite holeplug to the ground surface in accordance with Ontario Regulation 128 (Amendment to O. Reg. 903).

The fieldwork was supervised throughout by a member of Golder's technical staff, who located the borehole, arranged for the clearance of underground services and for traffic protection, supervised the drilling, sampling and in situ testing operations, logged the borehole, 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 was located using a measuring tape relative to the stake positioned in the field by IBI. The as-drilled borehole location (relative to MTM NAD83 system) and the ground surface elevation (referenced to Geodetic datum) were subsequently surveyed by IBI and forwarded to Golder. The borehole location is depicted on Drawing 1 and the borehole coordinates and ground surface elevation are presented below.

Borehole Number	MTM NAD83 Zone 17 Northing (m)	MTM NAD83 Zone 17 Easting (m)	Ground Surface Elevation (m)
BH07-3	5377800.2	339185.0	257.0

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

Based on terrain mapping by the Ontario Geological Survey, the subsurface soils in the vicinity of the site consist of glaciolacustrine deposits comprising clay and silt¹.

4.2 Site Stratigraphy

Detailed descriptions of the subsurface soil and groundwater conditions as encountered in Borehole BH07-3, 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 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. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary beyond the borehole location.

In summary, the subsoil conditions at the site consist of granular fill (roadway shoulder materials) underlain by firm silty clay to clay. A more detailed description of the subsurface conditions encountered in the borehole is provided in the following sections.

4.2.1 Fill

Fill, consisting of gravelly sand containing trace to some silt, was encountered from the ground surface to a depth of 1.5 m.

The measured SPT 'N' value within the fill material was 8 blows per 0.3 m of penetration, indicating that the deposit had a loose relative density.

The natural water content of the sample of fill was 7 percent. The results of one grain size distribution test on a sample of the fill material are shown on Figure 1.

4.2.2 Silty Clay to Clay

A silty clay to clay deposit was encountered below the fill at a depth of 1.5 m below the existing ground surface, corresponding to Elevation 255.5 m. The silty clay to clay deposit extended to the bottom of the borehole at a depth of 9.8 m below ground surface, Elevation 247.2 m.

¹ Northern Ontario Engineering Geology Terrain Study, OGS Map 5027

The measured SPT 'N' values within the silty clay to clay deposit ranged from 1 to 6 blows per 0.3 m of penetration. Field 'N' vane testing carried out within the deposit yielded undrained shear strength values between 27 kPa and 46 kPa indicating the deposit had a firm consistency.

The natural water content of the silty clay to clay deposit varied between 28 percent and 61 percent. Atterberg limits testing carried out on two samples of the deposit gave liquid limits of 49 percent and 65 percent, and plastic limits of 21 percent and 26 percent, corresponding to a plasticity index of 28 percent and 39 percent. The results are shown on Figure 2 and indicate that the two samples tested are classified as silty clay of intermediate plasticity and clay of high plasticity.

The results of grain size distribution tests on three samples of the silty clay to clay deposit are shown on Figure 3.

4.2.3 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 of silty clay to clay taken in the borehole were noted to be wet. The unstabilized groundwater level observed in the open borehole was recorded at a depth of 7.5 m below the existing ground surface during drilling, corresponding to Elevation 249.5 m. Based on the results of moisture content testing on samples of the subsoils, the unstabilized groundwater level may likely be between 2 m and 3 m below ground surface, corresponding to Elevation 254 m to Elevation 255 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, the Designated MTO Contact for Golder, conducted a quality control review of the report.

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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 #12). The recommendations are based on interpretation of the factual data obtained from the borehole 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 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 11 approximately 4.5 m from the edge of pavement at Station 10+827 facing the westbound traffic. Borehole BH07-3 was advanced at Station 10+827 approximately 1.3 m from the centre of the proposed sign location. Borehole BH07-3 penetrated a 1.5 m thick layer of gravelly sand fill, underlain by a deposit of firm silty clay to clay from 1.5 m below ground surface to the bottom of the borehole at 9.8 m. The unstabilized groundwater level in the open borehole upon completion of drilling was 7.5 m below the existing ground surface (Elevation 249.5 m). Based on the results of moisture content testing on samples of the subsoils, the unstabilized groundwater level may likely be between 2 m and 3 m below ground surface, corresponding to Elevation 254 m to Elevation 255 m.

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 undrained shear strengths 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 Foundation

As indicated in Section 6.2, 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 caisson is extended 5 m below the design frost depth except where bedrock is encountered within this depth. As shown on the depth of frost penetration isopleths for Northern Ontario², the depth of frost penetration for the Matheson area is approximately 2.4 m. The typical caisson founding level would therefore be 7.4 m below the final ground surface. Bedrock was not encountered to the depth drilled at this site; therefore, the foundation for this sign support should be designed as a caisson 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 cohesive soils, the unfactored passive lateral earth pressure, P_p (kPa), distributed along the depth of the caisson foundation may be calculated using the following equation:

$$P_p = \sigma_z + 2 c_u$$

where σ_z is the total vertical stress (kPa);
 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 2.4 m below ground surface should be neglected to account for frost action.

² Ontario Provincial Standard Drawing (OPSD) 3090.100

6.2.2 Spread Footing

We understand from IBI that the preferred alternative foundation design for the support of VMS #12 is a spread footing, having a length of 5 m and a width of 2.5 m with the centre of the footing located 4.5 m from the edge of pavement. The founding depth of the spread footing should be within the firm silty clay to clay deposit and below the depth of frost penetration (i.e. 2.4 m below the final ground surface).

Based on the subsurface conditions encountered in Borehole BH07-3, we do not anticipate the footing will be constructed within an open cut adjacent to the existing roadway or along the existing backslope. Provision for protection of the existing pavement structure and existing backslope will be required in accordance with MTO's Special Provision No. 105S19, designed to meet Performance Level 2.

Relevant design parameters for the shoring are provided below.

Design Parameter	Fill
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 *	0.54

* Earth pressure coefficients for horizontal backfill.

For the cohesive soil, the passive pressure, P_p , and active pressure, P_a , acting on the shoring may be calculated using the following equations:

$$P_p = k_p \sigma_z + 2 c_u \sqrt{k_p}$$

$$P_a = k_a \sigma_z - 2 c_u \sqrt{k_a}$$

Refer to Table 1 for values of c_u specified by depth and elevation. For cohesive soil, k_a and k_p factors are equal to 1.

Where an open cut excavation will be considered, the fill material and silty clay to clay subsoils 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 fill materials and silty clay to clay should be considered Type 4 soil. For these conditions and soil types, the

walls of an excavation should be sloped from its bottom at a slope of one horizontal to one vertical or three horizontal to one vertical, respectively, or flatter.

As noted in Section 6.2, the unstabilized groundwater level in Borehole BH07-3 was encountered at a depth of 7.5 m below the existing ground surface. Based on the results of moisture content testing on samples of the subsoils, the unstabilized groundwater level may likely be between 2 m and 3 m below the existing ground surface at the borehole location, corresponding to Elevation 254 m to Elevation 255 m. For footing design at this site, the groundwater level should be assumed to be at Elevation 255 m. Depending on the seasonal time of footing construction, perched groundwater may be encountered during construction. The hydraulic conductivity of the silty clay to clay is considered low, such that the perched groundwater may be removed from the excavation by pumping from properly filtered sumps.

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.

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 prior to footing construction, to ensure that adequate preparation of the foundation area has been carried out.

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 the affected material be removed immediately prior to placing concrete. We recommend the prepared subgrade be protected using a 150 mm thick mud mat comprised of a minimum strength 5 MPa concrete placed across the bottom of the excavation immediately upon completion of the excavation and review by the QVE, to limit the disturbance and to provide a platform for construction of the spread footing.

6.2.3 Geotechnical Resistance

Spread footings constructed on the properly prepared subgrade at or below a depth of 2.4 m below final grade may be designed based on a factored geotechnical axial resistance of 150 kPa at Ultimate Limit States (ULS) for a 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 width / diameter or founding depths differ 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 taken into account in accordance with Clause 6.7.4 and C.6.7.4 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 a 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 Slope Stability and Drainage Considerations

The site of the proposed sign is located within an existing roadway cut, where the backslope is currently 3 m high and graded at 3 horizontal to 1 vertical (3H:1V). A ditch for surface and pavement structure drainage is located parallel to the roadway along the toe of the existing backslope. Based on a survey of the existing slope at the proposed sign location provided by IBI, the ditch invert is approximately 0.9 m below and approximately 5.2 m north of the westbound lane edge of pavement.

Reportedly, the proposed sign will be located 4.5 m north of the westbound lane edge of pavement and the existing shoulder will be “widened” over the proposed foundation where the existing ditch is presently located. In order to maintain drainage for the existing ditch, backslope and pavement structure, several drainage options were proposed by IBI’s highway designers. During preliminary grading design, the following options were proposed by IBI’s highway designers:

- Reroute the existing ditch through a 500 mm perforated smooth wall HDPE drainage pipe on the existing ditch alignment at the proposed sign location. The pipe would be located above the proposed sign foundation and the grade of the existing backslope would be maintained. A

shallow/narrow drainage swale would be constructed at the toe of the backslope for surface drainage; and

- Realign the existing ditch around the proposed sign, which would involve cutting back the toe of the existing backslope and regrading the backslope to 2H:1V. Over the regraded slope, a 500 mm thick drainage blanket would be installed consisting of granular fill materials. The location of the existing crest would be maintained.

After discussions with and correspondence from IBI's highway designer, we understand that the first option is the preferred alternative, while maintaining a shoulder width of 3 m. Our comments and recommendations regarding both options are presented below.

From a geotechnical perspective, the first option presented above is considered suitable because it maintains the existing 3H:1V slope configuration, which has remained stable over the long term. However, should the drainage pipe be located over the proposed sign foundation, the founding depth of the footing should be lowered such that the thickness of soil cover over the foundation subgrade should equate to the sum of the diameter of the pipe and the frost penetration depth of 2.4 m (i.e. for a 500 mm pipe, the founding depth should be 2.9 m below final grade).

Based on the existing subsurface conditions and a preliminary slope stability analysis, the proposed slope configuration for the second option results in an estimated factor of safety of less than 1.3 for the long term drained condition. In order to maintain a factor of safety of greater than 1.3 for a 2H:1V slope configuration, we recommend that the existing backslope be subexcavated/reggraded from the north foundation subgrade sloping northerly at 2.25H:1V. In order to maintain the location of the existing crest of the backslope, the drainage blanket should be constructed overtop of the backslope to a final slope of 2H:1V. Alternatively, the drainage blanket could also be sloped to a 2.25H:1V configuration up from the toe of the cut slope. This would result in the final backslope crest location being set back approximately 1.5 m and reduce the quantity of drainage blanket material required. The drainage blanket should have a minimum thickness of 500 mm at the proposed sign location.

The 2.25H:1V cut slope below the drainage blanket at the proposed sign location should taper to a 3H:1V slope configuration to match the existing slope where the realigned ditch will transition to the existing ditch location beyond the proposed sign foundation area. The drainage blanket, having a minimum thickness of 500 mm at the proposed sign location, should also taper to the existing 3H:1V backslope where the realigned ditch will transition to the existing ditch.

Prior to constructing the drainage blanket, the topsoil/organics on the slope should be stripped to expose the native subgrade. The drainage blanket should consist of approved granular material meeting MTO's Special Provision SP110S13 such as Granular 'A' or 'B' Type II. The first lift of the drainage blanket material should be thin (i.e. less than 100 mm) and "punched" into the native subgrade using a non-vibratory steel drum roller. Each following lift should be placed in 300 mm

loose lifts and uniformly compacted to a target density of 95 percent standard Proctor maximum dry density. Caution should be taken in order to minimize “pumping” the underlying cohesive subgrade by over-compaction. The QVE should review and approve the subgrade prior to placement of the drainage blanket.

Drainage of the existing pavement structure will be adequately achieved by both alternatives listed above, provided the inverts and capacities of the proposed culvert or realigned ditch are equivalent to the invert and capacity of the existing ditch along the rerouted or realigned section length and provided that the longitudinal ditch slope along the shoulder is not blinded with silty clay to clay material from the construction operations.

6.4 Construction Considerations

As indicated in Section 6.3, 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 standard Proctor maximum dry density. The use of native excavated materials as backfill is not recommended.



It is recommended 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 firm cohesive soils, which should be expected to be stable provided good construction practices are carried out. Stockpiles should not be placed such that they surcharge the excavation and/or the existing backslope. The Contractor shall be responsible to ensure 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, the 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 #12, HIGHWAY 11 WESTBOUND
W.P. 5674-04-01

Borehole No.	Stratum	Depth ¹ (m)	Elevation (m)	Groundwater Elevation ² (m)	Design Parameters ³			
					C_u	γ	γ'	ϕ'
BH07-3	Loose Gravelly Sand Fill	0.0 – 1.5	257.0 to 255.5	255.0 (Assumed)	-	18	8	27
	Firm Silty Clay to Clay	1.5 – 7.0	255.5 to 250.0		25	17	7	-
	Firm Silty Clay to Clay	7.0 – 9.8	250.0 to 247.2		40	17	7	-

- NOTES:**
1. Depths are given below the ground surface at the borehole location.
 2. Possible range of stabilized groundwater elevation at the time of drilling based on moisture contents of soil samples.
 3. Design parameters: C_u = undrained shear strength (kPa);
 γ = bulk unit weight (kN/m³); and
 γ' = effective unit weight below the groundwater level (kN/m³).
 ϕ' = effective friction angle (degrees)

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 07-1191-0039-12			RECORD OF BOREHOLE No BH07-3				1 OF 1 METRIC							
W.P. 5764-04-01		LOCATION N 5377800.2 ; E 339185.0		ORIGINATED BY ID										
DIST _____ HWY 11		BOREHOLE TYPE Power Auger, 108mm ID, Hollow Stem Augers		COMPILED BY BB										
DATUM Geodetic		DATE November 15, 2007		CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
257.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.0	Gravelly Sand, trace to some silt (FILL) Loose Brown Moist		1	SS	8		256							32 56 (12)
255.5	SILTY CLAY to CLAY		2	SS	6		255							0 3 42 55
1.5	Firm Grey Wet		3	SS	2		254							
			4	SS	2		253	X +						1 9 69 21
			5	SS	2		252	X +						
			6	SS	1		251	X +						
			7	SS	2		250	X +						0 1 30 69
			8	SS	2		248	X +						
247.2	End of Borehole													
9.8	Notes: 1. Water level at a depth of 7.5m (Elev. 249.5m) upon completion of drilling.													

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

CONT No.
WP No. 5674-04-01

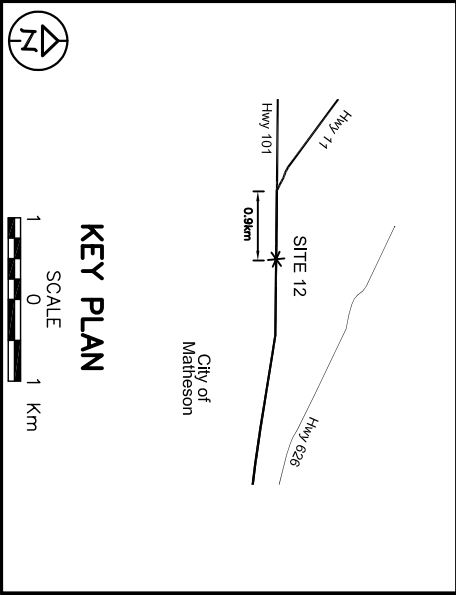


VARIABLE MESSAGE SIGN #12
HIGHWAY 11 WESTBOUND, MATHESON
BOREHOLE LOCATION

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



LEGEND			
	Borehole Location		
No.	ELEVATION(m)	CO-ORDINATES	
		NORTHING	EASTING
BH07-3	257.0	5377800.2	339185.0

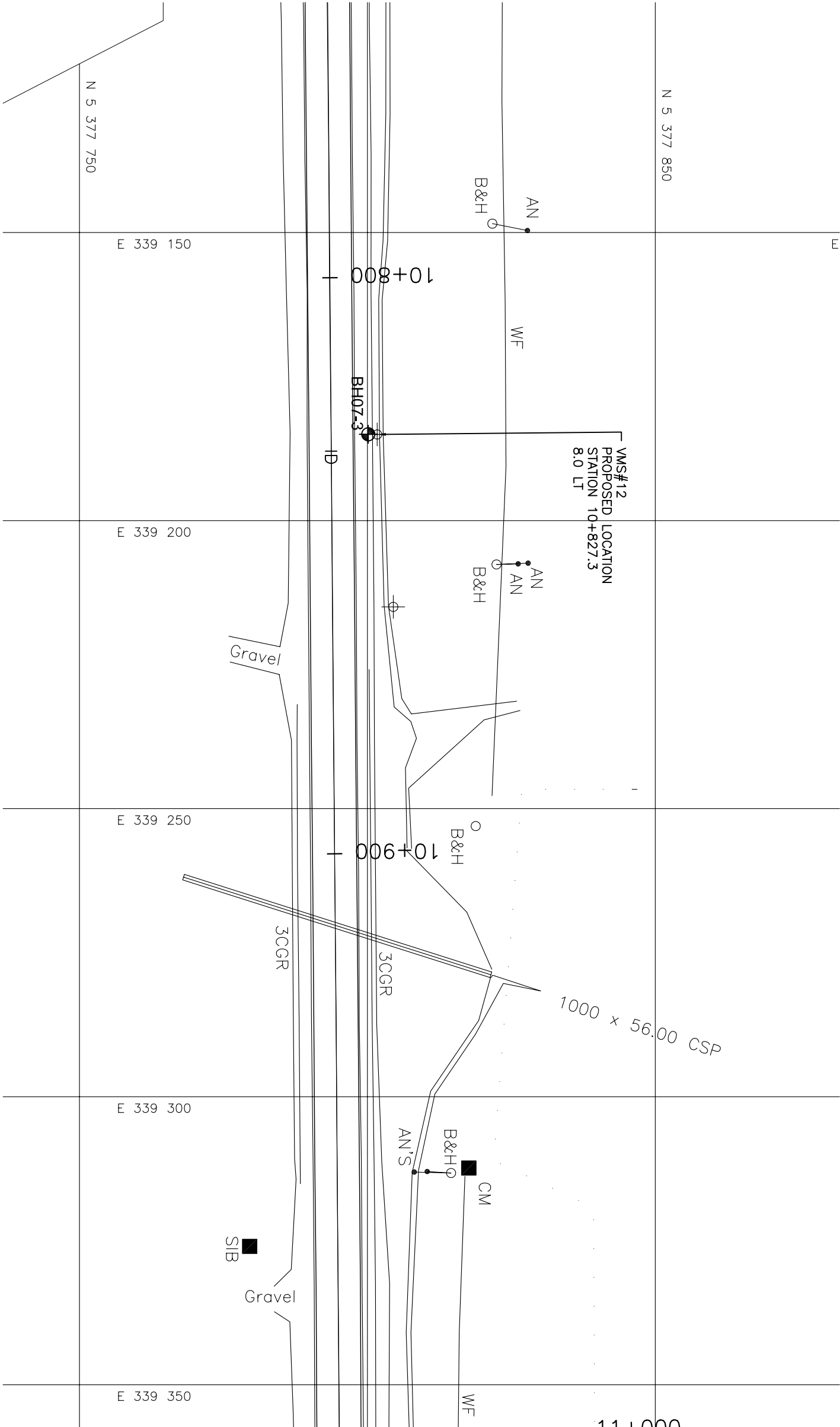
NOTES

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 O-P-S General Conditions.

REFERENCE

Base plans provided in digital format by IBI, drawing file no. kplnns.dwg and SBO7029.dwg, received January, 2007.



PLAN

SCALE

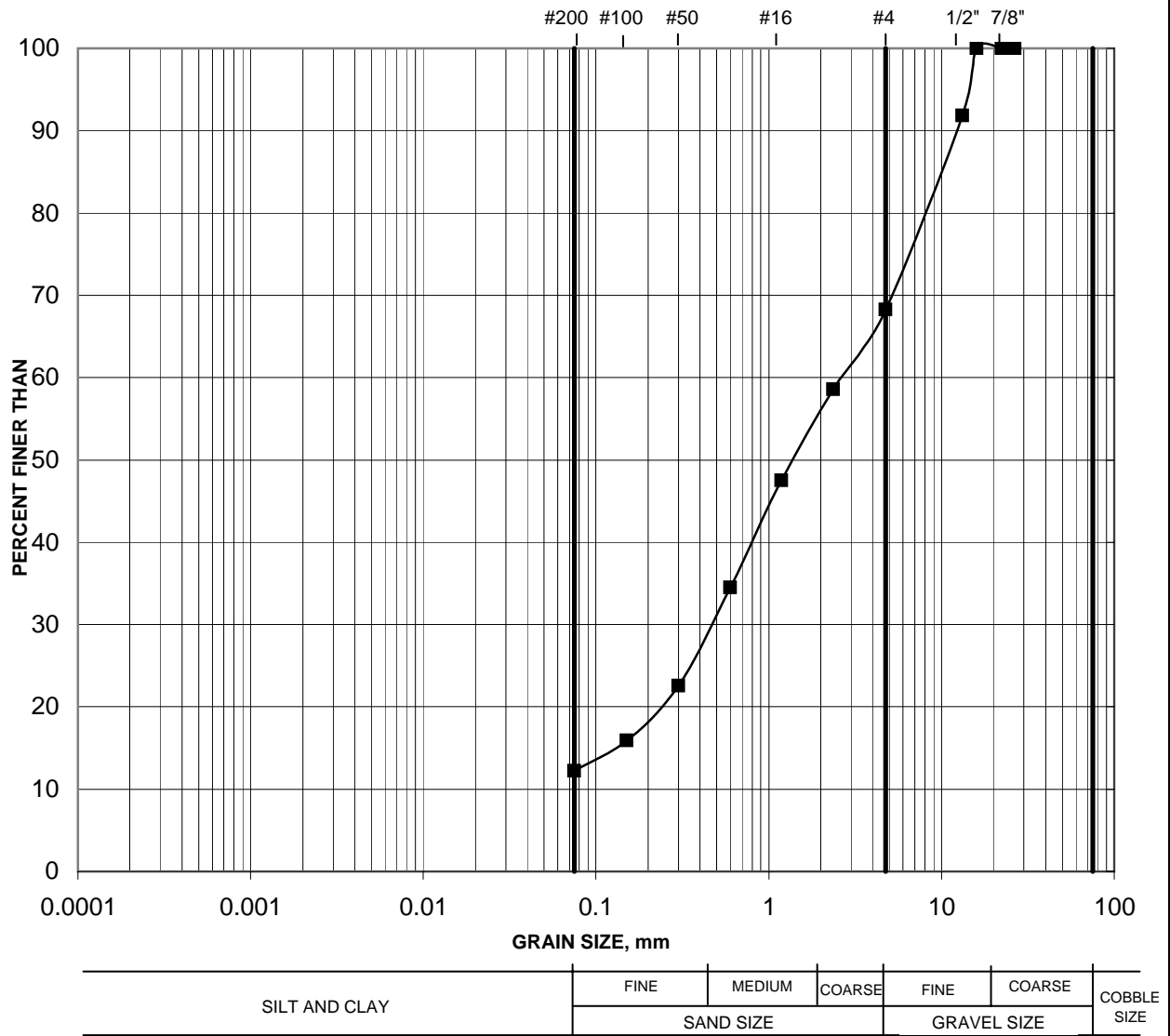


NO.	DATE	BY		REVISION	
Geocres No. 42A-70					
Hwy. 11			PROJECT NO. 07-1191-0039-12	DIST.	
SUBWD. AB	CHKD. JMAC		DATE: JUNE 2008	SITE:	
DRAWN. BB	CHKD. AB		APPD. JMAC	DWG. 1	

GRAIN SIZE DISTRIBUTION

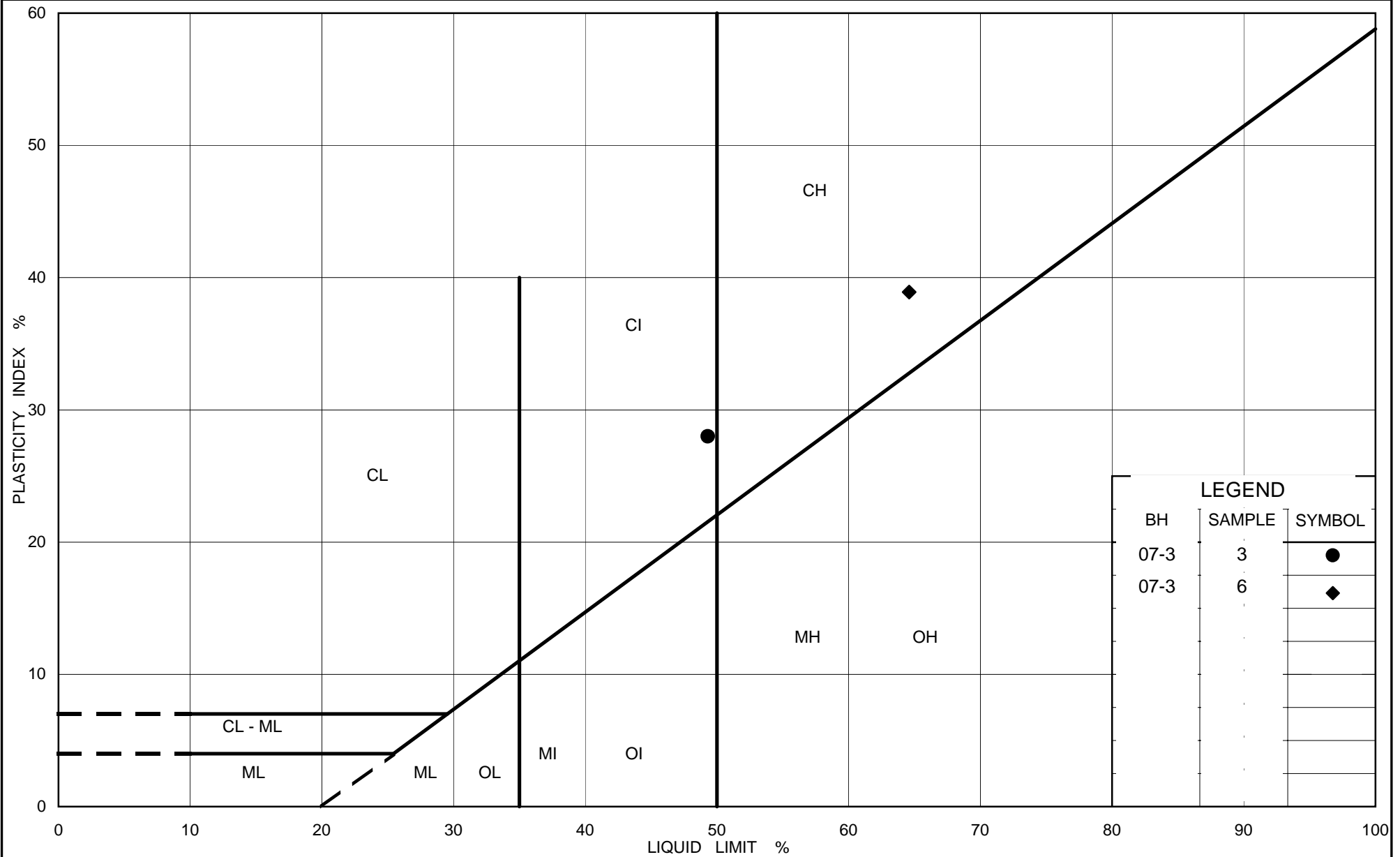
Gravelly Sand (FILL)

FIGURE 1



Borehole	Sample	Elevation (m)
07-3	1	255.9

Project: 07-1191-0039-12



Ministry of Transportation

Ontario

PLASTICITY CHART SILTY CLAY TO CLAY

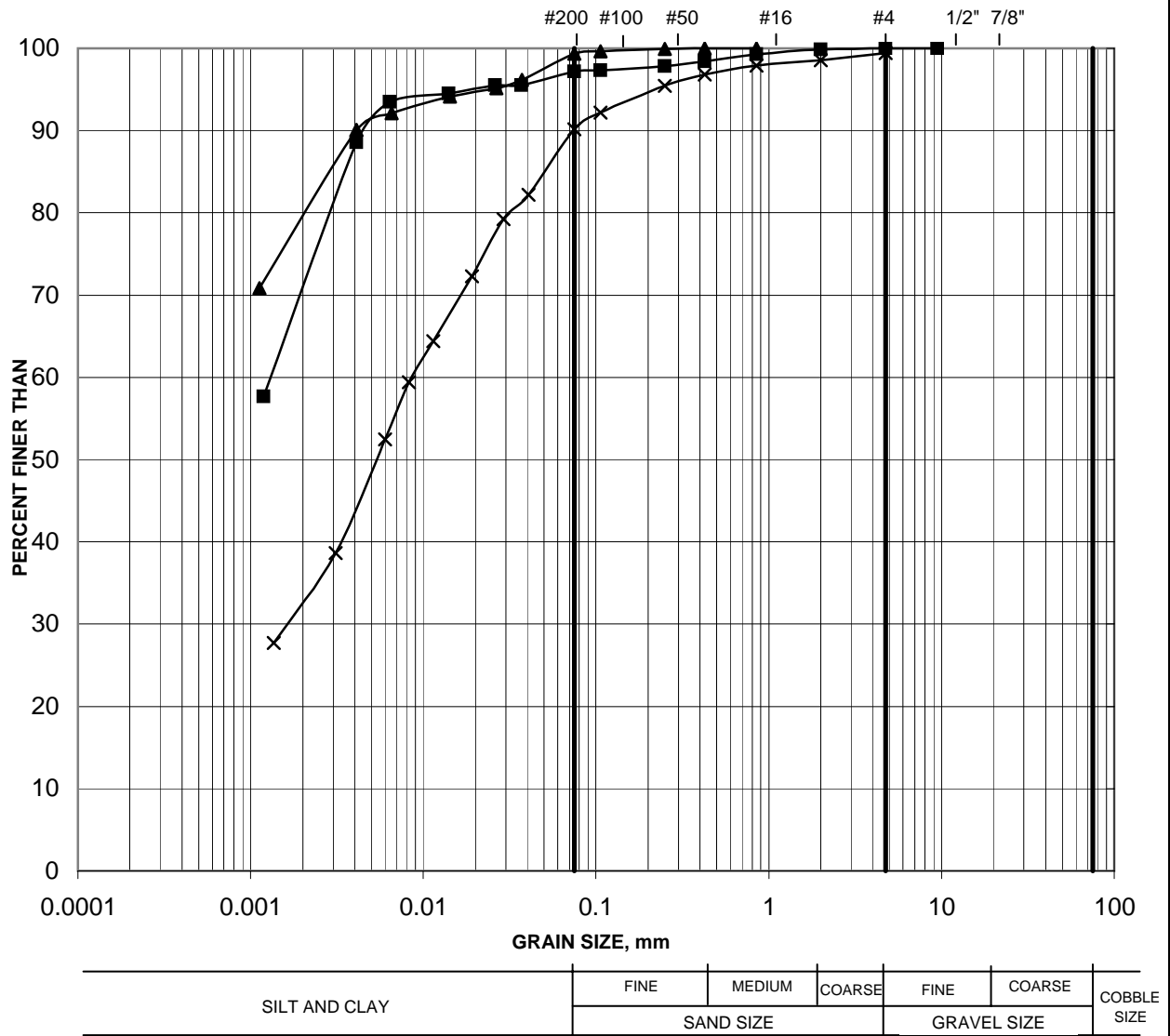
FIGURE No. 2

Project No. 07-1191-0039-12

Checked By: JMAC

GRAIN SIZE DISTRIBUTION SILTY CLAY TO CLAY

FIGURE 3



	Borehole	Sample	Elevation (m)
—■—	07-3	2	255.2
—×—	07-3	4	253.7
—▲—	07-3	7	249.1

Project: 07-1191-0039-12

APPENDIX A
NON-STANDARD SPECIAL PROVISION

CONTROL OF OVERBURDEN AND SUBGRADE SOILS FOR FOUNDATION EXCAVATIONS - Item No.

Non-Standard Special Provision

This special provision is to highlight the construction concerns for the installation of VMS#12 (Matheson). The Contractor shall be alerted that the foundation soils at the sign location consist of firm cohesive materials, which are susceptible to slope instability during foundation excavation as well as at the existing backslope). Stockpiles should be placed such that they do not surcharge excavations or the existing backslope.

In addition, disturbance of the subgrade during construction of the spread footing could influence the settlement of the structure. Caution should be carried out during excavation to minimize disturbance of the subgrade.

The contractor is responsible to ensure that appropriate construction procedures and equipment are used for construction.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the Work.

END OF SECTION