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**FOUNDATION INVESTIGATION
AND DESIGN REPORT
CULVERT 18
HIGHWAY 7 TWINNING FROM 0.7 KM WEST
OF JINKINSON ROAD WESTERLY 10.5 KM TO
2.5 KM WEST OF ASHTON STATION ROAD
W.P. 251-99-00**

Submitted to:

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GEOCRES No. 31F-158

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PART A

FOUNDATION INVESTIGATION REPORT

CULVERT 18

**HIGHWAY 7 TWINNING FROM 0.7 KM WEST OF JINKINSON ROAD
WESTERLY 10.5 KM TO 2.5 KM WEST OF ASHTON STATION ROAD
W.P. 251-99-00**

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in the former West Carleton Township which is now part of the City of Ottawa, and in Beckwith Township in Lanark County. The section of Highway 7 included in this assignment (W.P. 251-99-00) extends from 0.7 km west of Jinkinson Road westerly for 10.5 km, to 2.5 km west of Ashton Station Road, and includes service roads to accommodate future construction on Highway 7.

This report addresses the proposed new Culvert 18, to be located on the new North Service Road at approximately Station 11+700, to permit the North Service Road to cross over a tributary to Lavallee Creek.

2.0 SITE DESCRIPTION

The proposed Culvert 18 site, at the crossing of the new North Service Road over the Lavallee Creek tributary for the North Service Road, is located approximately 1.7 km east of Carleton Place and 180 m east of Lavallee Creek, in Beckwith Township in Lanark County. The North Service Road is to be located approximately 20 m south of the existing Trans-Canada Trail (former rail right-of-way) at the proposed tributary crossing location.

At the Culvert 18 site, the Lavallee Creek tributary is oriented approximately north-south, and flows in a northerly direction. Immediately west and east of the creek, the topography consists of relatively flat farm land that slopes gently downward toward the creek flood plain. The ground surface within the creek flood plain is at about Elevation 125.5 m, and the creek bed at the culvert site is at approximately Elevation 125.1 m. The creek flood plain consists of grass-covered marsh land; frozen standing water was observed throughout the flood plain at the time of the investigation in February 2007. The creek was also frozen at the time of investigation, with about 0.3 m of ice and water present above the creek bed.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out between February 13 and 15, 2007. Two boreholes (Boreholes 07-1 and 07-2) were drilled at the locations shown on Drawing 1. The boreholes were located as close as possible to the proposed culvert location given the flooding and partial freezing conditions at the time of the investigation: Borehole 07-1 was located outside of the creek flood plain on the creek bank immediately west of the proposed culvert location, while Borehole 07-2 was located within the creek floodplain adjacent to the southeast end of the proposed culvert. These two boreholes were supplemented with a hand augerhole (Borehole 07-3) located within the flooded area at the centre of the proposed culvert alignment, and advanced using a combination of hand auger and ice auger drilling equipment; this borehole was advanced to assess the presence of organic/alluvial soils at the creek bed.

Boreholes 07-1 and 07-2 were advanced using a track-mounted drill rig, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. These boreholes were advanced to practical refusal to augering at depths of 7.7 m and 7.8 m below the existing ground/ice surface; Borehole 07-1 was advanced a further 2.3 m into the bedrock using NQ-size coring equipment. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In situ vane testing was carried out within the cohesive deposits using an N-sized vane. Relatively undisturbed, 75-millimetre diameter thin-walled Shelby tube (ASTM D1587) samples of the silty clay were retrieved using a fixed piston sampler. The boreholes were sealed at their base using bentonite pellets in accordance with Ontario Regulation 903, then backfilled with the native clayey soils, and the site conditions were restored following completion of the work.

Borehole 07-3 was advanced to a depth of 1.8 m below the ice surface. The soils recovered on the augers were classified by visual and tactile examination.

The field work was supervised on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's laboratories in Ottawa and Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples at the Ottawa laboratory. An oedometer (consolidation) test was carried out on one sample of the clay deposit in the Mississauga laboratory.

The borehole locations were determined by Golder relative to existing site features. The borehole ground surface elevations were also determined by Golder and referenced to the elevation of boreholes from Golder's previous investigation for the North Service Road bridge over Lavallee Creek, to the west of this site. The borehole locations, including MTM NAD83 northing and

easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

<i>Borehole Number</i>	<i>Borehole Location</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
07-1	West of culvert alignment	5,000,924.7	335,572.7	126.2
07-2	East of culvert alignment	5,000,928.1	335,605.8	125.8
07-3	Centre of culvert alignment	5,000,928.2	335,596.6	125.7

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the Smith Falls Limestone Plain, as delineated in *The Physiography of Southern Ontario*¹, that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Smiths Falls Limestone Plain is characterized by shallow overburden deposits overlying sedimentary bedrock consisting of limestones, dolostones, sandstones and shales. The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. In the vicinity of Carleton Place, clay has been deposited within depressions in the bedrock that have been caused by faulting. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock.¹

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, two boreholes and one hand augerhole were advanced in the area of the proposed Culvert 18. The borehole locations, ground surface elevations and an interpreted stratigraphic profile are shown on Drawing 1.

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 6. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In general, the soils encountered at the site consist of topsoil and surficial organic-containing deposits of silty clay and clayey with thicknesses of about 0.9 m to 1.1 m, over an approximately 6.1 m to 7.2 m thick deposit of clay. The upper 1.5 m to 2.7 m of the clay deposit has been weathered to a grey-brown crust, while the underlying, unweathered portion of the deposit is grey in colour. Below a depth of about 7.3 m to 7.5 m, the silty clay is underlain by about 0.3 m of sandy silt till. The till is, in turn, underlain by dolomitic limestone bedrock that was encountered between about 7.7 m and 7.8 m depth (at approximately Elevations 118.2 m to 118.4 m, respectively).

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

¹ Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

4.2.1 Topsoil

About 300 mm of topsoil was encountered immediately below the ground surface in Borehole 07-1, which is located on the creek bank just west of the proposed culvert location.

4.2.2 Organic-Containing Clayey Silt to Silty Clay

A surficial deposit consisting of clayey silt and/or silty clay containing organic matter was encountered immediately below the creek bed/flood plain in Boreholes 07-2 and 07-3. Traces of shells were also noted in this deposit in Borehole 07-3. The deposit is about 0.9 m to 1.1 m in thickness, with its base encountered at Elevations 124.4 m and 124.6 m. The organic content of one sample of the alluvium was measured to be 4 per cent.

4.2.3 Clay

The topsoil and surficial clayey silt to silty clay, where present, are underlain by a clay deposit that is between 6.1 m and 7.2 m in thickness as encountered in Boreholes 07-2 and 07-1, respectively. The surface of the clay deposit was encountered between Elevations 124.4 m and 124.6 m within the creek channel and flood plain (Boreholes 07-2 and 07-3), and at about Elevation 125.9 m in Borehole 07-1 on the west bank of the creek.

At Boreholes 07-1 and 07-2, the upper 1.5 m to 2.7 m of the clay deposit has been weathered to a grey-brown crust. The measured SPT “N” values in this portion of the deposit ranged from 5 to 10 blows per 0.3 m of penetration, while in situ vane testing measured undrained shear strengths of greater than 96 kPa; these test results indicate that the weathered clay crust has a very stiff consistency. The result of a grain size distribution test on one sample of the weathered clay crust is shown on Figure 1. Atterberg limit testing on two selected samples of the weathered clay crust measured plastic limits of 23 and 26 per cent, liquid limits of 51 and 55 per cent, and plasticity indices of 28 and 29 per cent. These results, which are summarized on a plasticity chart on Figure 2, confirm that this material is generally a clay of high plasticity.

The clay below the depth of weathering is grey. The measured SPT “N” values within the unweathered grey clay range from 5 to 9 blows per 0.3 m of penetration, while in situ vane testing carried out in Boreholes 07-1 and 07-2 measured undrained shear strengths of greater than 96 kPa. These results indicate that the unweathered grey clay has a very stiff consistency. The result of a grain size distribution test on one sample of the unweathered clay is shown on Figure 3. Atterberg limit testing was carried out on five selected samples of the grey clay measured plastic limits of 21 to 24 per cent, liquid limits of 49 to 57 per cent, and plasticity indices of 28 to 34 per cent. These results, which are plotted on a plasticity chart on Figure 4, confirm that this material is a clay of high plasticity.

Oedometer consolidation testing was carried out on one sample of the grey clay obtained just below the depth of weathering. The results of that testing, which are provided on Figures 5A to 5D and summarized in the table below, indicate that this material is overconsolidated, with a preconsolidation pressure about 430 kPa and an overconsolidation ratio of approximately 12 in the upper portion of the unweathered material.

Borehole/ Sample Number	Sample Depth/Elev. (m)	Unit Weight (kN/m ³)	σ_p' (kPa)	σ_{vo}' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e _o	OCR	Cv
07-2 / 3	3.1 / 122.7	17.2	430	35	395	0.41	0.055	0.96	12	0.02

Notes:

- σ_p' - Apparent preconsolidation pressure
- σ_{vo}' - Computed existing vertical effective stress
- Cc - Compression index
- Cr - Recompression index
- e_o - Initial void ratio
- OCR - Overconsolidation ratio
- Cv - Coefficient of consolidation

4.2.4 Sandy Silt Till

A 0.1 m to 0.3 m thick layer of glacial till was encountered below the clay deposit in Boreholes 07-1 and 07-2. The surface of this till deposit was encountered between Elevations 118.5 m and 118.7 m in these boreholes.

Based on local experience and observations of the drilling resistance, the glacial till consists of a heterogeneous mixture of gravel and cobbles in a matrix of sandy silt, containing trace clay; the result of a grain size distribution test conducted on one sample of the sandy silt till is shown on Figure 6. Due to the limited thickness of this deposit at the site, only limited standard penetration testing could be carried out before sampler refusal was encountered on the bedrock surface.

4.2.5 Dolomitic Limestone Bedrock

Dolomitic bedrock, containing interbeds of sandstone, underlies the till deposit at this site, with the surface of the bedrock encountered between Elevations 118.2 m and 118.4 m. The bedrock was confirmed by coring for 2.3 m in Borehole 07-1.

The dolomitic limestone bedrock at the site is fresh, medium strong, and very thinly to medium bedded. The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples were approximately 62 and 73 percent, indicating that the bedrock is of fair quality. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes, although minor sub-vertical jointing was also observed. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions


The water level in Borehole 07-1, located on the bank at the edge of the floodplain, was at a depth of 0.7 m (Elevation 125.5 m) during drilling. The floodplain was submerged at the time of the investigation, with the ice level at about Elevation 125.8 m.

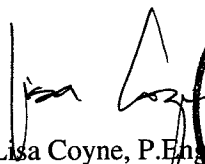
The groundwater and surface water levels are expected to fluctuate seasonally, and are expected to be higher during wet periods of the year.

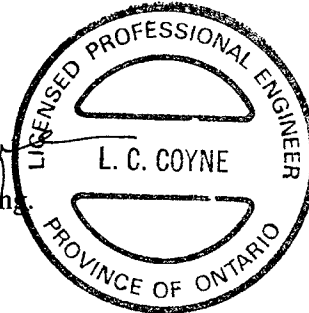
5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Susan Trickey, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

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PART B

FOUNDATION DESIGN REPORT

CULVERT 18

**HIGHWAY 7 TWINNING FROM 0.7 KM WEST OF JINKINSON ROAD
WESTERLY 10.5 KM TO 2.5 KM WEST OF ASHTON STATION ROAD
W.P. 251-99-00**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed Culvert 18, to carry the Lavallee Creek tributary under the new North Service Road. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives for the design of the proposed culvert. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. 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 Culvert Foundations

The new Culvert 18 could consist of a box culvert or a cast-in-place, rigid frame open footing culvert. Both pre-cast and cast-in-place culvert structures are appropriate for this site, though the use of pre-cast culvert sections (either open footing or box culvert) would be advantageous in minimizing the construction time and, therefore, the time during which surface water control will be required.

A rigid frame open footing culvert, and any associated wing walls/retaining walls, can be founded on spread footings extended below the topsoil and surficial, organic-containing clayey silt to silty clay, to be supported on the very stiff, weathered, grey-brown clay crust. A box culvert, if adopted, can also be founded below the topsoil and surficial, organic-containing clayey silt to silty clay, on the very stiff, weathered, grey-brown clay crust.

Deep foundations are not required for Culvert 18, as shallow foundations will provide sufficient bearing resistance for the new culvert and satisfactory settlement performance under the approximately 2.7 m high embankment loading. Therefore, detailed design guidelines are not provided for deep foundations since, although feasible, deep foundations would not be economical.

6.2.1 Founding Elevation

If a box culvert is adopted for Culvert 18, it is recommended that the surficial, organic-containing clayey silt to silty clay be removed from under the footprint of the new culvert. The design subgrade level for a box culvert at this site should be taken as Elevation 124.3 m. The box culvert should be provided with a minimum of 400 mm of OPSS Granular "A" bedding;

additional Granular “A” should be placed according to the depth of subexcavation required below the design invert level.

Spread footings for an open footing culvert and any associated wing walls/retaining walls should be constructed at or below Elevation 124.0 m, in order to be founded below the surficial organic-containing clayey silt to silty clay, on the very stiff, weathered clay crust. In addition, the spread footings should be founded at a minimum depth of 1.8 m below the lowest surrounding grade, to provide adequate protection against frost penetration.

6.2.2 Geotechnical Resistance

A box culvert or strip footings placed on the properly prepared subgrade at or below the founding elevations given above should be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 350 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 250 kPa. This SLS resistance value for the culvert foundations is based on 25 mm of settlement, and takes into account the settlement that would occur due to the loading from the new Culvert 18 and the new North Service Road embankment fill overlying the culvert. Considering the overconsolidated nature of the clay, the total ground settlement resulting from compression of the clay under the combined loading from the weight of the 2.7 m high embankment and the culvert foundations will be less than 10 mm to 15 mm.

These geotechnical resistances 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 the *Canadian Highway Bridge Design Code (CHBDC)*.

6.2.3 Resistance to Lateral Loads / Sliding Resistance

The resistance to lateral forces/sliding resistance between the concrete of the culvert and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*.

If a pre-cast concrete box culvert is adopted, it will be constructed on a granular pad/granular bedding on top of the very stiff, weathered clay crust. For this case, the coefficient of friction, $\tan \delta$, between the concrete and the Granular “A” pad can be taken as 0.6, and the coefficient of friction between the Granular “A” and the very stiff, weathered clay crust (for long-term conditions) may be taken as 0.45. The short-term condition for the granular-clay interface should also be checked using $\tan \phi' = 0$ and c' equal to the undrained shear strength of the clay crust, which may be taken as 75 kPa (which takes account of reduction in the shear strength of the soil mass due to fissuring).

If an open footing culvert is adopted, the coefficient of friction, $\tan \phi'$, between cast-in-place concrete footings and the very stiff clay subgrade should be taken as 0.45 for long-term

conditions. The short-term condition should also be checked using $\tan \phi' = 0$ and c' equal to the undrained shear strength of the clay crust, which may be taken as 75 kPa (which takes account of reduction in the shear strength of the soil mass due to fissuring).

These values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3 Culvert Backfill and Erosion Protection

The bedding, backfill and levelling pad requirements for the culverts should be in accordance with OPSD 803.010.

Backfill to the culvert walls and any associated retaining walls should consist of granular fill meeting the requirements of OPSS Granular “A” or Granular “B” Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO’s Special Provision SP105S10.

Backfill above the culvert could also consist of OPSS Granular “A” or Granular “B” Type II fill, although earth fill may also be used for the new embankment construction. The culvert should be designed for the full overburden pressure and live load, assuming an embankment fill unit weight of 22 kN/m³ for Granular “A”, 21 kN/m³ for Granular “B” Type II, or 20 kN/m³ for earth fill.

If the creek flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) for the new culvert. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205, and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level.

6.4 Lateral Earth Pressures for Design

The lateral earth pressures acting on the culvert replacement walls and headwalls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

The following recommendations are made concerning the design of the walls. As discussed in Section 6.3, these recommendations assume that the backfill to the culvert walls consists of free-draining granular fill meeting the requirements of OPSS Granular “A” or Granular “B” Type II,

placed and compacted in accordance with MTO's Special Provision SP105S10, with longitudinal drains and weep holes installed as necessary to provide positive drainage of the granular backfill.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(I) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(I) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the use of granular fill and the following parameters (unfactored) may be assumed:

	Granular "A"	Granular "B" Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- If the wall support and superstructure allow lateral yielding, active earth pressures should be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the *CHBDC*, this site is located in Seismic Performance Zone 3. The site-specific zonal acceleration ratio for Carleton Place is 0.2. Based on experience, for the subsurface conditions at this site, no significant amplification of the ground motion will occur. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.2$.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding (i.e. the culvert walls), the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.3$). For structures which allow lateral yielding (i.e. any wing walls or retaining walls associated with the new culvert), k_h is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.1$).

The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	<i>Case I</i>	<i>Case II</i>	
		<i>Granular A</i>	<i>Granular B Type II</i>
Yielding wall	0.39	0.30	0.30
Non-yielding wall	0.62	0.50	0.50

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.20. This corresponds to displacements of up to approximately 50 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d)$$

where $\sigma_h(d)$ is the lateral earth pressure (kPa) at depth, d;
 K_a is the static active earth pressure coefficient;
 K_{AE} is the seismic active earth pressure coefficient;
 γ is the unit weight of the backfill soil (kN/m^3),
as given previously;
d is the depth below the top of the wall (m); and
H is the total height of the wall (m).

6.5 Construction Considerations

6.5.1 Groundwater and Surface Water Control

Control of the creek water will be necessary at the culvert site, in order for foundation construction to be carried out in dry conditions. Depending on the creek flow at the time of construction, the flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam. Assuming that the cofferdam and/or temporary bypass are effective, any seepage into the excavation during normal creek flow conditions should be adequately controlled by pumping from properly filtered sumps.

Surface water should be directed away from the excavation area, to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade; further discussion on this aspect is provided in Section 6.5.3.

6.5.2 Excavations

Temporary excavations for the culvert will extend through the existing topsoil, surficial organic-containing clayey silt to silty clay, and into the very stiff weathered clay crust. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects, and MTO Special Provision SP 902S01 (Excavation and Backfilling to Structures). The overburden materials at both culvert locations are classified as Type 3 soil, according to the OHSA. Where space permits, temporary open-cut excavations through these materials should be made with side slopes formed no steeper than 1 horizontal to 1 vertical (1H:1V).

It is not anticipated that temporary excavation support will be required as part of the new Culvert 18 and North Service Road construction at this site, since there is sufficient room to carry out open-cut excavations. However, if temporary excavation support is adopted, they should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of the temporary shoring systems should meet Performance Level 2 as specified in SP105S19, provided that any utilities that may be present adjacent to the temporary shoring system can tolerate this level of deformation.


6.5.3 Subgrade Protection

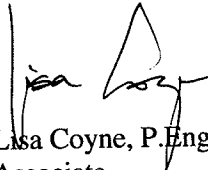
The clay that is exposed at the founding/subgrade level will be susceptible to disturbance from construction traffic and ponded water. In order to limit this degradation, it is recommended that a working mat of lean concrete be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed either with a note on the General Arrangement drawing, or with a Non-Standard Special Provision (NSSP).

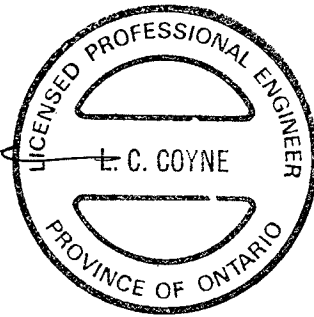
7.0 CLOSURE

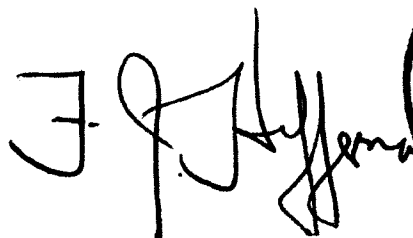

This Foundation Design Report was prepared by Ms. Susan Trickey, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

GOLDER ASSOCIATES LTD.


for: Susan Trickey, EIT
Geotechnical Group


Lisa Coyne, P.Eng.
Associate




A circular professional seal for F. A. Heffernan. The outer ring contains the text "REGISTERED PROFESSIONAL ENGINEER" at the top and "PROVINCE OF ONTARIO" at the bottom. The center of the seal features a stylized "E" shape with the name "F. A. HEFFERNAN" printed across it.

Fintan J. Heffernan, P.Eng.
Designated MTO Foundations Contact

SAT/LCC/FJH/sat/lcc

N:\ACTIVE\2004\1111\04-1111-007 MMM HWY 76 - REPORTS\FINAL REPORTS\04-1111-007 RPT04 07NOV CULVERT 18.DOC

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

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

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.)

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

c_u, s_u

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

Piezo-Cone Penetration Test (CPT)

A 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.

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. in 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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains > 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

RECORD OF BOREHOLE No 07-1

1 OF 1 **METRIC**

PROJECT 04-1111-007-6000

W.P. 251-99-00

LOCATION N 5000924.7; E 335572.7

ORIGINATED BY P.A.H.

DIST HWY 7

BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem

COMPILED BY J.M.

DATUM Geodetic

DATE Feb. 13/14, 2007

CHECKED BY S.A.T.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED	WATER CONTENT (%)					
126.2	GROUND SURFACE													
126.0	TOPSOIL													
0.3	CLAY (Weathered Crust) Very stiff Grey brown Moist to wet		1	SS	7									
			2	SS	10									
			3	SS	7									
123.1	CLAY Very stiff Grey Wet		4	SS	7									
3.1			5	SS	9									
			6	TP	PH									
118.7	Sandy SILT, some gravel, trace to some clay (TILL) Grey Wet		7	SS	>100									
118.4	Dolomitic limestone (BEDROCK) Fresh Very thinly to medium bedded Grey Medium strong		8	NQ RC	REC 100%									
7.8	Bedrock cored between 7.8m 10.15m depth. For bedrock coring details refer to Record of Drillhole 07-1.		9	NQ RC	REC 100%									
116.0	End of Borehole													
10.2	Note: Water level in open borehole at a depth of 0.7 m (Elevation 125.5 m) on completion of deilling.													

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

MIS-MTO 001 04-1111-007-6000 GPJ GAL-MISS.GDT 5/8/07

LOCATION: N 5000924.7: E 335572.7

DRILLING DATE: Feb. 13/14, 2007

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

[illegible]

DEPTH SCALE

1 : 75



LOGGED: P.A.H.

CHECKED: S.A.T.

RECORD OF BOREHOLE No 07-2

1 OF 1 **METRIC**

PROJECT 04-1111-007-6000

W.P. 251-99-00

LOCATION N 5000928.1; E 335605.8

ORIGINATED BY P.A.H.

DIST HWY 7

BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem

COMPILED BY J.M.

DATUM Geodetic

DATE

CHECKED BY S.A.T.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
125.8	GROUND SURFACE													
0.0	ICE													
0.4	WATER													
124.6	Clayey silt and silty clay, some organic matter (ALLUVIUM) Grey brown to dark grey Wet		1	SS	1		125							
1.2	CLAY (Weathered Crust) Very stiff Grey brown Wet		2	SS	5		124							0 0 33 67
123.1							123							
2.7	CLAY Very stiff Grey Wet		3	TP	PH		122							
							121							
			4	SS	5		120							
							119							
118.5														
118.2	Sandy SILT, some gravel, trace to some clay (TILL) Grey Wet		6	SS	>100									
7.7	End of Borehole Auger Refusal Probable Bedrock													

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 07-3

1 OF 1 **METRIC**

PROJECT 04-1111-007-6000

W.P. 251-99-00

LOCATION N 5000928.2; E 335596.6

ORIGINATED BY P.A.H.

DIST HWY 7

BOREHOLE TYPE Hand Auger

COMPILED BY J.M.

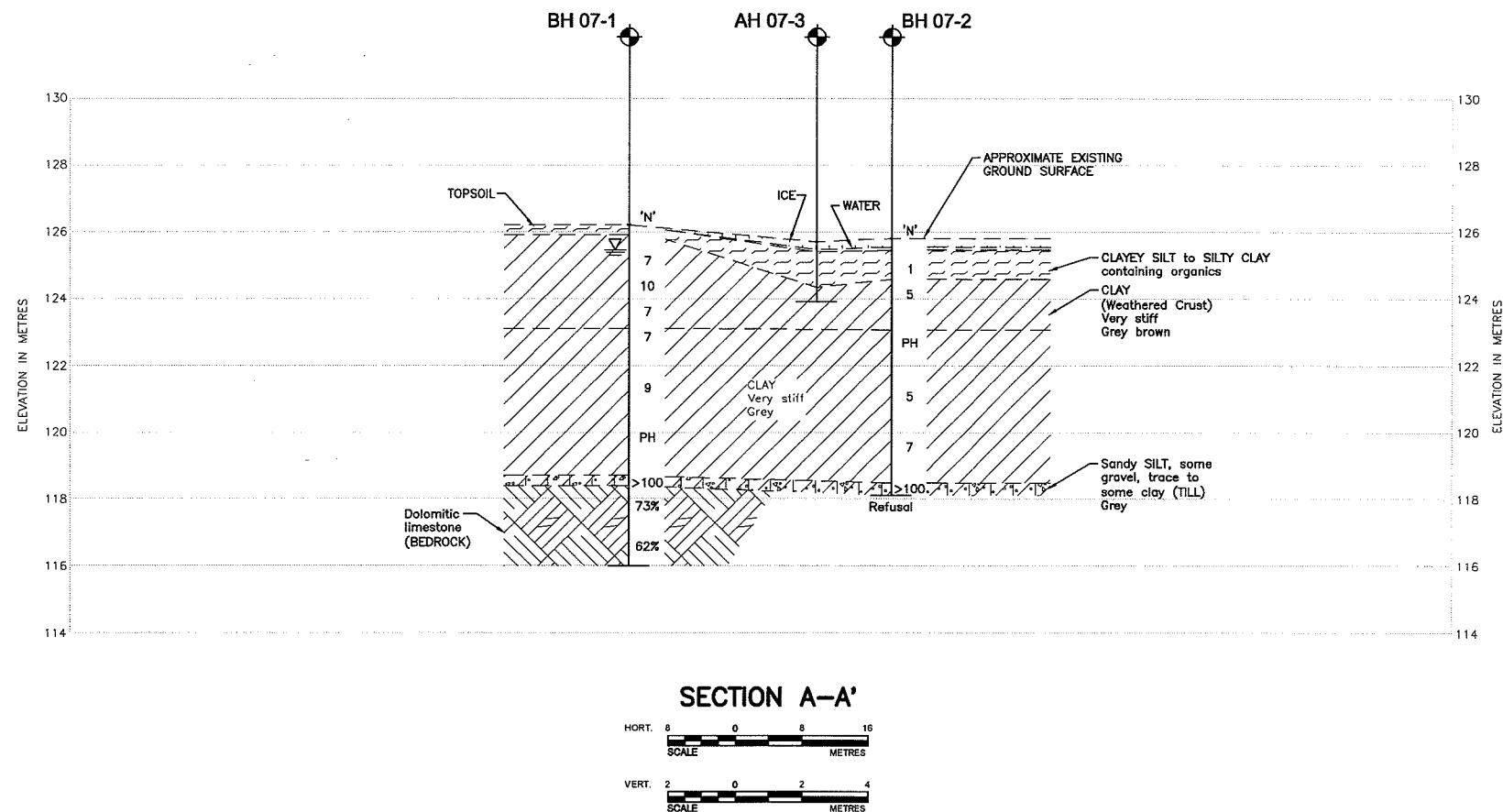
DATUM Geodetic

DATE _____

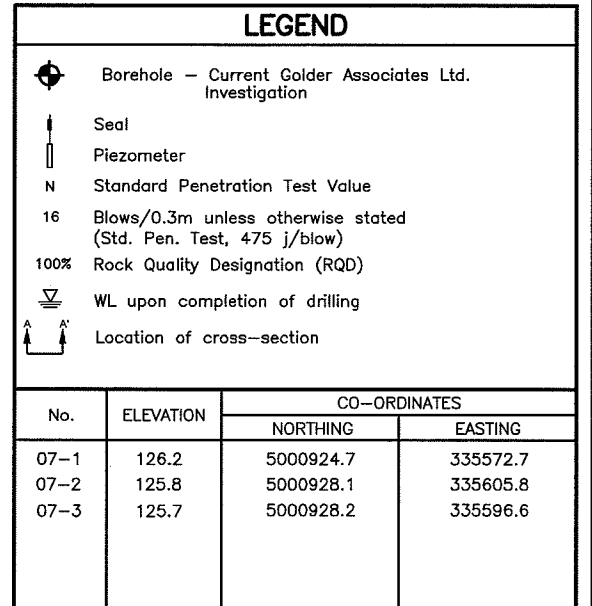
CHECKED BY S.A.T.

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										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED												
125.7	GROUND SURFACE							20	40	60	80	100	25	50	75					
0.0	ICE																			
	WATER																			
0.5	ORGANIC Material Silty clay, some organic matter, trace shells (ALLUVIUM) Dark grey Wet		1	A.S.	-		125													
124.4	CLAY																			
1.4	Very stiff																			
123.9	Grey-brown						124													
1.8	End of Augerhole																			
												</								

+³, X³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE



SHEET



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

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 OPS General Conditions.

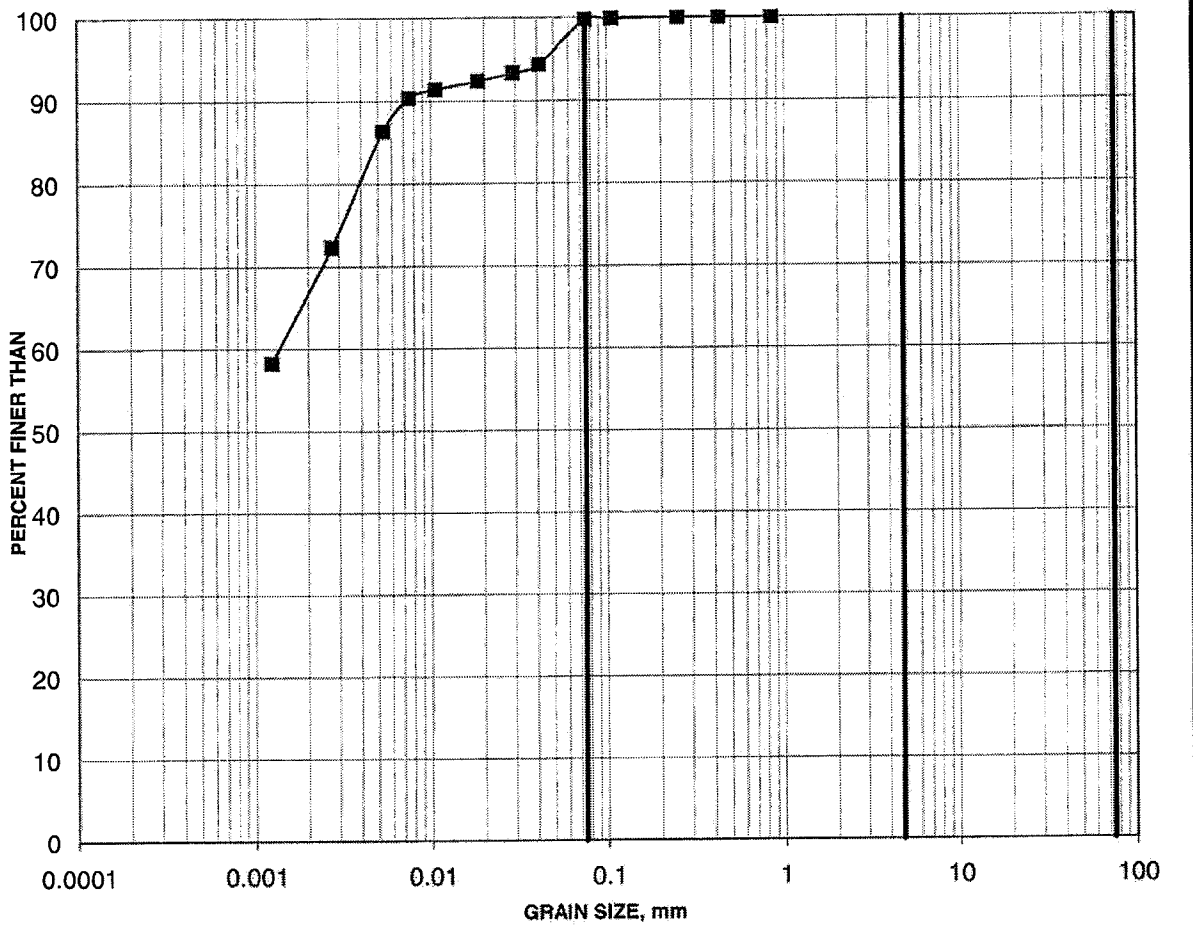
Electronic general arrangement file provided by Marshall Macklin Monaghan
on July 11, 2005.

NO.	DATE	BY	REVISION	
Geocres No.				
HWY. 7		PROJECT NO. 04-1111-007		SITE.
SUBM'D. SAT	CHKD. LCC	DATE: MARCH 2007		SITE:
DRAWN: JM	CHKD. SAT	APPD. LCC	DWG. 1	

GRAIN SIZE DISTRIBUTION TEST RESULTS

Weathered Clay Crust

FIGURE 1



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
07-2	2	1.37-1.98

Received:

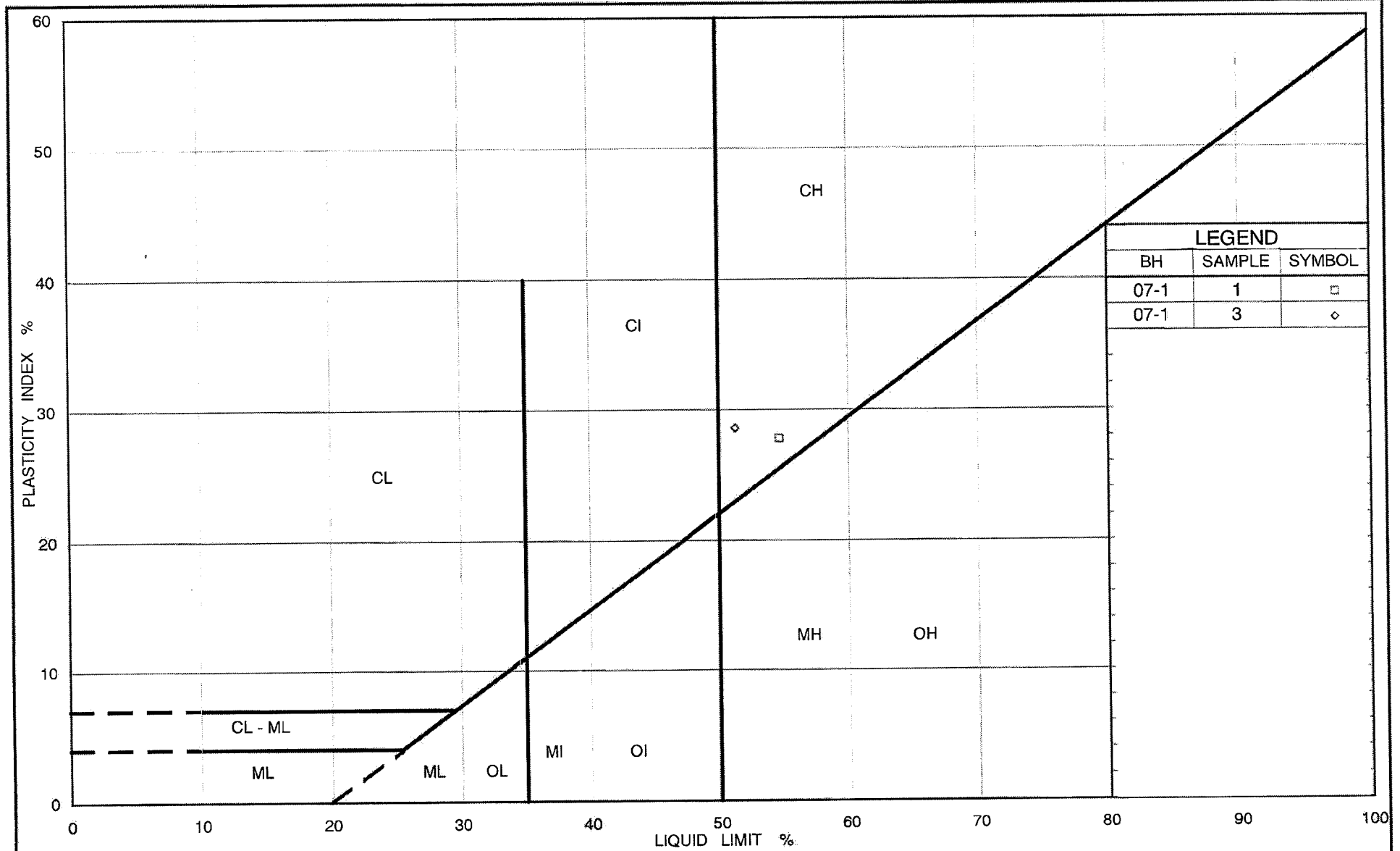
Project: 041111007

Golder Associates

9-May-07

Created by: MaD

Checked by: BaJ



Ministry of Transportation

Ontario

PLASTICITY CHART Weathered Clay

FIGURE 2

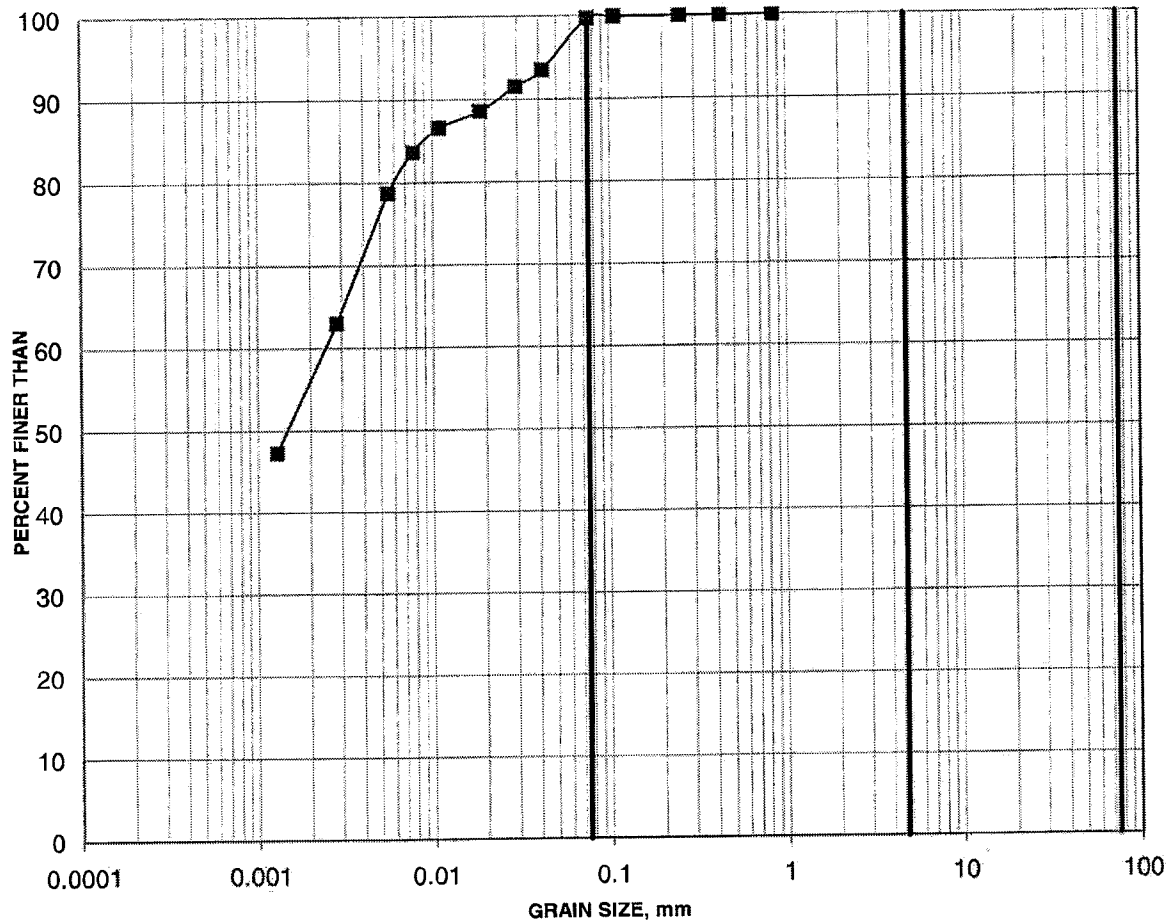
Project No. 041-111007

Checked By: SAT

GRAIN SIZE DISTRIBUTION TEST RESULTS

Unweathered Clay

FIGURE 3



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
07-1	4	3.05-3.66

Received:

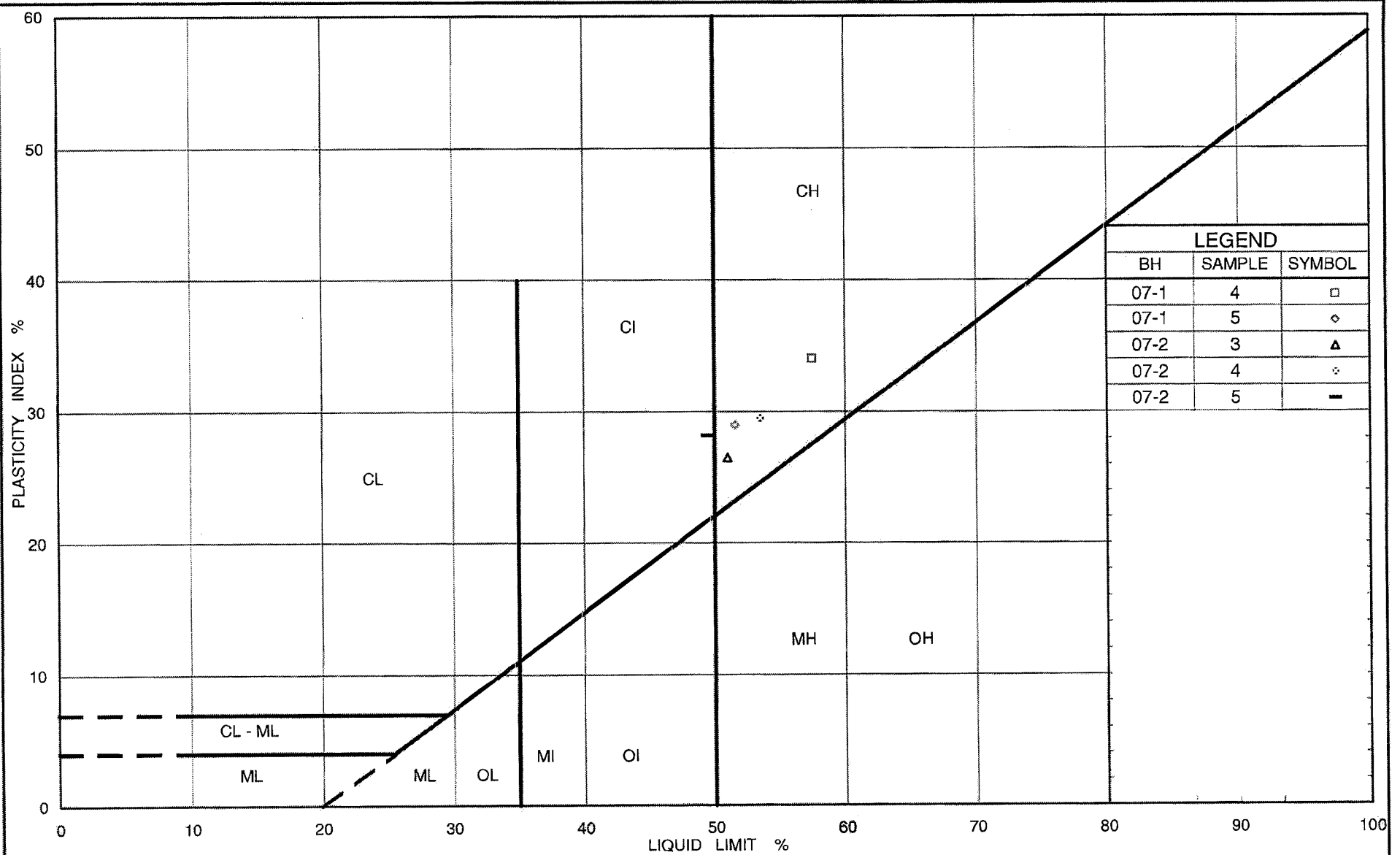
Project: 041111007

Golder Associates

9-May-07

Created by: MaD

Checked by: BaJ



Ontario

Ministry of Transportation

PLASTICITY CHART Unweathered Clay

FIGURE 4

Project No. 041-111007

Checked By: SAT

CONSOLIDATION TEST RESULTS **UNWEATHERED CLAY**

FIGURE 5A

SAMPLE IDENTIFICATION

Project Number	04-1111-007	Sample Number	3
Borehole Number	07-2	Sample Depth, m	2.9-3.35

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	02/25/2007		
Date Completed	03/07/2007		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	18.67
Sample Diameter, cm	6.32	Dry Unit Weight, kN/m ³	13.87
Area, cm ²	31.39	Specific Gravity, measured	2.77
Volume, cm ³	59.64	Solids Height, cm	0.970
Water Content, %	34.59	Volume of Solids, cm ³	30.45
Wet Mass, g	113.53	Volume of Voids, cm ³	29.19
Dry Mass, g	84.35	Degree of Saturation, %	100.0

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.900	0.959	1.900				
4.87	1.899	0.958	1.900	2	3.82E-01	1.08E-04	4.05E-06
9.47	1.894	0.952	1.897	13	5.87E-02	5.72E-04	3.29E-06
19.67	1.887	0.945	1.891	38	1.99E-02	3.61E-04	7.06E-07
39.23	1.881	0.939	1.884	30	2.51E-02	1.61E-04	3.97E-07
78.07	1.871	0.929	1.876	30	2.49E-02	1.36E-04	3.30E-07
156.16	1.855	0.912	1.863	53	1.39E-02	1.08E-04	1.47E-07
311.72	1.831	0.887	1.843	21	3.43E-02	8.12E-05	2.73E-07
623.59	1.776	0.831	1.804	41	1.68E-02	9.28E-05	1.53E-07
1247.92	1.679	0.731	1.728	41	1.54E-02	8.18E-05	1.24E-07
2496.15	1.565	0.613	1.622	32	1.74E-02	4.81E-05	8.21E-08
1247.92	1.577	0.626	1.571				
311.72	1.609	0.659	1.593				
78.07	1.648	0.699	1.629				
19.69	1.686	0.738	1.667				
4.87	1.714	0.767	1.700				

Note:

k calculated using cv based on t₉₀ values.

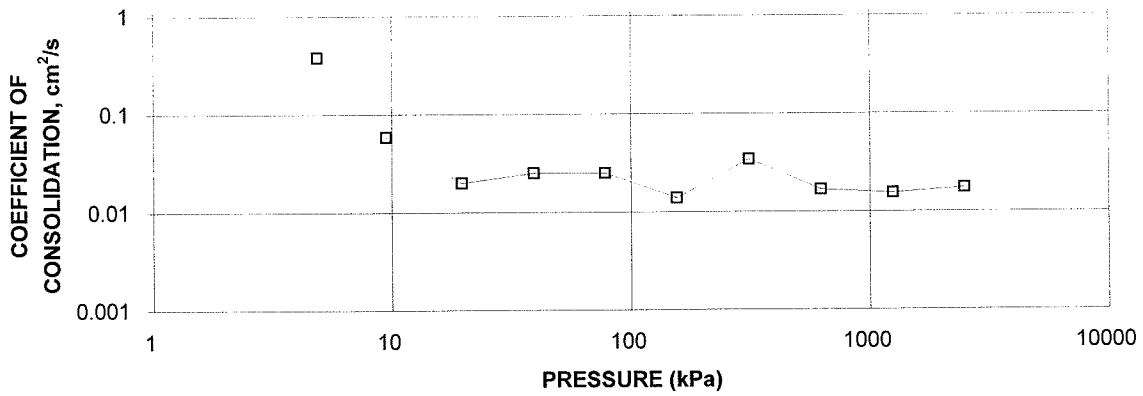
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.71	Unit Weight, kN/m ³	20.28
Sample Diameter, cm	6.32	Dry Unit Weight, kN/m ³	15.37
Area, cm ²	31.39	Specific Gravity, measured	2.77
Volume, cm ³	53.80	Solids Height, cm	0.970
Water Content, %	31.90	Volume of Solids, cm ³	30.45
Wet Mass, g	111.26	Volume of Voids, cm ³	23.35
Dry Mass, g	84.35		

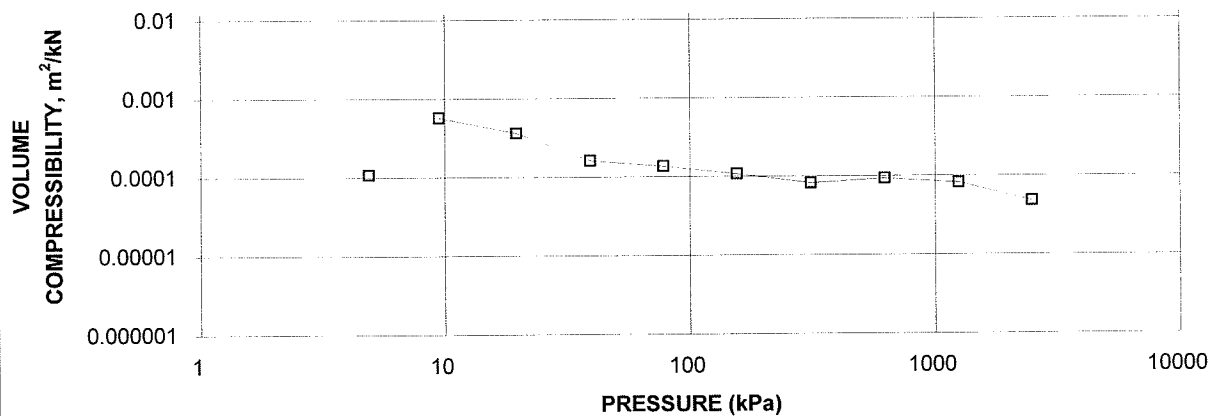
CONSOLIDATION TEST RESULTS UNWEATHERED CLAY

FIGURE 5B

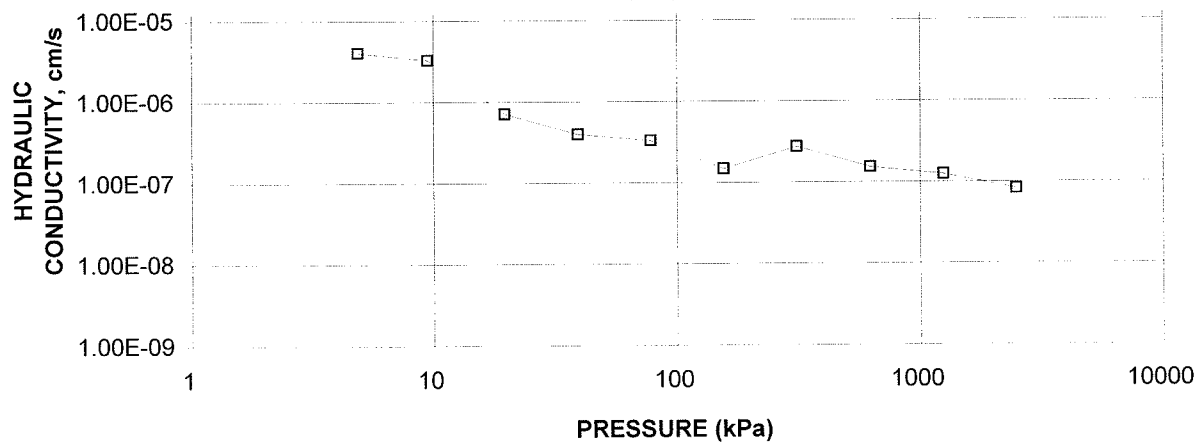
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa)
BH 07-2 SA 3



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH 07-2 SA 3



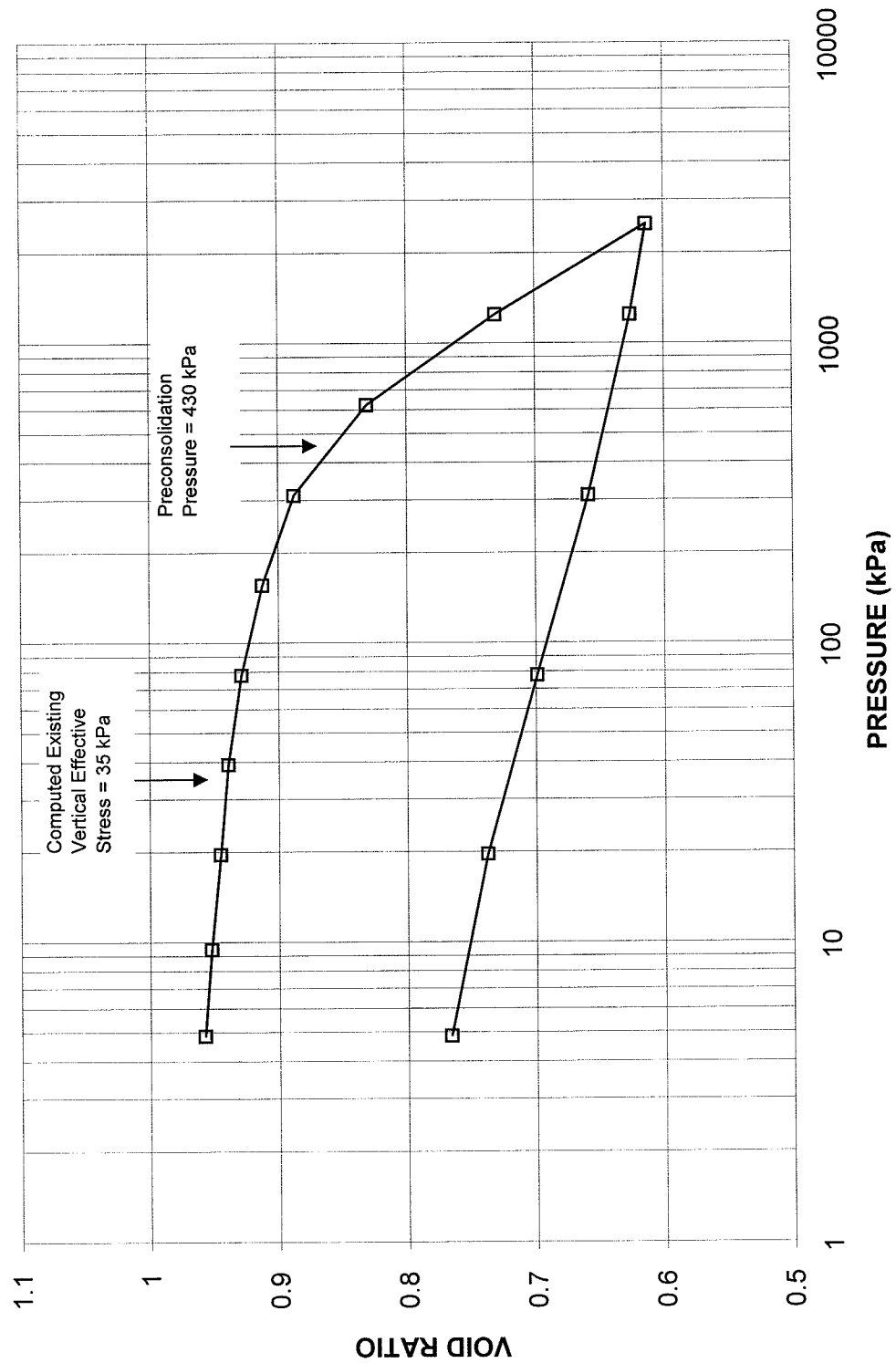
CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 07-2 SA 3



**CONSOLIDATION TEST RESULTS
UNWEATHERED CLAY**

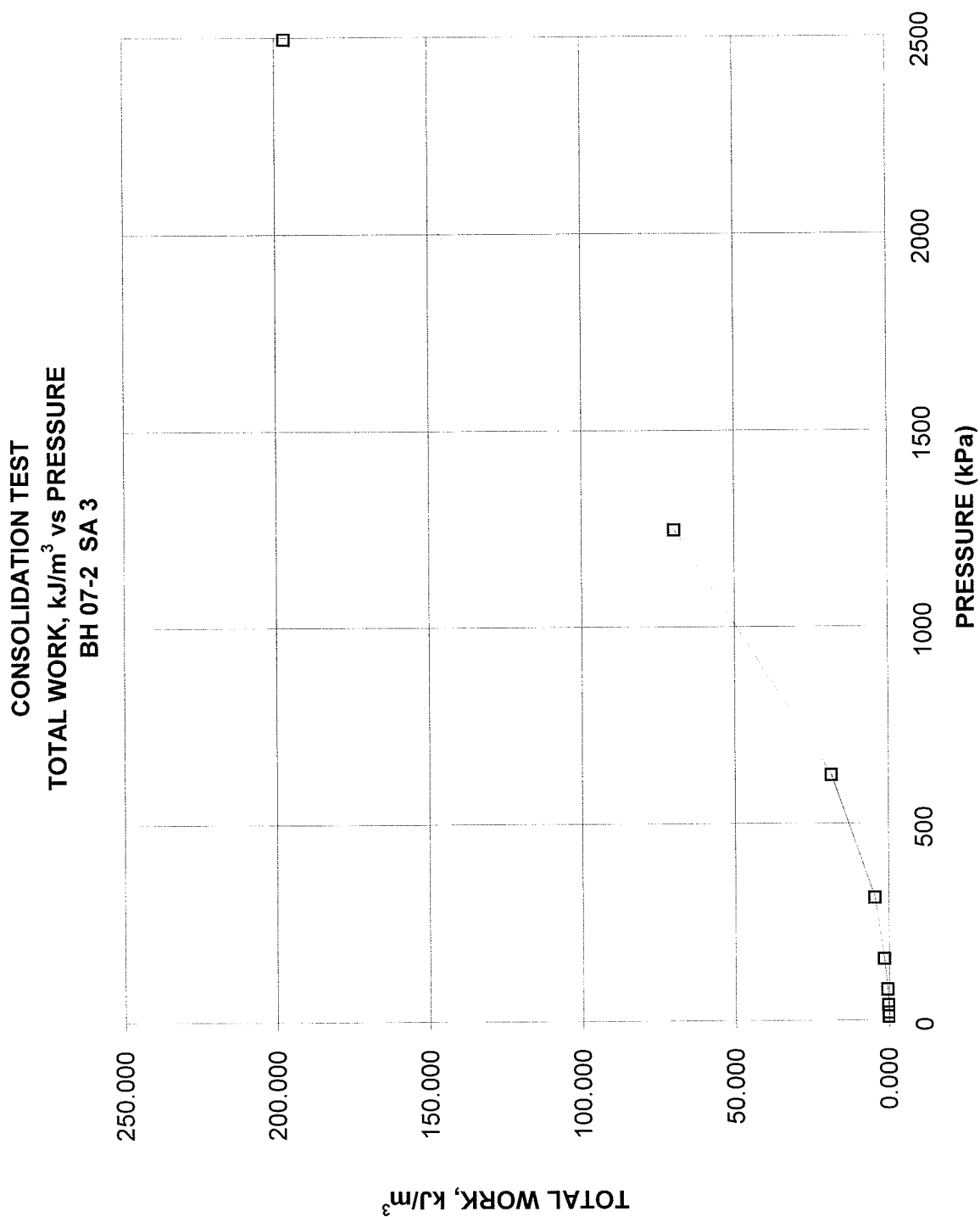
FIGURE 5C

**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 07-2 SA 3**



CONSOLIDATION TEST RESULTS UNWEATHERED CLAY

FIGURE 5D



Project No. 04-1111-007

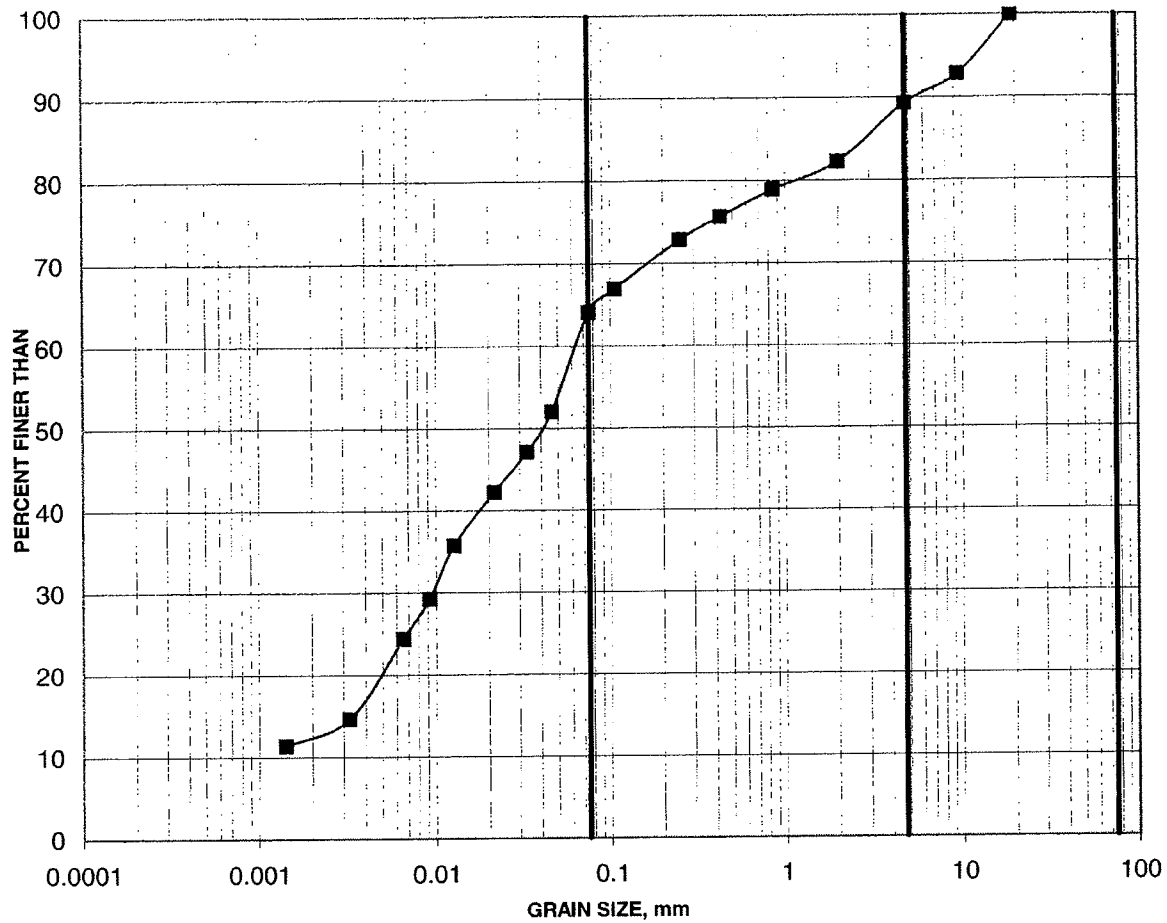
Golder Associates

Prepared By: LFG

Checked By: MM

GRAIN SIZE DISTRIBUTION Sandy Silt Till

FIGURE 6



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
—■— 07-1	7	7.62-7.80

Received:

Project: 041111007

Golder Associates

17-May-07

Created by: MaD

Checked by: BaJ