Golder Associates Ltd.

2180 Meadowvale Boulevard
Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT MOLSON PARK DRIVE OVERPASS STRUCTURE SITE 30-179 HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11 G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

Submitted to:

URS Cole, Sherman
75 Commerce Valley Drive East
Thornhill, Ontario
L3T 7N9

DISTRIBUTION:

1	Copy (Unbound)	-	URS Cole, Sherman, Thornhill, Ontario
2	Copies (Bound)	-	URS Cole, Sherman, Thornhill, Ontario
3	Copies	-	MTO Southwestern Region, London, Ontario
1	Copy	-	MTO Foundations Section, Downsview, Ontario
2	Copies	-	Golder Associates Ltd., Mississauga, Ontario

January 2002

001-1143F-7

TABLE OF CONTENTS

PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
MOLSON PARK DRIVE OVERPASS
STRUCTURE SITE 30-179
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

TABLE OF CONTENTS

LIST OF DRAWINGS

LIST OF APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Cole, Sherman (Cole, Sherman) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the ultimate widening of Highway 400 from 1 km south of Highway 89, northerly 30 km to Highway 11, in Simcoe County, Ontario. Foundation engineering services are required for the widening and / or replacement of eighteen existing overpass and underpass structures, as well as five structural culverts.

This report addresses the widening of the existing Molson Park Drive overpass structure. Existing subsurface data for this site from an investigation conducted by the MTO in 1997 (*"Foundation Investigation Report for Molson Park Drive Replacement Bridge, Highway 400"*, dated July 1997 – GEOCRES File No. 31D-362) were used to determine the subsurface conditions for this preliminary design study.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1192, dated June 2000.

2.0 SITE DESCRIPTION

The existing Molson Park Drive overpass structure is located about 3 km south of the Essa Road (Simcoe Road 30, formerly Highway 27) interchange, in Barrie, Ontario. The MTO has designated this overpass as Structure Site 30-179.

At the site, the original ground surface was at about Elevation 298 m to 301 m. The Highway 400 grade is at about Elevation 302 m to 302.5 m. Molson Park Drive has been constructed in cut, with its grade at about Elevation 296 m under Highway 400.

The existing overpass is a single span, steel box girder structure with integral abutments supported on driven steel H-piles. The contract drawings indicate that the pile cap underside is at Elevation 300 m at the south abutment, and Elevation 300.5 m at the north abutment. The design pile lengths shown on the contract drawings — 22.6 m and 23.1 m at the south and north abutments, respectively — correspond to a pile tip level of Elevation 277.4 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at this site by the Ministry of Transportation, Ontario (MTO) in April 1997. At that time, a total of nine boreholes were drilled – Boreholes 1, 2, 5 and 6 were advanced in the vicinity of the north abutments, and Boreholes 3, 4, 7, 8 and 9 were advanced in the vicinity of the south abutment. The boreholes were extended to between 19 m and 28 m below the original ground surface, to between Elevations 283 m and 273 m.

The boreholes were advanced using hollow stem augers. 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. The groundwater conditions in the open borehole were observed following the drilling operations. Laboratory index and classification tests, consisting of natural moisture contents, Atterberg Limits and grain size distributions, were carried out on selected soil samples.

The borehole locations, referenced to the MTM NAD83 survey system, and elevations, referenced to the geodetic datum, were established by MTO. The approximate borehole locations and northing and easting co-ordinates are shown on the attached Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 30 km section of Highway 400 traverses, from south to north, the following physiographic regions as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984): the Simcoe Lowlands; the Peterborough Drumlin Field; a second lobe of the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Simcoe Lowlands are present from the southern limit of the project to just south of Innisfil Creek, and again from Essa Road (Simcoe Road 30, formerly Highway 27) to about 1 km north of Dunlop Street (Simcoe Road 90, formerly Highway 90). The Peterborough Drumlin Field occupies the belt between these lobes of the Simcoe Lowlands, extending from just south of Innisfil Creek, which is located about 1 km north of Highway 89, to Essa Road. The Simcoe Uplands extend from about 1 km north of Dunlop Street to beyond the northern limit of the project at Highway 11.

The two sections where Highway 400 crosses the Simcoe Lowlands consist of two lobes of a sand plain which include the shores of Kempenfelt Bay, the Nottawasaga River and Innisfil Creek. The surficial soils of these sections of the Simcoe Lowlands consist primarily of sand, although silt, clay or peat may be found in low-lying areas.

The surficial soils in the Peterborough Drumlin Field, in which the Molson Park Drive overpass site is located, consist primarily of gravelly sand till or sand and gravel deposits. Drumlins (glacially-shaped hills) are more frequent in the southern portion of the section of the Peterborough Drumlin Field traversed by Highway 400. Deposits of silt, clay or peat may be found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Uplands physiographic region are primarily sandy silt till deposits, known to contain occasional boulders. Low-lying areas may be infilled with shallow sand and gravel deposits, which are shoreline deposits of a former glacial lake that once flooded the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the 1997 boreholes, together with the results of in-situ and laboratory testing, are given on the Record of Borehole sheets and figures contained in Appendix A. 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.

Boreholes 1, 2, 5 and 6 were advanced in the vicinity of the north abutment, and Boreholes 3, 4, 7, 8 and 9 were advanced in the vicinity of the south abutment. The approximate locations and ground surface elevations for these borings are shown on the attached Drawing 1.

In summary, the Highway 400 fill is underlain by compact to very dense silty sand and silt deposits, overlying stiff to hard silty clay to clayey silt, in turn underlain by a deposit of very dense silty sand to sand. All nine boreholes were terminated in the lower silty sand to sand. A detailed description of the subsurface conditions encountered in the 1997 boreholes is provided in the following sections.

4.2.1 Fill

About 1 m to 2 m of silty sand fill, which was placed to raise the grade of Highway 400, was encountered in Boreholes 5 to 9. Measured Standard Penetration Test (SPT) 'N' values of 7 to 27 blows per 0.3 m of penetration indicate that the fill has a loose to compact relative density.

Below the silty sand fill in Boreholes 5, 7 and 8, a thin layer of organics was encountered.

4.2.2 Upper Silty Sand

Silty sand was encountered at ground surface or beneath a thin topsoil layer in Boreholes 1 to 4, and below the fill and organics (where present) in Boreholes 5 to 9. The silty sand ranges from 0.8 m in thickness in the drainage ditch areas, to a maximum of 6.9 m in thickness. The base of the deposit varies from Elevation 299 m to 293 m, typically declining from west to east.

Thin layers of sandy silt were encountered within the silty sand deposit, as noted on the records of Boreholes 6 and 8. The grain size distribution test results obtained for representative samples of the silty sand are shown on Figure 1 in Appendix A.

In the upper 1 m to 2 m of the silty sand deposit in Boreholes 6, 7 and 8, SPT 'N' values of 3 to 10 blows per 0.3 m of penetration indicate a very loose to loose relative density. Elsewhere, the measured SPT 'N' values in the silty sand ranged from 12 to greater than 100 blows, but were typically more than 30 to greater than 100 blows per 0.3 m of penetration, indicating that this portion of the silty sand deposit has a predominantly dense to very dense relative density.

4.2.3 Silt

A thin layer of silt, ranging in thickness from about 1.5 m to 3.5 m, was encountered in all of the boreholes. The base of the silt layer was encountered between Elevation 296 m and 291 m, generally declining from west to east across the site.

The silt layer contains trace to some sand. Grain size distribution test results obtained on the coarse portion of the deposit are shown on Figure 2 in Appendix A.

The measured SPT 'N' values in the silt ranged from 23 to greater than 100 blows per 0.3 m of penetration, indicating that the silt layer typically has a compact to very dense relative density. However, where the silt layer is closer to the ground surface in Borehole 1, the measured SPT 'N' values of 9 and 11 blows per 0.3 m of penetration are indicative of a loose relative density.

4.2.4 Silty Clay to Clayey Silt

A 1 m to 2.5 m thick deposit of silty clay to clayey silt was encountered in all of the boreholes. The base of the deposit varies between Elevations 295 m and 288.5 m and, like the overlying deposits, generally declines from west to east at the site.

The results of Atterberg Limits testing on samples of the silty clay and clayey silt portions of this deposit are shown on Figures 3 and 4 of Appendix A. The test results, which are summarized in the table below, indicate that both the clayey silt and silty clay are inorganic and of low to intermediate plasticity, respectively. The measured natural moisture contents in samples of the silty clay to clayey silt ranged from 18 to 27 per cent, and were typically near the plastic limit of the material.

Soil Type	*Plastic Limits*	*Liquid Limits*	*Plasticity Indices*
Silty Clay	17 to 20 %	35 to 47 %	18 to 27 %
Clayey Silt	16 to 18 %	26 to 34 %	10 to 16 %

The measured SPT 'N' values ranged from 14 to 69 blows per 0.3 m of penetration, indicating that the silty clay to clayey silt has a stiff to hard consistency.

4.2.5 Lower Silty Sand to Sand

In all of the boreholes, a lower deposit of very dense silty sand to sand was encountered underlying the silty clay to clayey silt deposit. The surface of this deposit was typically encountered between Elevation 292 m and 288.5 m, generally declining from west to east. In Borehole 1, in the north western portion of the structure site, the surface of this lower silty sand to sand deposit was encountered at Elevation 295 m. This deposit extended to the maximum investigated depth, corresponding to approximately Elevation 273 m.

The silty sand to sand deposit contains trace quantities of gravel. Gravelly sand interlayers, between 0.5 m and 3 m in thickness, were encountered in Boreholes 1, 2, 4, 5, 7 and 8. These gravelly sand interlayers were typically encountered at or near the surface of the deposit. Grain size distribution envelopes for the silty sand to sand deposit and for the gravelly sand interlayers are shown on Figures 5 and 6, respectively.

The measured SPT 'N' values ranged from 85 to greater than 100 blows per 0.3 m of penetration, but were typically well in excess of 100 blows per 0.3 m of penetration. The lower silty sand to sand deposit has a very dense relative density.

4.3 Groundwater Conditions

The groundwater conditions were observed in the open boreholes following the 1997 drilling operations. There were no piezometers installed in the boreholes. Perched water was encountered in all of the boreholes between about Elevation 297.5 m and 294 m, typically just above the silt layer within the upper silty sand (in Boreholes 2 to 4 and 6 to 9), or within the silt layer (in Boreholes 1 and 5). It is considered that this shallow water table is perched by the silty clay to clayey silt deposit which underlies the upper silty sand and silt layers. This upper aquifer therefore has a saturated thickness of about 1 m to 4 m, although it may be up to about 6 m in saturated thickness as encountered in Borehole 6.

The 1997 borehole records note that the boreholes, which all terminated in the lower silty sand to sand stratum, were dry on completion of drilling, with the exception of Borehole 4 where the water level was at Elevation 272.8 m, about 0.1 m above the base of the borehole. It is noted that the dry conditions and low water level in Borehole 4 may not represent the stabilized conditions at the site.

In addition, it should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

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

DJE/LCC/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F\2002\RPT07-02JAN-MOLSON DOC

PART B

PRELIMINARY FOUNDATION DESIGN REPORT
MOLSON PARK DRIVE OVERPASS
STRUCTURE SITE 30-179
HIGHWAY 400 WIDENING FROM 1 KM SOUTH OF HIGHWAY 89 TO HIGHWAY 11
G.W.P. 30-95-00, AGREEMENT NO. 3005-A-000074

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation design recommendations for the widening of the existing Molson Park Drive overpass structure, associated with the widening of Highway 400. The recommendations are preliminary only and are based on interpretation of the factual data obtained during the subsurface investigation carried out in 1997 at this site. The interpretation and recommendations provided are intended for planning purposes only, to provide the information necessary at this stage of the study. 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. Further foundation investigation will be required at this structure site as part of the detailed design stage of the project.

It is understood that Highway 400 will be widened from its existing six-lane configuration to an interim configuration of eight lanes, and an ultimate configuration of ten lanes, and that an alternative for a twelve-lane express / collector system is under consideration between Molson Park Drive and Duckworth Street in Barrie. Throughout the project length, it is expected that the existing highway platform will be widened by between 13 m and 30 m. Widening of the Molson Park Drive overpass structure will therefore be necessary.

At the Molson Park Drive site, the Highway 400 grade is at about Elevation 302 m (about 2 m to 3 m above the adjacent ground), while Molson Park Drive has been constructed in cut and is at about Elevation 296 m under the highway. The existing overpass is a single-span, steel box girder structure with integral abutments supported on driven steel H-piles. The contract drawings indicate that the underside of the pile cap is at Elevation 300 m to 300.5 m. The HP 310 x 110 piles are approximately 23 m long, with their tips at Elevation 277.4 m.

5.2 Bridge Foundation Options

According to the 1997 borehole investigation data, the Highway 400 platform fill is underlain by compact to very dense silty sand and silt deposits, overlying stiff to hard silty clay to clayey silt, in turn underlain by a deposit of very dense silty sand to sand. A perched groundwater table exists at the site, atop the clayey silt to silty clay deposit; the perched water was encountered in the boreholes near the base of the upper silty sand or within the silt layer.

Based on these subsurface conditions and the configuration of the existing structure, it is recommended that any widened portion of this structure also be supported on driven steel H-piles. Consideration could be given to the use of spread footings if the widened structure can be maintained separate from the existing structure; however, in this case perched abutments supported on spread footings within the approach embankments should be adopted. The native soils at shallow depth are variable and spread footings would have to be extended to below the perched groundwater level over at least a portion of the footing length. The extent and elevation of the loose to compact silt deposit in the area of the proposed widening should be determined to assess the feasibility of the shallow footing option. Groundwater control will be required; however, because of the presence of the silt subsoil, well-point dewatering would be difficult.

Preliminary recommendations for spread footings for perched abutments, and for driven steel H-pile foundations are provided in the following sections.

5.3 Spread Footings

Consideration could be given to the use of abutment footings perched within the approach embankments. Given the existing road grades, perched abutments may not be feasible since the approach embankment is only about 2 m to 3 m high. Some sub-excavation of the loose upper sands and the silt deposit, where present, would be required.

5.3.1 Axial Geotechnical Resistance

For spread footings placed within the approach embankments on a compacted Granular 'A' core, a factored geotechnical resistance at ULS of 900 kPa may be assumed for preliminary design. The geotechnical resistance at SLS will depend on the thickness of Granular 'A' and the consistency and thickness of the underlying soils; a value of 350 kPa may be assumed for preliminary design. These values assume that the granular pad has a thickness of at least one footing width.

The geotechnical resistances provided herein 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 Ontario Highway Bridge Design Code (OHBDC).

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC. The angle of friction between the concrete footings and the compacted Granular 'A' pad should be taken as 30 degrees; the corresponding coefficient of friction would be 0.58.

5.3.3 Frost Protection

The footings should be provided with a minimum of 1.5 m of soil cover for frost protection.

5.4 Driven Steel H-Piles

Consideration should be given to supporting the widened portions of the overpass on steel H-piles driven to found within the very dense, lower silty sand to sand deposit. Based on the 1997 borehole investigation results and the configuration of the existing deep foundations, it is anticipated that the pile founding level would be at about Elevation 277.5 m. It is noted that heavy driving is anticipated, given the high Standard Penetration Test 'N' values and the presence of the gravelly sand interlayers which may be difficult to penetrate.

5.4.1 Axial Geotechnical Resistance

For preliminary design, the factored axial resistance at ULS for steel HP 310x110 H-piles driven to found within the very dense, lower silty sand to sand deposit may be taken as 1,400 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,200 kN.

To achieve the above design resistances, the piles should be driven to about Elevation 277.5 m. The piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

5.4.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

If integral abutments are under consideration, there may also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the following equation:

$$k_h = \frac{n_h z}{B}$$

where n_h is the constant of subgrade reaction
z is the depth (m)
B is the pile diameter (m)

The piles will extend through the generally dense to very dense upper silty sand and silt layers, the relatively thin silty clay to clayey silt deposit, and into the very dense lower silty sand to sand deposit. Above Elevation 294 m at the north abutment and above Elevation 292 m at the south abutment, the range in value of n_h may be taken as 10 MPa/m to 20 MPa/m in the structural analysis. Below these elevations, the range of n_h may be taken as 16 MPa/m to 20 MPa/m in the structural analysis.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	*Subgrade Reaction Reduction Factor R*
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.4.3 Frost Protection

The pile caps should be provided with 1.5 m of soil cover for frost protection.

5.5 Lateral Earth Pressures

The lateral pressures acting on the bridge abutments and associated retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments, in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00

- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the abutment wall, in accordance with OHBDC Figure 6-7.4.3. Compaction equipment should be used in accordance with OPSS 501.06.

- The granular fill may be placed either in a zone with width equal to at least 1.5 m behind the back of the stem (Case I from OHBDC Figure 6-7.4.1) 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 from OHBDC Figure 6-7.4.4).

- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

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

- For Case II, the pressures are based on the granular fill as placed 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 lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the abutment and retaining walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

5.6 Design of Permanent Cut Slopes

In the vicinity of this overpass, the Molson Park Drive cut is up to about 5 m in depth, with the road grade at about Elevation 296 m and the drainage ditch invert level as low as Elevation 295 m. Based on the topographic information on the Engineering and Title Records plates and on site reconnaissance, the existing cut slopes are formed at a gradient of about 2 horizontal to 1 vertical (2H:1V). For preliminary design purposes, a maximum gradient of 2H:1V may be assumed for any new permanent cut slopes.

It is noted that the base of the cut will extend below the perched groundwater level at the site. Protection of the lower portion of the cut slope, for example using a drainage blanket, will be necessary to prevent sloughing of the toe of the permanent cut slopes. It is understood that prior to the reconstruction of this structure, seepage and sloughing were evident in some areas of the permanent cut slope toe.

5.7 Design and Construction Considerations

5.7.1 Dewatering

A perched water table was encountered in the 1997 boreholes, between Elevation 297.5 m and 294 m. This shallow aquifer appears to be perched by the silty clay to clayey silt deposit, and has a saturated thickness of about 1 m to 4 m associated with the upper silty sand and silt layers. A groundwater control scheme will be required for pile cap construction if excavations extend below these elevations.

Where encountered at the subgrade or pile cap underside level, the silt soils will be particularly susceptible to disturbance from ponded water and construction traffic. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance.

5.7.2 Excavation

The footing or pile cap excavations will extend a minimum of 1.5 m below lowest surrounding grade, generally through loose to dense silty sand to silt soils. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The loose to very dense silty sand to silt soils would be classified as Type 2 to 3 soil, assuming that appropriate groundwater control is in place. Temporary open-cut slopes should therefore be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, footing or pile cap excavations could also be carried out within a braced excavation.

5.7.3 Obstructions

Although no cobbles or boulders were noted on the borehole records from the 1997 subsurface investigation, it should be recognized that cobbles and boulders are inherent in glaciolacustrine materials. Such obstructions should therefore be expected during driving of steel H-piles for deep foundations or for temporary roadway protection.

GOLDER ASSOCIATES LTD.

REGISTERED PROFESSIONAL ENGINEER
A. S POSCHMANN
PROVINCE OF ONTARIO

Lisa C. Coyne, P.Eng.,
Geotechnical Engineer

Anne S. Poschmann, P.Eng.,
Principal

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

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

DJE/LCC/ASP/FJH/clg
N:\ACTIVE\1100\001-1143F\2002\RPT07-02JAN-MOLSON.DOC

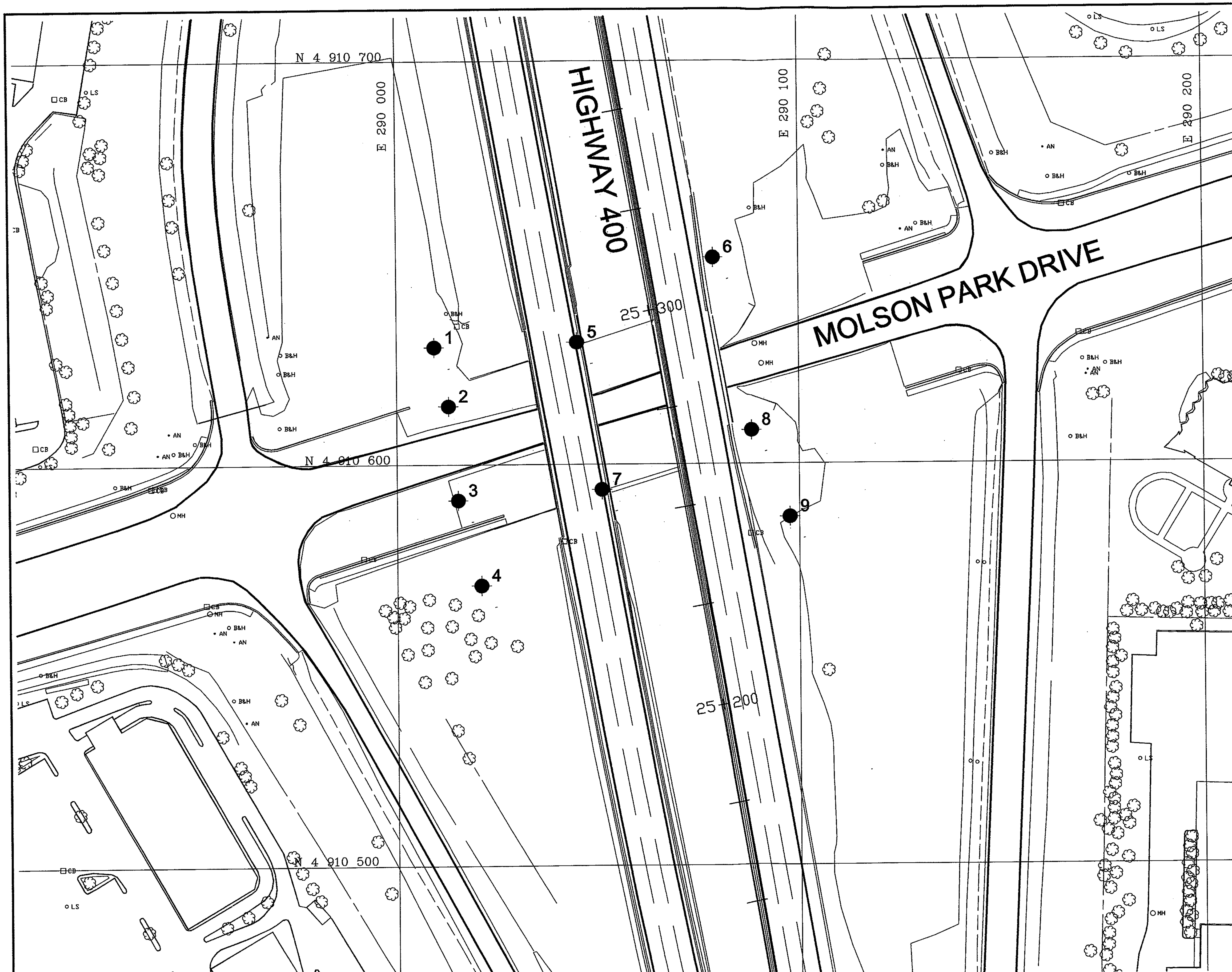
DIST	HWY 400
CONT. No.	
GWP No.	30-95-00

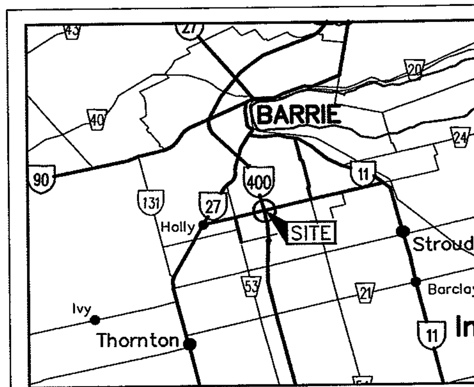
MOLSON PARK DRIVE OVERPASS
HWY 400
BOREHOLE LOCATION PLAN

SHEET

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

0 2.0 4.0 6.0 8.0 10.0 20.0
SCALE IN KILOMETRES

LEGEND

- ● Borehole, previous investigation
- ⊕ Borehole, present investigation

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
1	300.9	4,910,629.3	290,009.5
2	296.1	4,910,614.6	290,013.0
3	296.1	4,910,591.1	290,015.3
4	300.4	4,910,569.8	290,021.0
5	301.8	4,910,630.5	290,044.8
6	302.0	4,910,651.4	290,078.7
7	301.3	4,910,593.7	290,050.8
8	301.3	4,910,608.4	290,088.2
9	300.2	4,910,586.7	290,097.7

REFERENCE
This drawing was created from digital file "50205.dwg" provided by URS Cole Sherman

0 10 20 30 40 50 100m
SCALE IN METRES

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

Geocres No.

HWY. No. 400		PROJECT NO.: 001-1143F	
SUBM'D. LCC	CHKD: ASP	DATE: JANUARY 2001	SITE 30-179
DRAWN: MHW	CHKD. LCC	APPD. ASP	DWG. 1

P1143F12.DWG

APPENDIX A

RECORDS OF BOREHOLES AND TEST RESULTS – 1997 INVESTIGATION

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

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

N (BLOWS/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1

METRIC

W.P. 26 - 96 - 01
LOCATION CO - ORDS: N 4 910 629.3; E 290 009.5
ORIGINATED BY M V&P C

DIST 33 HWY 400
BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST
COMPILED BY M V

DATUM GEODETIC
DATE 1997 04 02
CHECKED BY T C K

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
300.9	Ground Surface
0.0	Topsoil
299.4 / 1.5	SILTY SAND, Loose
296.2 / 4.7	SILT, Trace of Sand, Loose to Compact
295.0 / 5.9	SILTY CLAY, Stiff to Very Stiff
	Gravelly Sand
276.1 / 24.8	SILTY SAND TO SAND, Trace of Gravel, Very Dense
24.8	End of Borehole

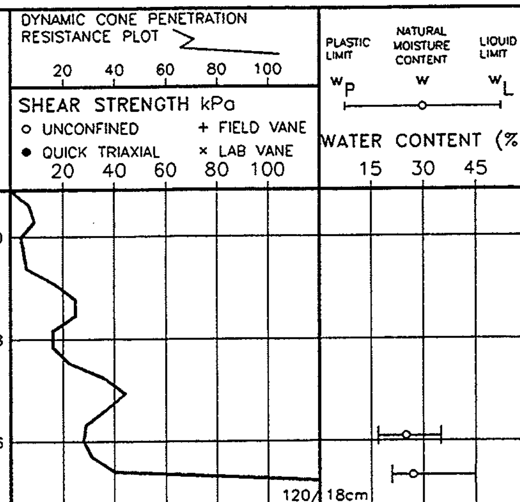
* Note:
Borehole Dry on Completion
Perched Water Level Encountered at El. 297.4

SAMPLES

NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	9	
2	SS	11	0 5 (95)
3	SS	13	
4	SS	21	
5	SS	156	24 66 (10)
6	SS	128	
7	SS	133	
8	SS	100/13cm	31 52 (17)
9	SS	123	
10	SS	128	
11	SS	145	
12	SS	113	
13	SS	125	
14	SS	161	
15	SS	154	
16	SS	177	4 86 (10)
17	SS	130	

GROUND WATER CONDITIONS: *

ELEVATION SCALE: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278



UNIT WEIGHT γ kN/m^3

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15\overset{20}{\diamond}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 614.6; E 290 013.0 ORIGINATED BY M V

DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V

DATUM GEODETIC DATE 1997 04 01 CHECKED BY T C K

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.1	Drainage Ditch					
0.0 / 295.3	SILTY SAND, Trace of Organic, Compact	1	SS	21	*	
0.8 / 293.8	SILT, Some Sand, Compact	2	SS	30		2 18 (80)
		3	SS	24		
2.3 / 292.4	SILTY CLAY, Stiff to Very Stiff	4	SS	14		
		5	SS	23		
3.7	SILTY SAND TO SAND, Trace of Gravel, Very Dense	6	SS	125		5 81 (14)
		7	SS	125 /25cm		
	Gravelly Sand	8	SS	141		15 71 (14)
		9	SS	100 /10cm		
		10	SS	150 /20cm		
		11	SS	168 /20cm		
		12	SS	102 /15cm		
		13	SS	137		3 86 (11)
		14	SS	177		
277.4		15	SS	146		
18.7	End of Borehole					

Elevation scale: 294, 292, 290, 288, 286, 284, 282, 280, 278

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100 — 120/28cm

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 15 30 45

UNIT WEIGHT γ kN/m³

* Note:
  Borhole Dry on Completion
  Perched Water Level Encountered at El. 296.0

+3, ×5 : Numbers refer to Sensitivity   15 ⊕ 5 (20) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 26 - 96 - 01

LOCATION CO - ORDS: N 4 910 591.1; E 290 015.3

ORIGINATED BY M V

DIST 33 HWY 400

BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST

COMPILED BY M V

DATUM GEODETIC

DATE 1997 04 14

CHECKED BY T C K

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L

WATER CONTENT (%): 15 30 45

UNIT WEIGHT γ kN/m^3

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
296.1	Drainage Ditch						
0.0	Silt and Organic				*		
	SILTY SAND, Very Dense	1	SS	54			0 79 (21)
294.2		2	SS	71		294	
1.9	Clayey Silt, Hard	3	SS	43			0 5 (95)
	SILT, Trace of Sand, Dense to Very Dense	4	SS	59			
292.4 / 3.7	SILTY CLAY, Hard	5	SS	31		292	
291.1 / 5.0		6	SS	55			
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	7	SS	156		290	6 84 (10)
		8	SS	168		288	10 78 (12)
		9	SS	118		286	
		10	SS	108		284	
		11	SS	115		282	
		12	SS	147		280	0 94 (6)
		13	SS	140		278	
274.3		14	SS	130		276	
21.8	End of Borehole						

Dynamic cone penetration: 120/23cm

* Note:

Borehole Dry on Completion

Perched Water Level Encountered at El. 295.9

+3, ×5: Numbers refer to Sensitivity

15⟡5 (%) STRAIN AT FAILURE (20)

RECORD OF BOREHOLE No 4

 METRIC

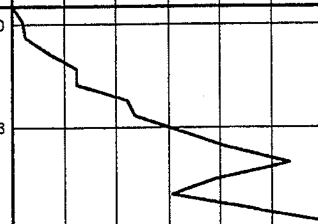
W.P. 26 - 96 - 01	LOCATION	CO - ORDS: N 4 910 569.8; E 290 021.0	ORIGINATED BY M V
DIST 33 HWY 400	BOREHOLE TYPE	CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST	COMPILED BY M V
DATUM GEODETIC	DATE	1997 04 10	CHECKED BY T C K

Column headings: SOIL PROFILE (ELEV / DEPTH; DESCRIPTION; STRAT PLOT) — SAMPLES (NUMBER; TYPE; 'N' VALUES) — GROUND WATER CONDITIONS — ELEVATION SCALE — DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100) — PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 15 30 45 — UNIT WEIGHT γ kN/m³ — REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION
300.4	Ground Surface
0.0	Topsoil
	SILTY SAND, Compact to Very Dense
295.2 / 5.2	SILT, Trace of Sand, Dense to Very Dense
292.9 / 7.5	CLAYEY SILT, Hard
291.0 / 9.4	SILTY SAND TO SAND, Trace of Gravel, Very Dense (Gravelly Sand layer near El. 288)
272.7 / 27.7	End of Borehole

NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS
1	SS	28	
2	SS	38	
3	SS	54	
4	SS	63	
5	SS	45	
6	SS	55	
7	SS	30	
8	SS	32	
9	SS	133	/23cm
10	SS	102	/10cm
11	SS	136	/5cm
12	SS	85	
13	SS	96	
14	SS	88	
15	SS	120	
16	SS	128	
17	SS	119	/15cm
18	SS	112	/15cm ▼

Elevation scale: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278, 276, 274



Water content at El. ≈291.8: w_P ≈ 17, w ≈ 22, w_L ≈ 27 (%)

Sample	GR	SA	SI	CL
3	0	81	(19)	
6	0	9	(91)	
11	38	55	(7)	
12	2	93	(5)	
18	1	91	(8)	

• Note: Water Level on Completion at El. 272.8

Perched Water Level Encountered at El. 295.9

$+^3, \times^5$: Numbers refer to Sensitivity $15 \overset{20}{\diamond} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No 5

METRIC

W.P. 26 - 96 - 01
LOCATION CO - ORDS: N 4 910 630.5; E 290 044.8
ORIGINATED BY M V&P C

DIST 33 HWY 400
BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST
COMPILED BY M V

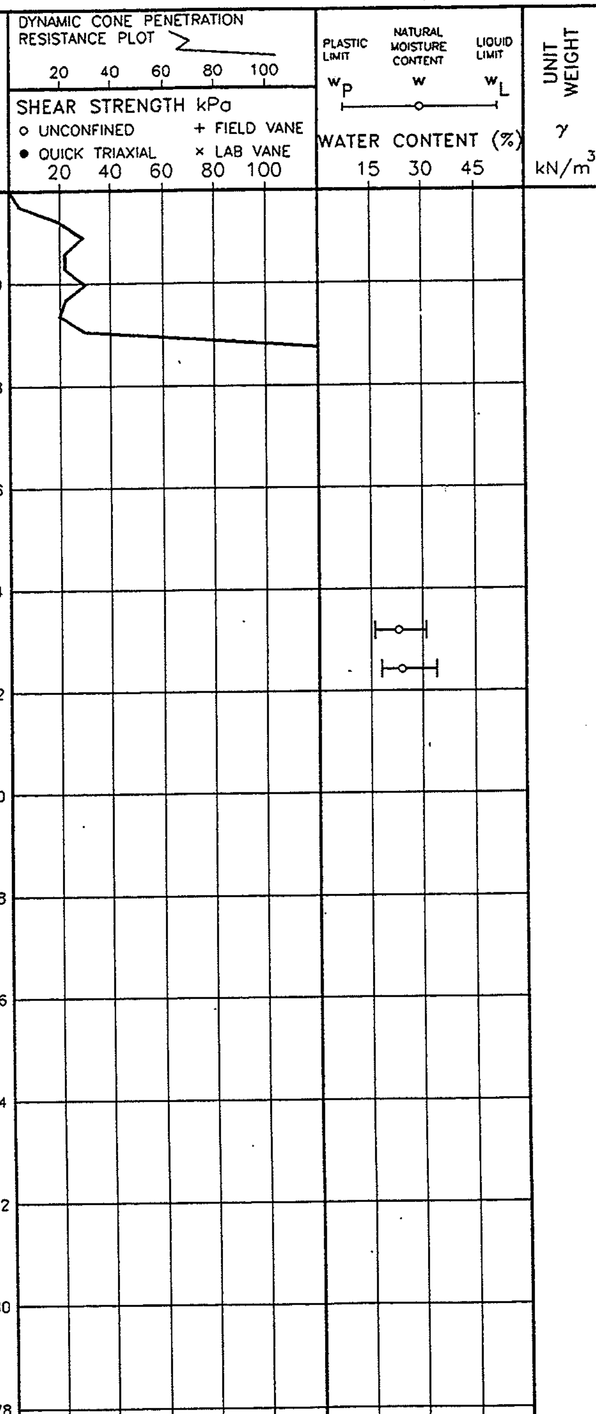
DATUM GEODETIC
DATE 1997 04 02 & 03
CHECKED BY T C K

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
301.8	Ground Surface
0.0	SILTY SAND, Compact, (Fill) Organic
299.9 / 1.9	SILTY SAND, Compact to Very Dense
295.9 / 5.9	SILT, Trace of Sand, Compact to Dense
293.3 / 8.5	CLAYEY SILT, Very Stiff
292.0 / 9.8	SILTY SAND TO SAND, Trace of Gravel, Very Dense (Gravelly Sand)
277.8 / 24.0	End of Borehole

SAMPLES

NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	25	*	
2	SS	10		
3	SS	14		0 80 (20)
4	SS	52		
5	SS	96		
6	SS	97		0 62 (38)
7	SS	31		
8	SS	36		
9	SS	23		0 6 (94)
10	SS	29		
11	SS	23		
12	SS	28		
13	SS	148	/20cm	
14	SS	200	/28cm	24 67 (9)
15	SS	210	/25cm	
16	SS	195	/28cm	
17	SS	210	/28cm	
18	SS	230	/25cm	
19	SS	235	/25cm	8 72 (20)
20	SS	268	/23cm	
21	SS	257	/25cm	0 92 (8)



* Note:
  Borehole Dry on Completion
  Perched Water Level Encountered at El. 295.6

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15\overset{20}{\diamond}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 6

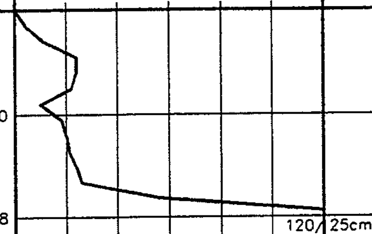
METRIC

W.P. 26 - 96 - 01	LOCATION CO - ORDS: N 4 910 651.4; E 290 078.7		ORIGINATED BY M V
DIST 33	HWY 400	BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST	COMPILED BY M V
DATUM GEODETIC		DATE 1997 04 08	CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
302.0	Ground Surface					
0.0	Topsoil; SILTY SAND, Trace of Gravel, Trace of Organic, Compact (Fill)	1	SS	27	*	
300.1		2	SS	19		
1.9	SILTY SAND, Loose	3	SS	9		
		4	SS	41		
	Sandy Silt	5	SS	50		0 35 (65)
	SILTY SAND, Dense to Very Dense	6	SS	60		
		7	SS	250		
		8	SS	269		
293.3		9	SS	144		0 71 (29)
8.7	SILT, Trace of Sand, Very Dense	10	SS	103		
		11	SS	62		0 10 (90)
291.0		12	SS	162 /25cm		0 12 (88)
11.0	SILTY CLAY, Hard	13	SS	30		
		14	SS	69		
288.5						
13.5	SILTY SAND TO SAND, Trace of Gravel, Very Dense	15	SS	154		9 78 (13)
		16	SS	125 /10cm		
		17	SS	154 /23cm		
		18	SS	176 /15cm		
		19	SS	123 /15cm		
277.8		20	SS	131 /15cm		0 90 (10)
24.2	End of Borehole					

Elevation scale: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)



PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 15 30 45

UNIT WEIGHT γ kN/m^3

• Note:
Borhole Dry on Completion
Perched Water Level Encountered at El. 297.3

$+^3, \times^5$: Numbers refer to Sensitivity; $15 \diamond 5$ (%) STRAIN AT FAILURE (20)

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 7

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 593.7; E 290 050.8 ORIGINATED BY M V

DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V

DATUM GEODETIC DATE 1997 04 11 CHECKED BY T C K

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
301.3	Ground Surface					
0.0	SILTY SAND, Loose (Fill)	1	SS	7	*	
300.2 / 1.1	Organic; Very Loose to Loose	2	SS	7		0 71 (29)
		3	SS	3		
	SILTY SAND, Dense	4	SS	49		
		5	SS	40		
295.4 / 5.9	SILT, Trace of Sand, Dense to Very Dense	6	SS	46		0 16 (84)
		7	SS	52		
		8	SS	40		0 8 (92)
293.1 / 8.2	SILTY CLAY, Hard	9	SS	38		
		10	SS	37		
291.3 / 10.0	SILTY SAND TO SAND, Trace of Gravel, Very Dense	11	SS	138		
		12	SS	104	/15cm	
		13	SS	155		
	Gravelly Sand	14	SS	126	/15cm	26 63 (11)
		15	SS	175		6 86 (8)
		16	SS	115		
		17	SS	119		
		18	SS	117		
276.5		19	SS	159		
24.8	End of Borehole					

Elevation scale: 300, 298, 296, 294, 292, 290, 288, 286, 284, 282, 280, 278

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100): 120/25cm

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 15 30 45

UNIT WEIGHT γ kN/m^3

* Note:
Borehole Dry on Completion
Perched Water Level Encountered at El. 295.7

$+^3, \times^5$: Numbers refer to Sensitivity

15 ⟡ 5 (20) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 8

METRIC

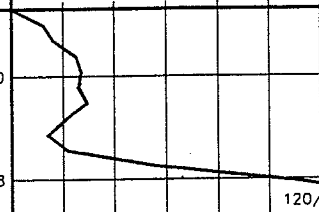
W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 608.4; E 290 088.2 ORIGINATED BY P C

DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V

DATUM GEODETIC DATE 1997 04 04 CHECKED BY T C K

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
301.3	Ground Surface						
0.0	SILTY SAND, Compact (Fill)	1	SS	14	*	300	
299.9 / 1.4	Organic	2	SS	5			
	Loose	3	SS	10			
	SILTY SAND, Very Dense	4	SS	49		298	
		5	SS	85			0 82 (18)
		6	SS	78			
		7	SS	98		296	
		8	SS	62			
	Sandy Silt	9	SS	61		294	
293.0		10	SS	72			0 32 (68)
8.3	SILT, Trace of Sand, Dense	11	SS	40			0 8 (92)
291.3		12	SS	36		292	
10.0 / 290.1	SILTY CLAY, Hard	13	SS	40		290	
11.2	Gravelly Sand	14	SS	251			15 76 (9)
	SILTY SAND TO SAND, Trace of Gravel, Very Dense	15	SS	254	/25cm	288	
		16	SS	165		286	
		17	SS	241		284	
282.6		18	SS	239			0 83 (17)
18.7	End of Borehole						

Dynamic Cone Penetration Resistance Plot (20 40 60 80 100); Shear Strength kPa: o Unconfined, + Field Vane, • Quick Triaxial, × Lab Vane (20 40 60 80 100); Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L; Water Content (%) 15 30 45; Unit Weight γ kN/m^3



* Note:
Borehole Dry on Completion
Perched Water Level Encountered at El. 295.1

$+^3$, $\times^5$: Numbers refer to Sensitivity $15 \diamond 5$ (20) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 9

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 586.7; E 290 097.7 ORIGINATED BY M V

DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V

DATUM GEODETIC DATE 1997 04 07 CHECKED BY T C K

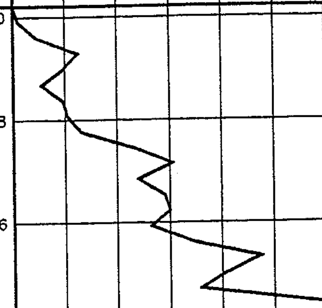
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
300.2	Ground Surface						
0.0	SILTY SAND, Trace of Organic, Loose (Fill)				*	300	
299.2 / 1.0	SILTY SAND, Compact to Dense	1	SS	12			
		2	SS	12			
		3	SS	35		298	0 88 (12)
		4	SS	42			
						296	
		5	SS	31			
		6	SS	29		294	
293.5 / 6.7	SILT, Some Sand, Compact to Dense	7	SS	51			0 17 (83)
		8	SS	29		292	
		9	SS	35			
		10	SS	43			0 24 (76)
290.1 / 10.1	SILTY CLAY, Hard	11	SS	30		290	
		12	SS	67			
289.2 / 11.0	SILTY SAND TO SAND, Trace of Gravel, Very Dense					288	
		13	SS	181	/28cm		
		14	SS	135	/23cm	286	
		15	SS	116	/15cm	284	
		16	SS	155	/23cm		
		17	SS	105	/15cm	282	10 79 (11)
						280	
		18	SS	156	/15cm	278	
						276	5 80 (15)
275.5 / 24.7	End of Borehole	19	SS	112	/15cm		

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100)

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100)

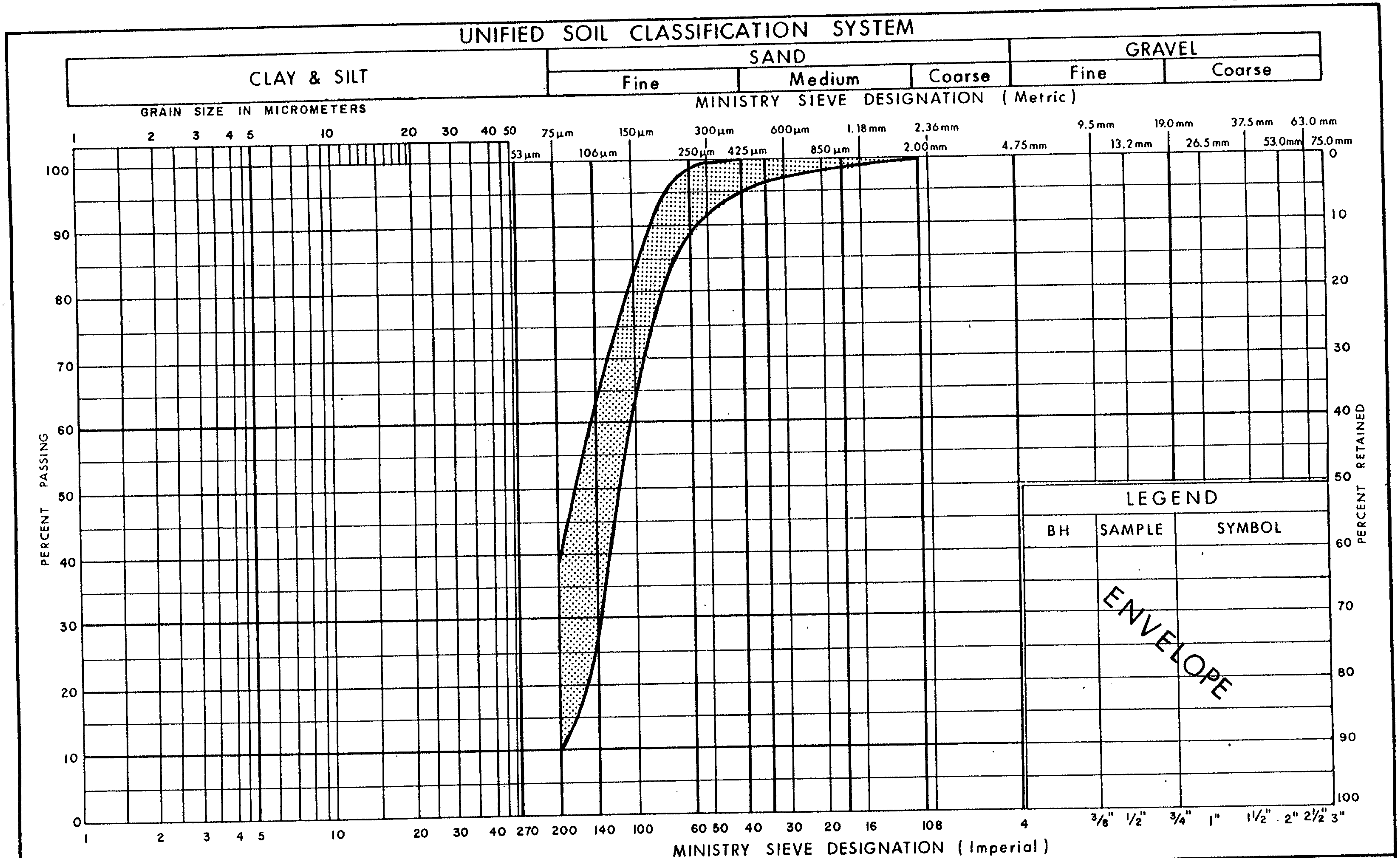
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 15, 30, 45

UNIT WEIGHT γ kN/m^3



* Note:
Borehole Dry on Completion
Perched Water Level Encountered at El. 294.0

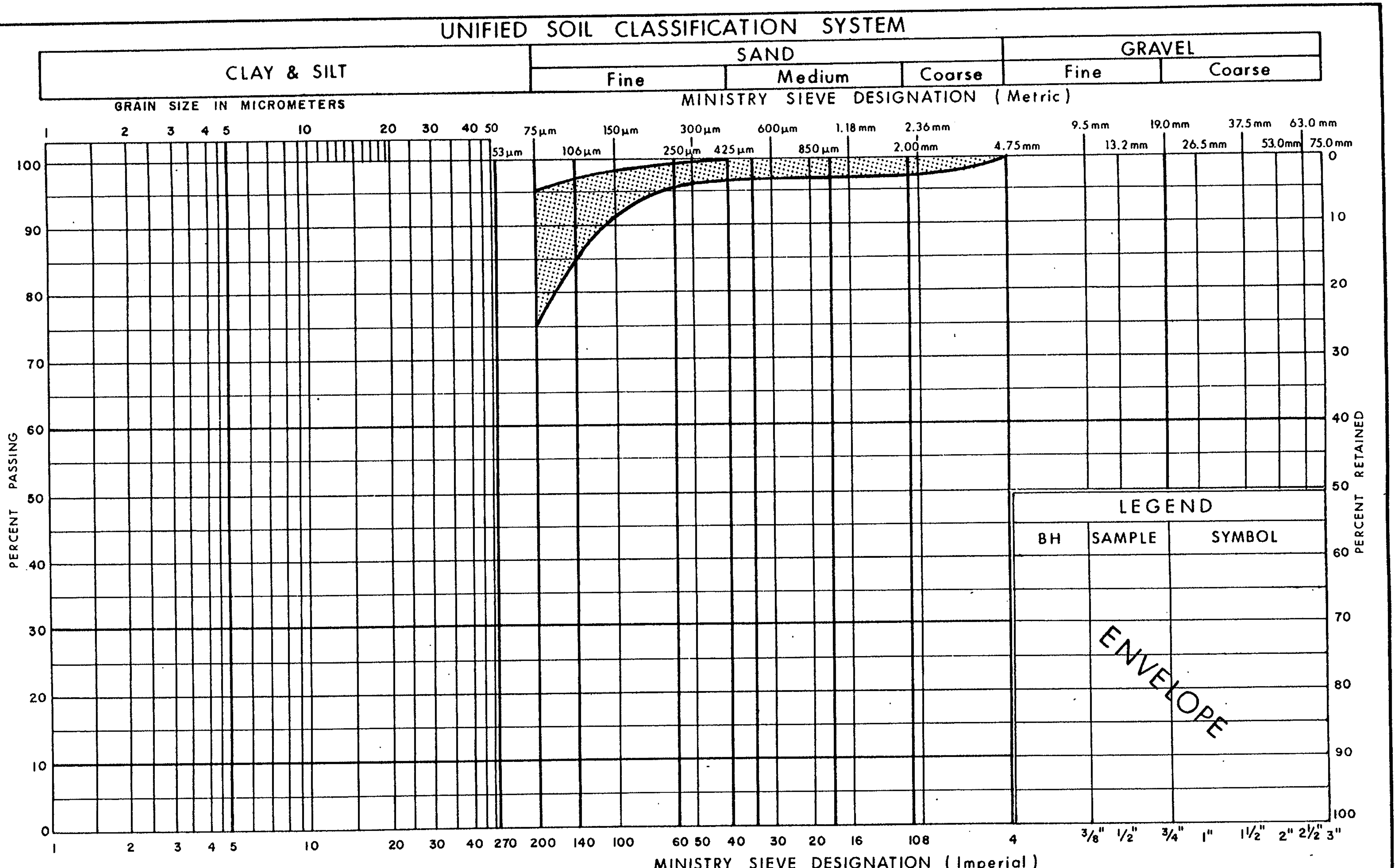
$+^3$, $\times^5$: Numbers refer to Sensitivity $15 \overset{20}{\diamond} 5$ (%) STRAIN AT FAILURE



GRAIN SIZE DISTRIBUTION
SILTY SAND

FIG No 1
W P 26-96-01

78 12 M



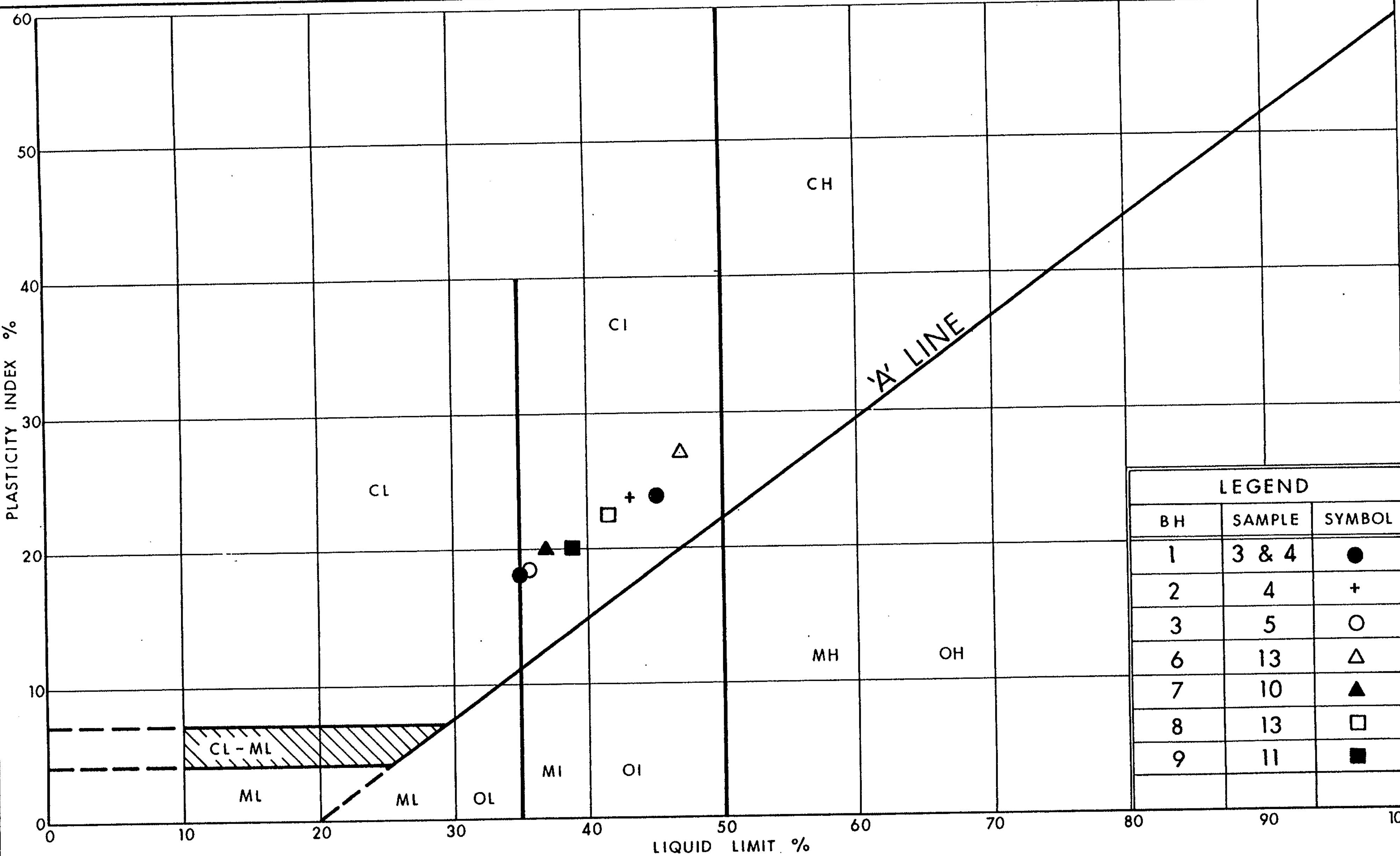
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILT, TRACE / SOME SAND

FIG No 2

W P 26-96-01



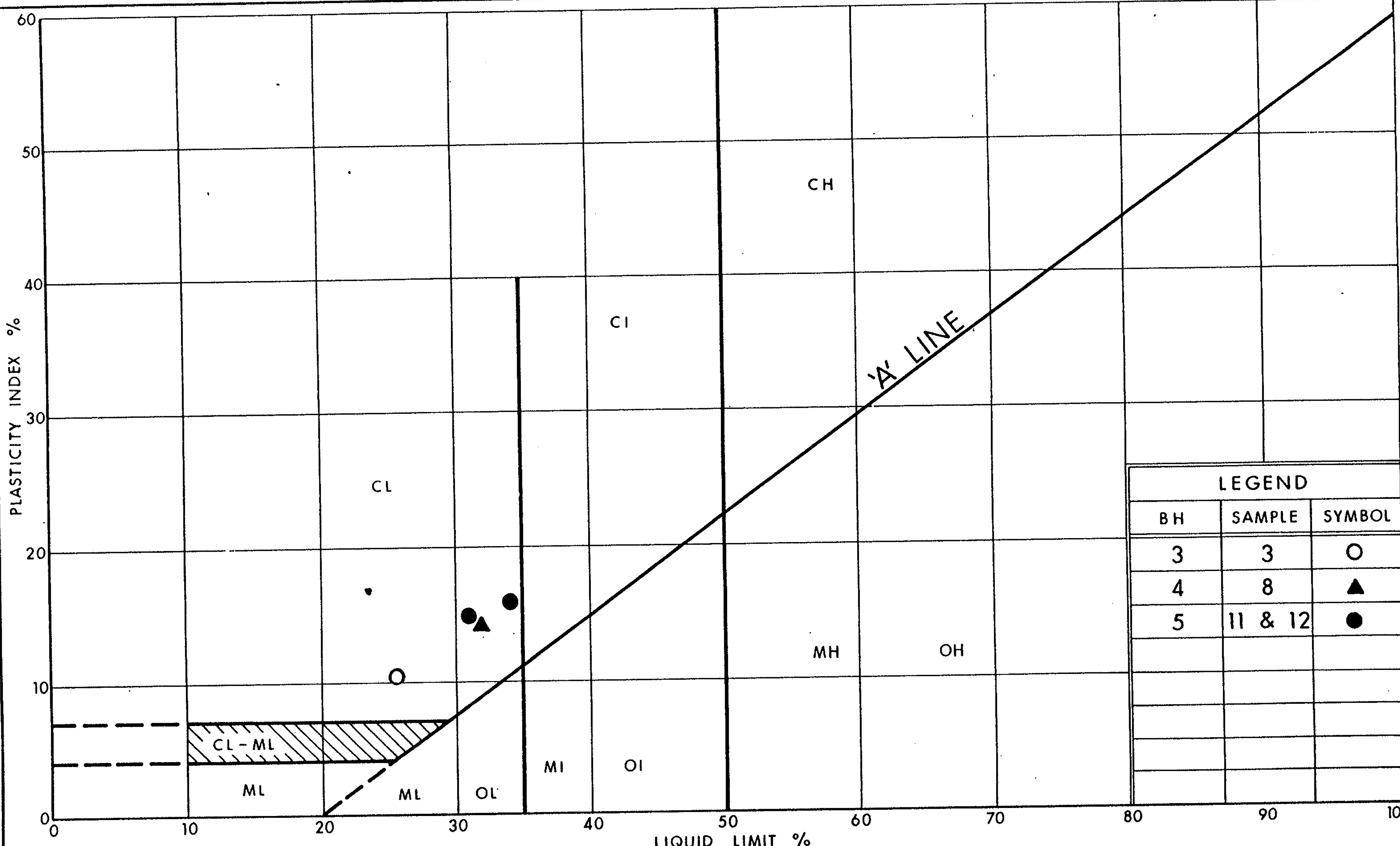
LEGEND		
BH	SAMPLE	SYMBOL
1	3 & 4	●
2	4	+
3	5	○
6	13	△
7	10	▲
8	13	□
9	11	■

Ministry of Transportation
Ontario

PLASTICITY CHART
SILTY CLAY

FIG No 3
W P 26-96-01

Oct 75, FF-S-21



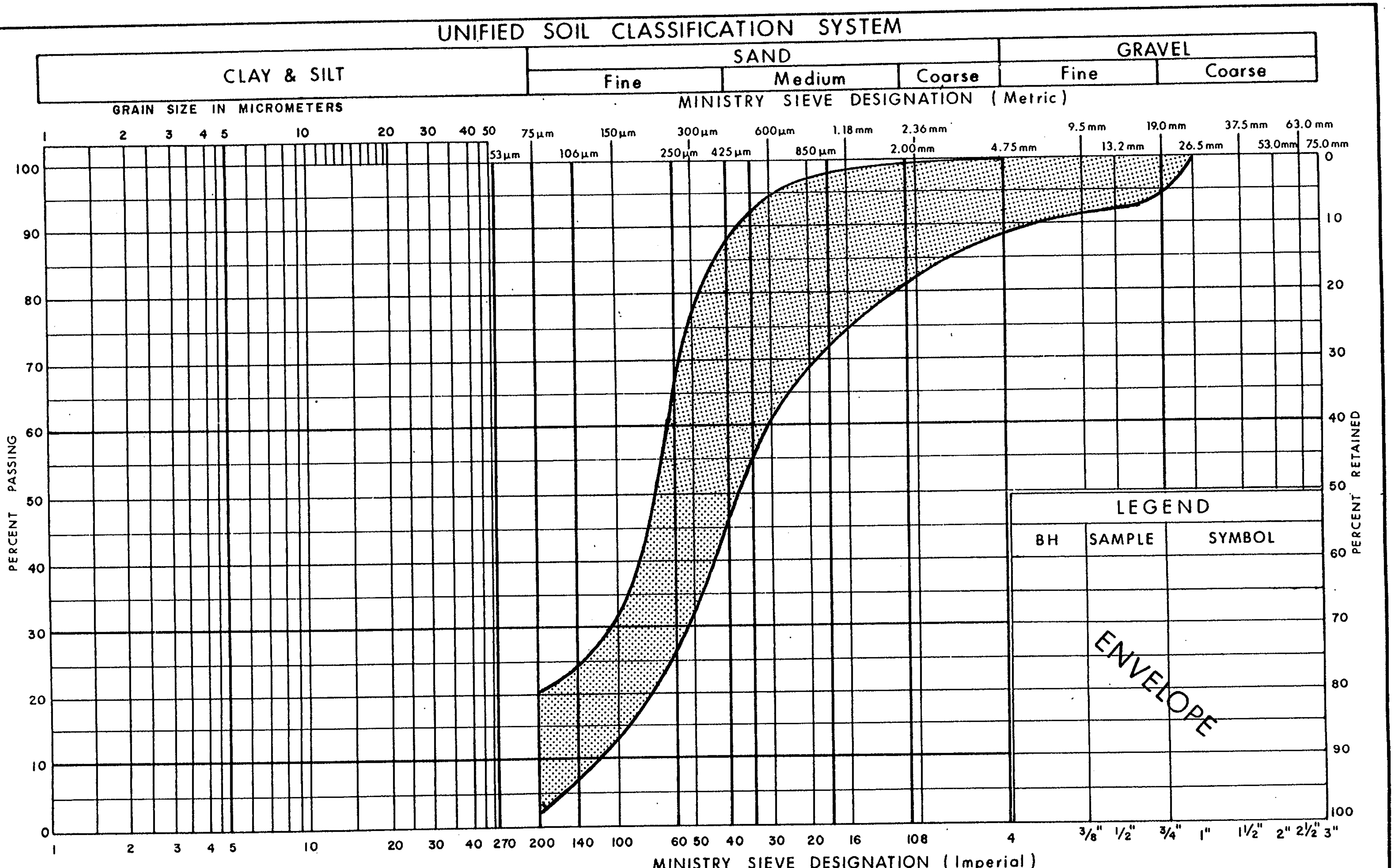
Ministry of Transportation
Ontario

PLASTICITY CHART

CLAYEY SILT

FIG No 4
WP 26-96-01

78 12 M



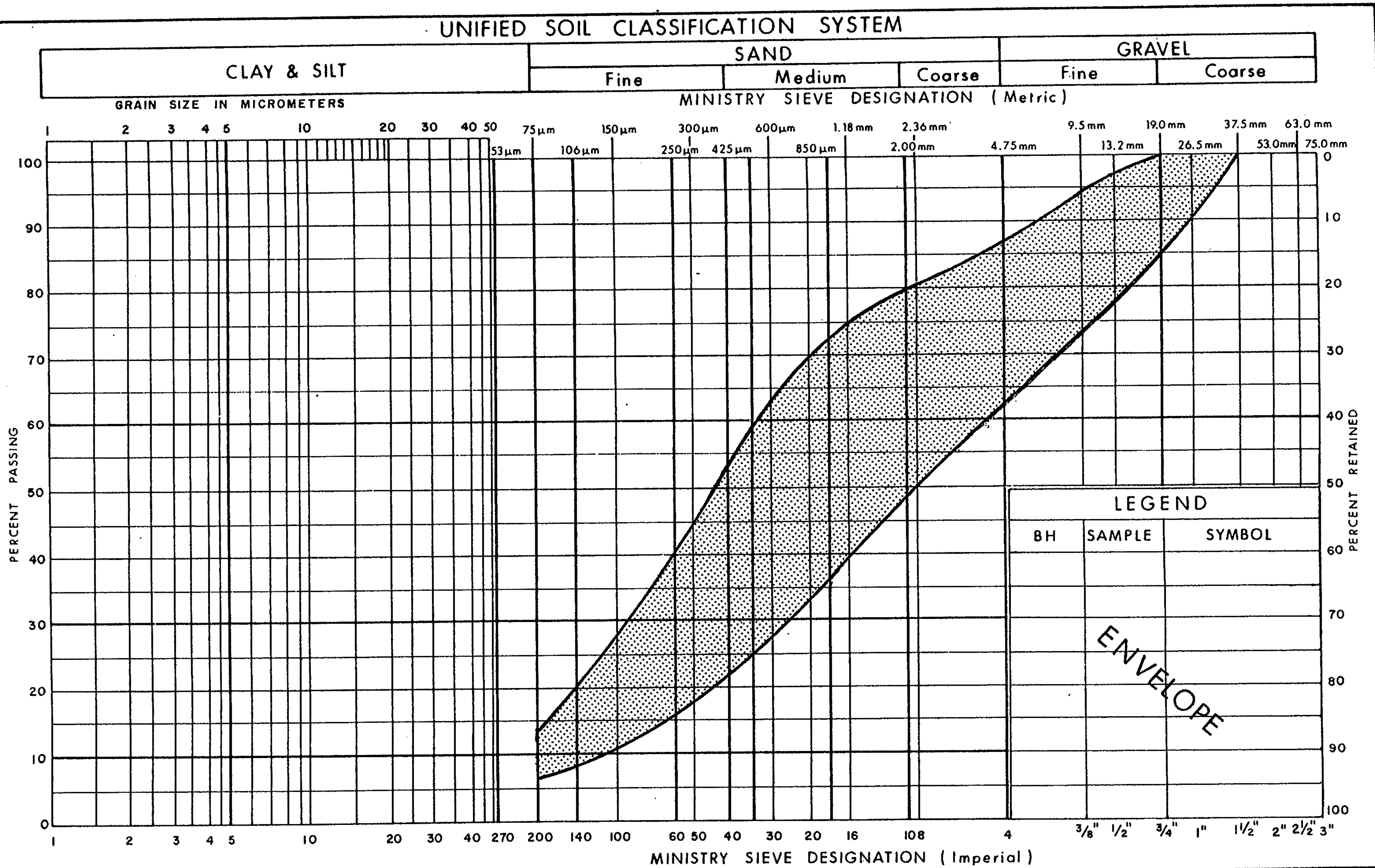
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY SAND TO SAND, TRACE OF GRAVEL

FIG No 5

W P 26-96-01

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
GRAVELLY SAND

FIG No 6

W P 26-96-01