

REPORT

Foundation Investigation and
Design Report
GWP 71-00-03
Highway 401
Horner Creek Bridge Widening
Site 23-117
Township of Blenheim, Ontario
London District

STANTEC CONSULTING LTD.

PROJECT NO. 1009213.01

GEOCRES NO. 40P2-67

PROJECT NO. 1009213

REPORT TO

**Stantec Consulting Ltd.
1400 Rymal Road East
Hamilton, Ontario
L8W 3N9**

FOR

**Foundation Investigation and Design
Report**

ON

**Horner Creek Bridge Widening
Site 23-117
Township of Blenheim, Ontario
London District
GWP 71-00-03
GEOCRES No. 40P2-67**

February 8, 2008

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FOUNDATION INVESTIGATION REPORT

**Horner Creek Bridge Widening
Site 23-117
Highway 401
Township of Blanford-Blenheim, Ontario
GWP 71-00-03
District – London**

1.0 INTRODUCTION

Jacques Whitford Limited (Jacques Whitford) was retained by Stantec Consulting Ltd. (Stantec), to complete a Foundation Investigation and Design Report for the proposed widening of the Horner Creek Bridge on Highway 401, GWP 71-00-03.

The work was carried out under Agreement No. 3005-E-0013 and in general accordance with our proposal dated September 1, 2005. Authorization to proceed with the investigation was provided by Mr. Dave Emery, P.Eng., Principal with Stantec Consulting Ltd. under a Subconsultant Agreement dated March 1, 2006.

The purpose of the geotechnical investigation was to determine the subsurface soil conditions at the east and west abutments, and at the approaches by advancing a total of 6 boreholes, and to provide a Foundation Investigation and Design Report.

This factual report has been prepared specifically and solely for the project described herein. It contains the factual results of the foundation investigation and laboratory testing.

2.0 BACKGROUND

Two previous investigations were carried out at this site as follows:

- Geotechnical Investigation by Racy, MacCallum & Associates Ltd., report dated September 30, 1958, Geocres No. 40P2-13.
- Geotechnical Memorandum by the Ministry of Transportation, Ontario, dated April 10, 1995, Geocres No. 40P2-055.

The subsurface conditions encountered in both investigation generally consisted of sandy silts, silty sand, and fine sand with varying amounts of gravel and occasional cobbles and boulders. An organic layer was reported in both investigations at a depth of about 2 m to 2.5 m below the original ground surface. Groundwater was reported at depths in the range of approximately 1 m to 2 m below original grade.



The factual Record of Borehole sheets from both investigations along with a borehole location plan for the 1958 investigation, are provided in **Appendix D** for reference.

3.0 SITE DESCRIPTION

The site location is a section of Highway 401 at Horner Creek in the Township of Blenheim, Ontario, as shown on the Key Plan on Drawing No. 1 in **Appendix A**. Representative photographs of the site are also provided in **Appendix A**.

Highway 401 in the area of Horner Creek is a four-lane road and carries east and west bound traffic. For the purpose of this report, Highway 401 will be considered to be oriented east-west with increasing chainage to the east. The Highway is generally built to a rural freeway section with partially paved shoulders, side ditches and a wide grass covered central median. The central median narrows at the bridge location.

The road profile is generally level and the pavement is typically constructed on embankments that are about 3 m to 4 m higher than the surrounding lands. The lands adjacent to the highway are generally undulating. Based on visual observations at the time of the investigation, the existing approach embankments have side slopes that range from approximately 2:1 (horizontal to vertical) near the bridge locations to about 3:1 away from the existing bridge structure. The side slopes are generally vegetated with field grasses and weeds, and do not show any signs of instability.

The width of the creek at the bridge location ranges from approximately 10 m to 15 m (across the water surface). At the time of this investigation the surface of the water was at an elevation of about 288.4 m, with the water in the creek flowing from north to south.

The Horner Creek Bridge was reportedly constructed in 1960 and rehabilitated in 1998. The existing structures are twin two-span reinforced concrete Tee beam rigid frame structures. Each span is approximately 14 m long and approximately 16 m wide. There is approximately 3.5 m between each structure.

4.0 PHYSIOGRAPHY

The project site is generally within an area identified by Chapman and Putnam (1984) as the Oxford Till Plain interlaced with glacial spillways. The till plain is characterized as pale brown calcareous loam and the glacial spillways are generally composed of uniform sandy and gravelly materials.

Physiographic mapping of the site indicates drumlinized till plains.

5.0 INVESTIGATION PROGRAM

5.1 Scope of Work

The scope of work for the investigation was as follows:

- To investigate the subsurface soil and groundwater conditions at six borehole locations;
- To conduct a laboratory testing program on representative soil samples obtained from the boreholes; and,
- To prepare a Foundation Investigation and Design Report.

5.2 Field Investigation Procedures

5.2.1 Borehole Investigation

Prior to commencing the field investigation, the borehole locations were established in the field by Jacques Whitford personnel. The borehole locations were cleared of underground utilities by the various public utility companies in the presence of a representative of Jacques Whitford.

Traffic control during loading/unloading of drilling equipment and during drilling on Highway 401 was provided by On Track Safety Limited (OTS), using signs, traffic barrels and blocker vehicles, in accordance with the Ontario Traffic Manual (OTM) Book 7 Temporary Conditions.

The fieldwork for the investigation was carried out between April 17 and April 20, 2006. A total of 6 boreholes (HC-06-01 to HC-06-06) were advanced by GeoEnvironmental Drilling and Aardvark Drilling Inc. at the approximate locations shown on Borehole Location Plan and Soil Strata Plot, Drawing No. 1 in **Appendix A**.

The boreholes for the foundations were advanced to depths in the range of approximately 22.4 m to 31.4 m, elevations of about 257.8 m to 266.9 m, using a track mounted power auger equipped with 250 mm (outside diameter), hollow-stem augers and rotary casing drilling equipment. The boreholes for the approaches were advanced to depths of approximately 9.8 m and 8.2 m below existing grade, elevations of about 281.4 m and 283.2 m in Borehole HC-06-05 and HC-06-06, respectively.

Soil samples were recovered from all boreholes at regular intervals using a 50 mm Outside Diameter split-tube sampler by conducting Standard Penetration Tests (SPTs) in general accordance with the procedures outlined in the ASTM specification D1586-99.

Jacques Whitford field personnel recorded the conditions encountered in all boreholes at the time of the investigation. Soils were described in accordance with the MTO Soils Classification System.

The groundwater levels, where encountered, were measured in the boreholes during drilling. All boreholes were backfilled with bentonite slurry in accordance with Regulation 903.

All soil samples recovered from the boreholes were placed in moisture-proof bags and returned to our laboratory for detailed classification and testing as required.



The subsurface conditions encountered in the boreholes are summarized on the Record of Borehole sheets in **Appendix B**. Additional comments are provided in the subsequent sections of this report.

5.3 Survey

The borehole locations were measured by Jacques Whitford personnel and referenced to the stations of the existing bridge abutments at the median centerline. Offsets were referenced looking up chainage. The elevations were interpreted based on a topographic plan provided by Stantec. It is understood that the topographic plan was referenced to Geodetic datum.

The ground surface elevations at the borehole locations are provided on the Record of Borehole sheets in **Appendix B**.

5.4 Laboratory Testing

All soil samples returned to the laboratory were subjected to detailed visual examination and classification. Approximately 25% of the soil samples were submitted for routine testing including grain size distribution, Atterberg Limits and moisture content determination tests. The results of the laboratory testing are shown on the Record of Borehole sheets in **Appendix B**. The results of the grain size analyses and Atterberg Limits tests are shown on Figure Nos. 1 through 7 in **Appendix C**.

Five representative soil samples were submitted to DBA Engineering Ltd. of Markham, Ontario, for analysis of Organic Content. These testing were conducted in accordance with ASTM D-2974-87 test method.

Unless requested in advance, all samples will be stored for a period of 12 months, after issuance of this report.

6.0 RESULT OF THE INVESTIGATION

6.1 Subsurface Conditions

The subsurface conditions encountered in the boreholes are summarized on the Record of Borehole sheets provided in **Appendix B**. An explanation of the symbols and terms used on the Record of Borehole sheets is also provided in **Appendix B**.

A summary of the soil and groundwater conditions encountered is provided below.

6.2 Soil

6.2.1 Topsoil

Topsoil was encountered at the ground surface in Borehole HC-06-01 and HC-06-04, and below the sand and gravel fill (discussed in Section 5.2.3 below) in Borehole HC-06-06. The thickness of topsoil ranged from approximately 100 mm to 400 mm.



Laboratory testing performed on representative samples consisted of an Organic Content Test and Moisture Content determination test. The tests gave the following results:

Organic Content:

- 4 %

Moisture Content:

- 23%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**.

6.2.2 Sand Fill

Brown sand fill was encountered below topsoil in Borehole HC-06-01 and at the ground surface in Borehole HC-06-02 and HC-06-03.

The sand fill contained variable amount of silt and organics and trace clay. The sand fill ranged in total thickness from approximately 0.9 m to 2.7 m and generally extended to depths of approximately 0.9 m to 3.0 m below existing grade, elevations of about 286.7 m to 288.4 m.

Laboratory testing performed on representative samples consisted of a Grain Size Distribution and Moisture Content tests. The tests gave the following results:

Gradation:

- 0% gravel;
- 67% sand;
- 23% silt; and,
- 10% clay size particles.

Moisture Content:

- 9% to 30%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**. The results of the Grain Size distribution tests are plotted on Figure No. 1 in **Appendix C**.

6.2.3 Sand and Gravel Fill

Brown sand and gravel fill was encountered below the topsoil in Borehole HC-06-04 and at the ground surface of Boreholes HC-06-05 and HC-06-06.

The sand and gravel fill contained trace silt and organics. The sand and gravel fill ranged in total thickness from approximately 0.5 m to 3.0 m and generally extended to depths of approximately 0.6 m to 3.0 m below existing grade, elevations of about 289.3m to 288.4 m.

Laboratory testing performed on representative samples consisted of Grain Size Distribution and Moisture Content determination tests. The tests gave the following results:

Gradation:

- 34% and 54% gravel;
- 59% and 41% sand; and,
- 7% and 5% silt and clay size particles.

Moisture Content:

- 4% to 9%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**. The results of the Grain Size distribution tests are plotted on Figure No. 2 in **Appendix C**.

6.2.4 Native Sand

Two strata of native sand were encountered in all the boreholes except Borehole HC-06-01 where a single stratum of sand was encountered at a depth of about 3 m below existing grade, elevation 286.7 m, was approximately 15.3 m thick, extending to an elevation of approximately 271.4 m. The upper and lower sand layers are described separately in the following sections.

6.2.4.1 Upper Native Sand (SP)

A stratum of native sand was encountered underlying the fill materials in all boreholes. The native sand was grey in Boreholes HC-06-04 and HC-06-05, and brown to dark brown in the remaining boreholes.

The native sand contained various amount of gravel and silt (trace to with) and some organics. Seams and lenses of peat and organics were encountered in the upper sand. The native sand ranged in total thickness from approximately 1.5 m to 2.8 m and generally extended to depths of approximately 3.2 m to 5.3 m below existing grade, elevations of about 285.7 m to 286.5 m.

Based on the N-values obtained from the SPTs, the compactness of the native sand is generally very loose to loose becoming compact with depth.

Laboratory testing performed on representative samples consisted of Grain Size Distribution, Organic Content, and Moisture Content tests. The tests gave the following results:

Gradation:

- 0% to 16% gravel;
- 72% to 86% sand; and,
- 12% to 14% silt and clay size particles.

Organic Content:

- 1.3% and 1.6%

Moisture Content:

- 15% to 33%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**. The results of the Grain Size distribution tests are plotted on Figure No. 3 in **Appendix C**. The sand can be classified as an SP material.

6.2.4.2 Lower Native Sand (SW to SP-SM)

A second layer of grey native sand soil (lower native sand) was encountered below the peat (discussed in Section 5.2.5 below) in Boreholes HC-06-02, HC-06-03, HC-06-05, and HC-06-06, and encountered below the gravel (discussed in Section 5.2.6 below) in Borehole HC-06-04.

The lower native sand was encountered at an approximate depth of 3.8 m to 7.3 m below existing grade, elevation of about 283.9 m to 285.5 m, and contained various amount of gravel and silt (trace to some). Boreholes HC-06-05 and HC-06-06 were terminated in the sand at depths of approximately 9.8 m and 8.2 m below existing grade, elevations of about 281.4 m and 283.2 m.

Based on the N-values obtained from the SPTs, the compactness of the lower native sand was variable ranging from very loose to very dense, but was typically compact to very dense.

Laboratory testing performed on representative samples consisted of Grain Size Distribution and Moisture Content determination tests. The tests gave the following results:

Gradation:

- 0% to 16% gravel;
- 60% to 97% sand; and,
- 2% to 24% silt and clay size particles.

Moisture Content:

- 3% to 22%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**. The results of the Grain Size distribution tests are plotted on Figure No. 4 in **Appendix C**. The sand can be classified as an SW to SP-SM material.

6.2.5 Peat

A layer of black peat was encountered below the upper native sand in all boreholes except Borehole HC-06-01.

The peat was encountered at depths in the range of approximately 3.2 m to 5.3 m below existing grade, elevations of about 285.7 m to 286.5 m. The peat ranged in total thickness from approximately 0.6 m to 2.3m and generally extended to depths of approximately 3.8 m to 7.3 m below existing grade, elevations of about 283.9 m to 285.8 m.

Laboratory testing performed on representative samples consisted of Organic Content and Moisture Content determination tests. The tests gave the following results:

Organic Content:

- 35% and 41 %



Moisture Content:

- 162% to 194%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**.

6.2.6 Sandy Silt (ML)

A deposit of native sandy silt was encountered in Borehole HC-06-01, HC-06-03, and HC-06-04 at depths of approximately 14.3 m to 18.3 m below existing grade, elevations of about 271.4 m to 275.6 m. Borehole HC-06-01 and HC-06-04 were terminated in the sandy silt at depth of approximately 27.9 m and 24.5 m below existing grade, elevations of about 261.8 m and 265.4 m, respectively. The sandy silt was grey in colour and contained various amount of gravel and clay (trace to some).

Based on the N-values obtained from the SPTs, the compactness of the sandy silt was considered to be compact to very dense.

Laboratory testing performed on representative samples consisted of Grain Size Distribution and Moisture Content determination tests. The tests gave the following results:

Gradation:

- 1% to 17% gravel;
- 29% to 39% sand;
- 41% to 60% silt; and,
- 5% to 13% clay size particles.

Moisture Content:

- 11% to 17%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**. The results of the Grain Size distribution tests are plotted on Figure No. 5 in **Appendix C**. The sandy silt can be classified as an ML material.

Atterberg Limits Testing was attempted on selected samples and the sandy silt was found to be non-plastic.

6.2.7 Clayey Silt (CL-ML)

A stratum of native clayey silt was encountered in Borehole HC-06-02 and HC-06-03 at depths of approximately 20.0 m and 26.1 m below existing grade, elevations of about 269.3 m and 263.1 m, respectively. Borehole HC-06-02 was terminated in the clayey silt at a depth of approximately 22.4 m below existing grade, an elevation of about 266.9 m. The clayey silt was grey in colour and contained some sand and trace gravel.

Based on the N-values obtained from the SPTs, the consistency of the clayey silt was considered to be hard.

Laboratory testing performed on representative samples consisted of Grain Size Distribution, Atterberg Limits, Unit Weight, and Moisture Content determination tests. The tests gave the following results:

Gradation:

- 0% gravel;
- 4% sand;
- 72% silt; and,
- 24% clay size particles.

Atterberg Limits:

- 15% Plastic Limit;
- 22% Liquid Limit; and,
- 7% Plasticity Index.

Unit Weight:

- 21 kN/m³ and 22 kN/m³

Moisture Content:

- 15% to 20%

The results of the laboratory testing are provided on the Record of Borehole sheets in **Appendix B**. The results of the Grain Size distribution and Atterberg Limits tests are plotted on Figure No. 6 and 7 in **Appendix C**. The clayey silt can be classified as a CL-ML material.

6.2.8 Silty Sand (SP)

A stratum of grey native silty sand was encountered in Borehole HC-06-03 at a depth of approximately 30.0 m below existing grade, an elevation of about 259.2 m. Borehole HC-06-03 was terminated in the silty sand at depth of approximately 31.4 m below existing grade, an elevation of about 257.8 m.

Based on the N-values obtained from the SPTs, the compactness of the silty sand was considered to be very dense.

The silty sand can be classified as an SM material.

6.3 Groundwater

The groundwater conditions recorded in the boreholes at the time of the field investigation are shown in Table 5.2 below:

Table 6-1 Groundwater Level Measurements

Borehole	Depth*(m) in Open Borehole	Geodetic Elevation (m)
HC-06-01	3.0	286.7
HC-06-02	3.2	286.1
HC-06-03	3.5	285.7

Borehole	Depth*(m) in Open Borehole	Geodetic Elevation (m)
HC-06-04	1.5	288.4
HC-06-05	7.6	283.6
HC-06-06	4.6	286.8

Notes: * Measured from the ground surface

The water level in the creek at the time of the investigation was at an elevation of approximately 288.4 m and was approximately 0.5 m to 1 m deep.

It is noted that the groundwater levels and creek levels reported above are those encountered at the time of the field investigation in April 2006. These levels will be subject to fluctuations due to seasonal effects and precipitation events.

7.0 CLOSURE

A soil investigation is a limited sampling of a site. The information is gathered at specific borehole locations and can only be extrapolated to an undefined limited area around the locations. The extent of the limited area depends on the variability of the soil and ground water conditions as influenced by geological processes, as well as the history of the site reflecting natural conditions, construction activities, and site use.

We trust the above information meets with your present requirements. Should you have any questions or require further information, please do not hesitate to contact us at your convenience.

Yours truly,

JACQUES WHITFORD LIMITED

Original Signed By:

Geoffrey Creer, P.Eng.
Geotechnical Engineer

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FOUNDATION DESIGN REPORT

**Horner Creek Bridge Widening
Site 23-117
Highway 401
Township of Blanford-Blenheim, Ontario
GWP 71-00-03
District – London**

8.0 DISCUSSION

8.1 General

Highway 401 in the area of the Horner Creek Bridge is a four-lane road carrying east and west bound traffic. For the purpose of this report, Highway 401 will be considered to be oriented east-west with increasing chainage to the east.

The Highway is generally built to a rural freeway section with partially paved shoulders, side ditches and a wide grass covered central median. The road profile is generally level and the pavement is typically constructed on embankments that are generally 3 m to 4 m higher than the surrounding lands. The lands adjacent to the highway are generally undulating.

The width of the creek at the bridge location ranges from approximately 10 m to 15 m (across the water surface). At the time of this investigation the surface of the water was at an elevation of about 288.4 m, with the water in the creek flowing from north to south.

Horner Creek Bridge was reportedly constructed in 1960, rehabilitated in 1998 and consists of twin structures, one to convey the east bound traffic and one to convey the west bound traffic over Horner Creek. The existing structures are 2 span reinforced concrete Tee beam rigid frame structures. Each span is approximately 14 m in length approximately 16 m wide. There is approximately 3.5 m between the two structures. The existing bridge structures are reportedly supported on 55 foot (16.8 m) long end bearing steel piles.

8.2 Proposed Development

The Ministry of Transportation (MTO) is planning to widen a section of Highway 401 just east of Woodstock, Ontario. The planned widening will extend from approximately 1 km east of Interchange No. 238 (Highway 401 and Oxford Road 2), in the Township of Blanford, to approximately 4.1 km east of the Drumbo Road underpass in the Township of Blanford-Blenheim. The total length of the planned widening will be approximately 15.3 km.

The widening consists of adding a single lane to both the east and west bound lanes of the highway. It is understood that the widening at the bridge location will be located between the existing bridge structures as well as to the north and south of the existing structures.



It is anticipated that the approach fills will be adjusted by filling in the median ditch (approximately 3.5 m wide and 1 m deep) and by widening the roadway platform by approximately 1.2 m to both the north and south.

The planned widening will be carried out with pre-cast, pre-stressed concrete hollow core slabs supported on extensions to the existing piers and abutments.

8.3 Subsurface Conditions

The subsurface conditions encountered at the location of the Horner Creek Bridge generally consists of approximately 3 m of embankment fill underlain by an upper layer of loose to compact native sand with seams of organic matter and peat. A stratum of peat was encountered in all boreholes except Borehole HC-06-01 at depths of approximately 3.2 m to 5.3 m below existing grade, elevations of about 285.7 m to 286.5 m. The peat was underlain by a second stratum of compact to very dense sand, which was underlain by either compact to very dense sandy silt / silty sand or hard clayey silt.

Groundwater was encountered in the boreholes at depths in the range of approximately 1.5 m to 7.6 m below existing grade or elevations ranging from approximately 283.6 m to 288.4 m. The water level in the creek was at elevation 288.4 m at the time of the investigation.

8.4 Foundation Assessment

The following table provides a summary of the foundation options under consideration:

Foundation Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Spread Footings supported on native sand	Lowest cost.	Deep excavations up to about 7.5 m below existing grade would be required to found below the peat soils. Shoring and dewatering would be required.	Low to medium	Differential movement between the existing structure and the widened area. Deep excavations would be required adjacent to existing structure. Dewatering could be challenging.
Spread footings on granular pads	Higher geotechnical resistance.	Deep excavations up to about 8.5 m would be required to place engineered fill below the peat soils. Shoring and dewatering would be required.	Medium to High	Differential movement between the existing structure and the widened area. Deep excavations would be required adjacent to existing structure. Dewatering could be challenging.
End bearing piles driven to the hard clayey silt and dense silty sand.	High geotechnical resistance. Minimal settlement.	Potential to encounter cobbles and boulders during driving.	High	Possible tip damage during driving which would require a driving shoe. Potential down drag forces on the piles may result from placement of embankment fill material.

Foundation Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Friction piles founded in the sand and sandy silt	Some cost savings, reduction in number of splices.	Lower geotechnical resistance Differential settlement between the existing structure and widening, given that the piles for the existing structure were reportedly driven to the underlying hard/dense strata.	High	Piles likely spaced closer together, which would result in reduction of capacity for pile groups.

Based on the conditions encountered in the boreholes, the site is generally not suitable for the use of spread footings due to the organics encountered in the upper sands and the presence of the peat soils encountered at depth.

It is recommended that the abutment and pier extensions be supported on driven end-bearing piles to be consistent with the foundation for the existing bridge structures and alleviate potential construction challenges.

9.0 RECOMMENDATIONS

9.1 Pile Foundations

9.1.1 Geotechnical Resistance

Given the conditions encountered during this investigation, it is recommended to found the bridge widening on end bearing piles driven to the underlying very dense and hard stratum. This option would be consistent with the foundations for the existing structures that are reportedly supported on 16.8 m long (55 foot long) steel end bearing piles with anticipated tip elevations of approximately 268.5 m.

The abutment and pier extensions could be founded on HP310x110 piles driven to the underlying very dense and hard stratum at depths in the range of approximately 21 m to 28 m below existing grade, elevations of about 262 m on the west side to 268 m on the east side of the creek.

HP310 x 110 Steel H-Piles for the abutments and piers driven to the underlying hard and very dense strata may be designed using a factored geotechnical resistance at ULS of 1800 kN. The ULS value includes a resistance factor of 0.4.

A geotechnical reaction at SLS of 1600 kN is recommended for piles founded on the hard and very dense strata. The toe of the piles is expected to settle less than 20 mm at the SLS value. It is noted that the structural engineer will need to evaluate the elastic compression of the pile.

9.1.2 Down Drag Forces

It is understood that the widening will require about 1 m of fill in the central median and about 0.5 m of fill on the sides of the existing embankments.

The placement of the fill material will induce settlement of the underlying organic soils. Down drag forces, induced as a result of the settlement of the underlying soils under the approach fill, must be considered.

Based on the settlement estimates provided in Section 9.4, the unfactored negative skin friction calculated for an HP310X110 steel H-pile supporting the abutments is approximately 150 kN. The calculation for the negative skin friction was applied to the perimeter of a theoretical box around the pile and presumes that 10 mm of relative movement between the pile and surrounding soil is required to mobilize the down drag forces. Based on the settlement profile, 10 mm of relative movement is calculated to occur near the underside of the peat layer at an elevation of about 284 m to 286 m geodetic.

Down drag forces are not anticipated at the pier locations as it is presumed that the grades at these locations will remain unchanged.

It is noted that the original construction of the embankments would have imposed down drag loads on the existing abutment piles. Although the additional embankment load may induce a minor amount of settlement around them, there will likely be no down drag loads to the existing piles in addition to the current situation.

9.1.3 Lateral Forces

Lateral forces could be partially or fully resisted using battered piles. Where battered piles are not used, passive lateral resistance for vertical piles should be calculated as per C6.8.7.2 (Static Analysis i.e., Brom's method) of the CHBDC using the following unfactored geotechnical soil parameters:

Parameter	Native Sands and Silts	Clayey Silt
Bulk Unit Weight (kN/m ³)	19	21
Friction Angle	30°	-
Coefficient of passive earth pressure	3.0	-
Design Undrained Shear Strength (kPa)	-	100

An assessed horizontal passive resistance at ULS of 110 kN is recommended for the HP310x110 piles at this site, based on Table C6.4 of the CHBDC.

Lateral Deflections

The coefficient of horizontal subgrade reaction that is used for deflection calculation for non-cohesive soils may be estimated as follows:

$$k_s = n_h(z/d)$$

Where k_s = the coefficient of horizontal subgrade reaction (force per volume)

n_h = Co-efficient related to soil density. This may be taken as 4,400 kN/m³ for compact to loose sandy soils below the ground water (Table 20.3, p. 315, of the Canadian Foundation Engineering Manual, 3rd edition, 1992).

z = depth below grade

d = pile diameter

The coefficient of horizontal subgrade reaction that is used for deflection calculations may be estimated for cohesive soils as follows:

$$k_s = 67 C_u/d$$

Where k_s = the coefficient of horizontal subgrade reaction (force per volume)

C_u = undrained shear strength of the soil = 100 kPa for this application

d = pile diameter

An assessed horizontal geotechnical reaction at SLS of 40 kN is recommended for the HP310x110 piles at this site, based on Table C6.4 of the CHBDC.

9.1.4 Group Effects on Lateral Deflections

If piles are spaced at less than 8 pile diameters, center to center, parallel to the direction of lateral load, or less than 4 pile diameters, center to center, perpendicular to the lateral load, group effects will need to be considered and the lateral load at a specific deflection may need to be decreased.

The nature of pile-soil-pile interaction is complex, however is generally broken down into the following main components:

- Alteration of the soil state due to pile installation and the potential overlap of the alterations when nearby piles are driven; and,
- Superposition of strains and alterations of the soil failure zones when nearby piles are simultaneously loaded.

Studies (Reese, Isenhower and Wang, 2006) have reported the following reduction between single piles and pile groups.

Condition No. 1: Load is parallel to pile spacing

Pile Spacing c/c	Trailing Pile Group Pile Efficiency, e_T	Lead Pile Group Pile Efficiency, e_L
7d	1.0	1.0
4d	0.8	1.0
3d	0.7	0.9
2d	0.6	0.8

Condition No. 2: Load is perpendicular to pile spacing

Pile Spacing c/c	Group Pile Efficiency, e_P
4d	1.0
3d	0.9
2d	0.75



Where piles are on a skew to each other relative to the direction of load the Group Pile Efficiency may be calculated based on

$$e_s = (e_B^2 \cos^2 \alpha + e_p^2 \sin^2 \alpha)^{1/2}$$

where

e_B = either e_T or e_L from above

α = angle between direction of loading and the skew

Note that when piles are more than 3.3 pile diameters apart perpendicular to the direction of the load, the skew correction is not necessary. The lateral load at a specific deflection for each individual pile must consider the interaction of all piles within the group.

The reduction factor applied to a pile is the product of the efficiencies of all of the interactions of piles within that pile group.

9.1.5 Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with Section 6.8.5 of the CHBDC. The following factored ULS values may be used for design of piles in tension:

Pile Type	Pile Length (m)	Pile size	Factored ULS Tensile Resistance (kN)
H-Piles	20	HP310X110	350
	28	HP310X110	450

The values provide above include a ULS resistance factor of 0.3, but do not include the weight of the pile.

9.1.6 Piling Notes

Steel H-piles should be equipped with Type I reinforced flanges as per OPSD 3000.100.

The piles are anticipated to be approximately 20 m to 24 m in length, which will require the piles to be spliced during driving. Welded splices for steel H-piles should be in accordance with OPSD 3000.150.

Piles should be supplied and installed in accordance with SP903S01. The piles should be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance equal to twice the factored design load at ULS.

The following note should be added to the pile foundation drawings:

“Piles to be driven in accordance with standard SS 103-11 using an ultimate geotechnical resistance of 3600 kN per pile, but must be driven below elevation 262 m at the west abutment, 265 at the pier and 268 at the east abutment.”

9.2 Earth Pressure Design

To prevent hydrostatic pressure build-up, backfill against the abutments should consist of free draining granular materials. OPSS Granular A or OPSS Granular B, Type II both with less than 5% fines passing the 75µm sieve is recommended. The zone of granular backfill must be constructed in accordance with OPSD 3101.150, using a frost penetration depth, f , of 1.3 m. A subdrain should be installed as per OPSD 3102.100.

Earth pressure coefficients are provided below for different backslope conditions. In order to use the coefficients of pressure for a particular granular material, the granular backfill must be provided within a wedge extending from the base of the abutment at 45° (or smaller) to the horizontal. If a smaller wedge is used, the coefficients of earth pressure of the materials outside the backfill wedge must be used for lateral pressure design calculations.

For rigidly tied structures (e.g. bridge abutments), the at-rest pressure should be used for design, unless the wall can deflect enough (approximately 0.05% of the wall height) to establish the active pressure. The effect of compaction should be accounted for as per CHBDC Figure 6.6.

Lateral earth pressures may be calculated using the parameters in the following table:

Parameters	OPSS Granular A & OPSS Granular B Type II	OPSS Granular B, Type I & III	Native Sand and Sand Fill
Unit Weight (kN/m ³)	22	21	19
Angle of Internal Friction, ϕ	35°	32°	30°
Horizontal Backslope			
Coefficient of Active Earth Pressure, K_a	0.27	0.31	0.33
Coefficient of Passive Earth Pressure, K_p	3.69	3.25	3.0
Coefficient of Earth Pressure at Rest, K_o	0.43	0.47	0.5
2H:1V backslope			
Coefficient of Active Earth Pressure, K_a	0.39	0.47	0.54
Coefficient of Passive Earth Pressure, K_p	10.82	8.61	7.46

9.3 Seismic Design

9.3.1 Seismic Forces and Soil Profile Type

The zonal acceleration ratio for the Woodstock area, which is approximately 8 km west of the bridge structure, is 0.05 as per Table A3.1.7 of the CHBDC.

It is recommended that Soil Profile I as defined in the CHBDC Section 4.4.6 be used in the seismic design of this site.

9.3.2 Seismic Forces on Abutments and Retaining Walls

Abutments and retaining walls should be designed to resist the earth pressures produced under earthquake conditions. CHBDC Clause 4.6.4 recommends the use of the combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions, for routine design purposes.

The total active and passive thrusts under earthquake conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where;

K_{AE} = active earth pressure coefficient (combined static and seismic)

K_{PE} = passive earth pressure coefficient (combined static and seismic)

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight

For this site, the following preliminary design parameters were used to develop the recommended K_{AE} and K_{PE} values.

- Zonal Acceleration Ratio, A 0.05
- Horizontal Acceleration Coefficient, k_h 0.025
- Vertical Acceleration Coefficient, k_v 0.017
- Vertical back of wall
- For yielding abutments or walls

The above k_h value corresponds to $\frac{1}{2}$ of the A value, and the k_v value corresponds to 0.67 of the k_h value. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 9-1: Combined Coefficients of Static and Seismic Earth Pressure

Parameter	OPSS Granular A & Granular B Type II	OPSS Granular B Type I & Granular B Type III	Native Sand and Sand Fill
	Horizontal Backslope	Horizontal Backslope	Horizontal Backslope
Total Unit Weight, γ (kN/m ³)	22	21	19
Effective Friction Angle	35 degrees	32 degrees	30 degrees
Angle of Internal Friction between wall and backfill	0 degrees	0 degrees	0 degrees
Active Earth Pressure (K_{AE})	0.28	0.32	0.33
Height of application of P_{AE} from base as ratio of wall height (H)	0.342	0.341	0.340
Passive Earth Pressure (K_{PE})	3.64	3.21	3.0
Height of application of P_{PE} from base as ratio of wall height (H)	0.325	0.325	0.325

It is noted that the combined coefficients of static and seismic earth pressure deviate only slightly from the static coefficients presented above. This is due to the low zonal acceleration ratio at this site.

9.3.3 Liquefaction

An assessment of the potential for the liquefaction of the foundation soils was carried out using the Seed and Idriss (1971) simplified procedure (CHBDC C4.6.2). The results indicate that liquefaction is not a concern at this site.

9.4 Embankment Design and Construction

The existing embankments are generally constructed at a 2:1 (horizontal:vertical) side slopes and do not exhibit signs of instability.

The placement of the widening fill will induce settlement of the underlying peat soils. The placement of approximately 1 m of fill in the central median will induce approximately 20 mm to 30 mm of settlement of the organic peat soils at the median. A similar amount of settlement is anticipated across the existing inside shoulders.

The placement of the fill soils on the widening to the north and south of the existing embankments will likely induce settlement of the peat soils of as much as 150 mm at the existing shoulder rounding.

It is recommended that the fill be placed as soon as practical to allow the majority of the settlement to occur before the bridge widening is constructed. Calculations indicate that about 90% of the settlement will take place within the first 3 to 4 months after the construction of the embankments.

Prior to placing the additional embankment fill for the planned widening, all topsoil, loose, wet, organic and other deleterious material should be removed from the area of the proposed embankment.

Site preparation should be carried out in accordance with the requirements of SP902S01.

The new embankment fill material should be benched into the existing embankments in accordance with OPSD 208.010. Mid-height benches are generally recommended for embankments that are 8 m or higher. Given that the existing embankments are 3 m to 4 m high, mid-slope embankments are not anticipated.

10.0 CONSTRUCTION RECOMMENDATIONS

10.1 Open Cut Excavations

Earth excavation, if required, should be carried out in accordance with SP206S03. Side slopes for open cut excavations should conform to the requirements of the Occupational Health and Safety Act and Regulations for Construction Projects current at the time of construction.

In accordance with the present act, the existing fill and any excavations in the native soils below the anticipated water level should be considered Type 3 soils. Temporary excavations should be made with side slopes no steeper than 1:1 (horizontal:vertical) from the base of the excavation.

Pile caps for the extensions to the existing piers and abutments should be set at the same elevation as the existing pile caps to ensure excavations do not undermine the existing pile caps.

The construction should be subject to time constraints such that temporary excavations are open for no longer than 10 calendar days. Flatter side slopes will be required for open cut excavations in loose sand deposits below the water line unless appropriate dewatering methods are employed.

Excavation side slopes should be protected from erosion and should be inspected regularly for signs of instability. Slopes should be flattened as required to maintain safe working conditions.

10.2 Staging

It is understood that the widening will be constructed in 3 stages and that two lanes of traffic in both directions will be maintained at all times. Stage 1 will consist of moving the east bound traffic to the outside (south side of the existing structure) and constructing the median widening. Stage 2 will consist of shifting the east bound traffic to the median and constructing the south widening. Stage 3 will consist of shifting both the east and west bound lanes to the south side and constructing the north side widening.

10.3 Shoring

Shoring may be required to support Highway 401 during the installation of the piles and pile caps at the abutment and pier locations. In addition shoring or cut off walls may be required when working adjacent to the existing creek.

It is recommended that shoring above the water levels could consist of soldier piles and lagging. The soldier piles will be installed into the underlying sand and sandy silt deposits. Shoring below the water levels and cut off walls should consist of steel sheet piles.

Cobbles and boulders were inferred at some borehole locations and their influence should be considered during shoring design.

The temporary shoring may be designed using the lateral earth pressure parameters provided in Section 9.2, entitled Earth Pressure Design.

Shoring should meet the requirements of Performance Level 2 as per SP105S19 for Protection Systems.

10.4 Groundwater Control

Ground water was encountered during drilling at depths ranging from approximately 1.5 m to 7.6 m below existing grade, elevations ranging from about 283.6 m to 288.4 m. Ground water was more typically encountered at depths in the range of 3.0 m to 4.6 m below existing grade, elevations of about 285.7 m to 286.8 m.

It is noted that the water level in Horner Creek at the time of the investigation was at an elevation of about 288.4 m. It is anticipated that excavations above these levels will likely encounter seepage from perched water, which should be readily handled by sumps and contractors pumps. However, excavations below the elevations noted above may be difficult given the presence of wet sands and the adjacent creek. Therefore, earthwork required below the water levels noted should be carried out in the wet, and backfilled with clear stone. If excavations are to be carried out dry, dewatering, such as well points and cut off walls, will likely be required, however, this will be difficult. It is suggested that consideration be given to placing a working mat such as a concrete mud mat on the base of the excavation to help minimize disturbance to the exposed soils, and assist with groundwater control.

An NSSP alerting the contractor to this issue should be included in the contract documents.

The following table provides an estimate of the co-efficient of permeability, based on the grain size distribution tests:

Soil Type	Estimated Co-Efficient of Permeability (cm/sec)
Fill (sand and sand and gravel)	10^{-2} to 10^{-4}
Native Sand (SW to SP-SM)	10^{-1} to 10^{-3}
Sandy Silt (ML)	10^{-5} to 10^{-6}

10.5 Erosion and Sediment Control, and Scour Protection

Soil conditions encountered at the bottom of the creek consisted of sand and gravel fill and native sand. The following table outlines the K factor for the various soils encountered at the site based on the grain size distribution curves for the soils and the Wishmeier Nomograph:

Material type	K Factor	Scourability and Erosion Potential
Sand Fill	0.25	Slight
Sand and Gravel Fill	0.15	Very slight
Native Sand	0.15 – 0.25	Very slight to slight

10.5.1 Erosion and Sediment Control

Erosion and sediment control during construction may consist of silt fences and erosion control blankets. These should be provided by the contractor, as required, throughout the construction period to prevent runoff from entering the water course.

Erosion control and drainage measures will be required to ensure the long-term stability of the new and re-instated embankment slopes. The permanent slopes should be protected from erosion by placement of seed and mulch over topsoil in accordance with OPSS 572.

The vegetative slopes should be established as soon as possible after the completion of the embankments. Maintenance will be required over the first few years until the vegetated cover is well established.

10.5.2 Scour Protection

To minimize scour of the creek bed, it is recommended that adequate rip rap and geotextile scour protection be provided along the creek base, consistent with creek flow and velocity.

Scour protection could consist of a non-woven geotextile placed on the graded ground surface and covered with a minimum 300 mm thick layer of rip rap used for high velocities, and outlined in OPSD 810.010. The protected area should extend a minimum of 6 m beyond the structure and extend vertically on the embankments to about 0.5 m above the normal spring flow level.

10.6 Frost Protection

The site is location in an area with a mean freezing index of between 500 and 750 Degree days (°Days), (Canadian Foundation Engineering Manual 1992). Based on Figure 3.4 of the MTO Pavement Design and Rehabilitation Manual, the frost penetration depth for this area is approximately 1.3 m.

10.7 Construction Monitoring

It is recommended that the existing bridge structure be monitored to ensure that the construction of the bridge widening does not adversely impact the existing structure. The monitoring program should include a visual assessment of the existing structure on a regular basis, along with vibration monitoring during the installation of the piles for the planned widening.

11.0 CLOSURE

Use of this report is subject to the Statement of General Conditions attached. It is the responsibility of Stantec and the Ministry of Transportation Ontario, who are identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Jacques Whitford Limited should any these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report;
- Basis of the report;
- Standard of care;
- Interpretation of site conditions;
- Varying or unexpected site conditions; and,
- Planning, design or construction.

Yours truly,

JACQUES WHITFORD LIMITED

Original Signed By:

Geoffrey Creer, P.Eng.
Geotechnical Engineer

Original Signed By:

Fred J. Griffiths, Ph.D., P.Eng.
Designated Principal MTO Foundation Contact

GC/FG/aek

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STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Jacques Whitford Limited and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Jacques Whitford's present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Jacques Whitford is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

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INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Jacques Whitford at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

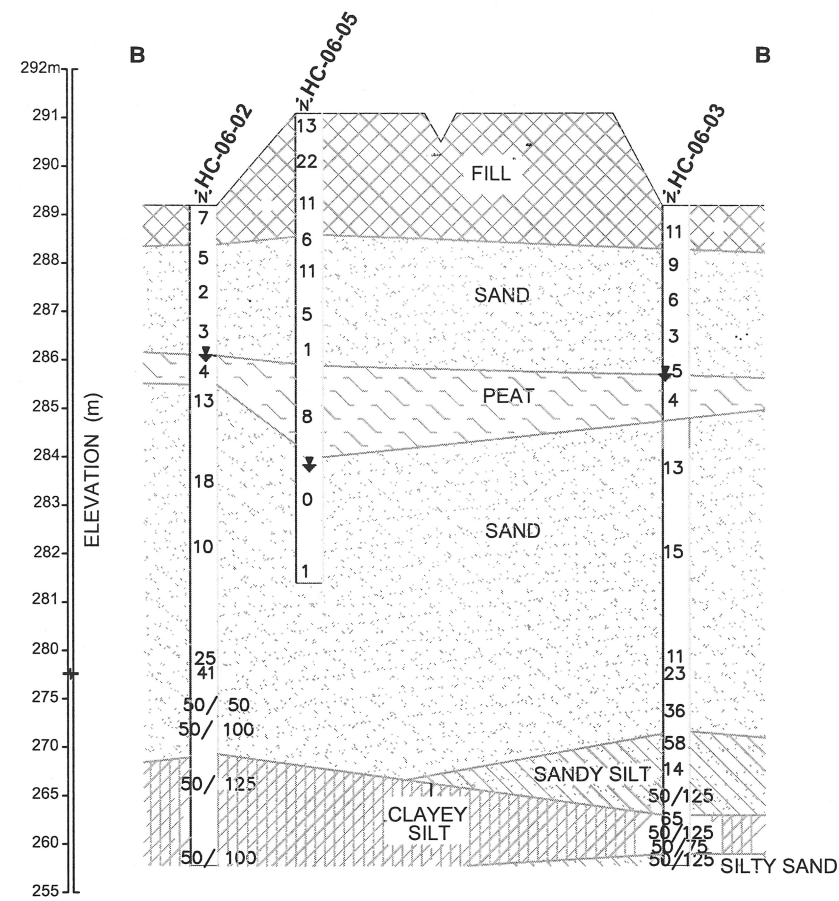
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Jacques Whitford must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Jacques Whitford will not be responsible to any party for damages incurred as a result of failing to notify Jacques Whitford that differing site or sub-surface conditions are present upon becoming aware of such conditions.


PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Jacques Whitford, sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Jacques Whitford cannot be responsible for site work carried out without being present.

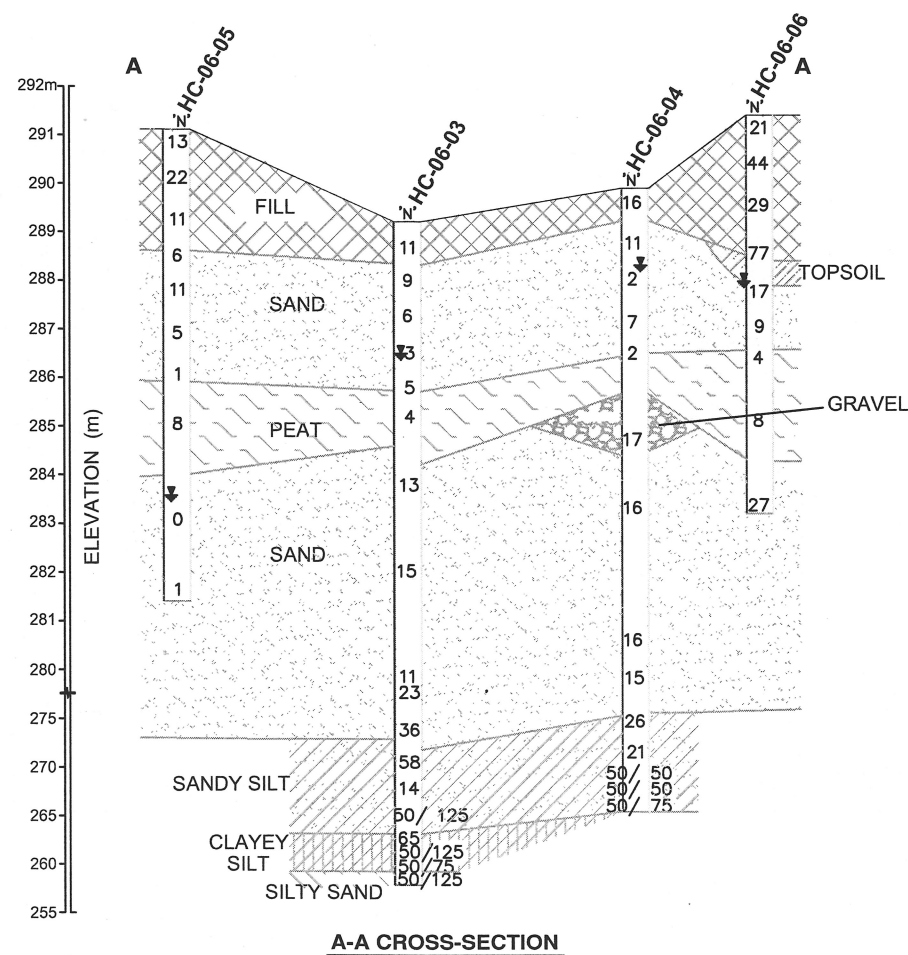
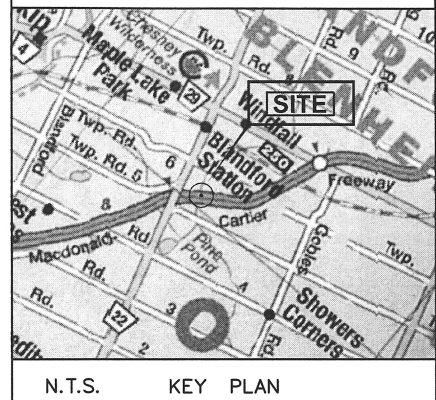


APPENDIX A

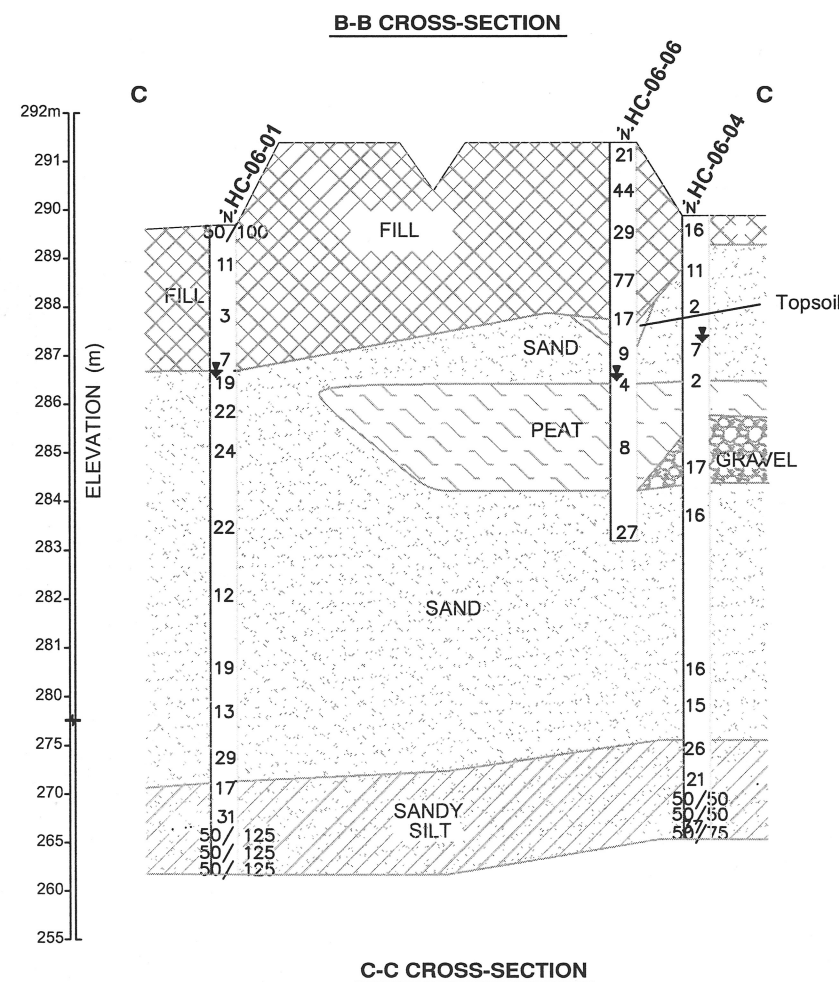
Drawings



HWY 401 CONT No— WP No - 71-00-00	 TRUE NORTH
HORNER CREEK BRIDGE WIDENING SITE 23-117 HIGHWAY 401 WOODSTOCK	



A-A CROSS-SECTION



C-C CROSS-SECTION

LEGEND			
☉	Bore Hole		
N	Blows/0.3m (Std Pen Test, 475 J/blow)		
↓	WL at time of investigation 04 03		

No	ELEVATION (m)	STATION	OFFSET
HC-06-01	289.7	10+645	24 LT
HC-06-02	289.3	10+620	22 LT
HC-06-03	289.2	10+620	27 RT
HC-06-04	289.9	10+645	26 RT
HC-06-05	291.2	10+595	17 LT
HC-06-06	291.4	10+860	17 RT

=NOTE=

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

NOTE: 1) The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions.

2) Base plan provided by Stantec Consulting Ltd.

3) This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.

REVISIONS	DATE	BY	DESCRIPTION
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GEOQUES No 40F2 - 67

HWY No 401		DIST
SUBM'D GC	CHECKED	DATE 2008-02-08
DRAWN HZ OL	CHECKED	APPROVED
		DWG 100921301.GEO-03A

APPENDIX B

Symbols and Terms Used on the Borehole and Test Pit Records
Record of Borehole Sheets

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Topsoil</i>	-	mixture of soil and humus capable of supporting good vegetative growth
<i>Peat</i>	-	fibrous fragments of visible and invisible decayed organic matter
<i>Till</i>	-	unstratified and unsorted glacial deposit which may include particle sizes from clay to boulders
<i>Fill</i>	-	materials not identified as deposited by natural geological processes

Terminology describing soil structure:

<i>Desiccated</i>	-	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	-	material breaks along plane of fracture
<i>Varved</i>	-	composed of regular alternating layers of silt and clay
<i>Stratified</i>	-	alternating layers or beds greater than 6mm (1/4") thick
<i>Laminated</i>	-	alternating layers or beds less than 6mm (1/4") thick
<i>Blocky</i>	-	material can be broken into small and hard angular lumps
<i>Lensed</i>	-	irregular shaped pockets of soil with differing textures
<i>Seam</i>	-	a thin, confined layer of soil having different particle size, texture, or color from materials above and below
<i>Well Graded</i>	-	having wide range in grain sizes and substantial amounts of all intermediate particles sizes
<i>Uniformly Graded</i>	-	predominantly one grain size

Soil descriptions and classification are based on the Unified Soil Classification System (USCS) (ASTM D-2488), which classifies soils on the basis of engineering properties. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. This system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm.

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with the standard of the Ministry of Transportation of Ontario:

<i>Trace or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>With</i>	20-30%

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N'-value*.

Compactness	'N'-value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

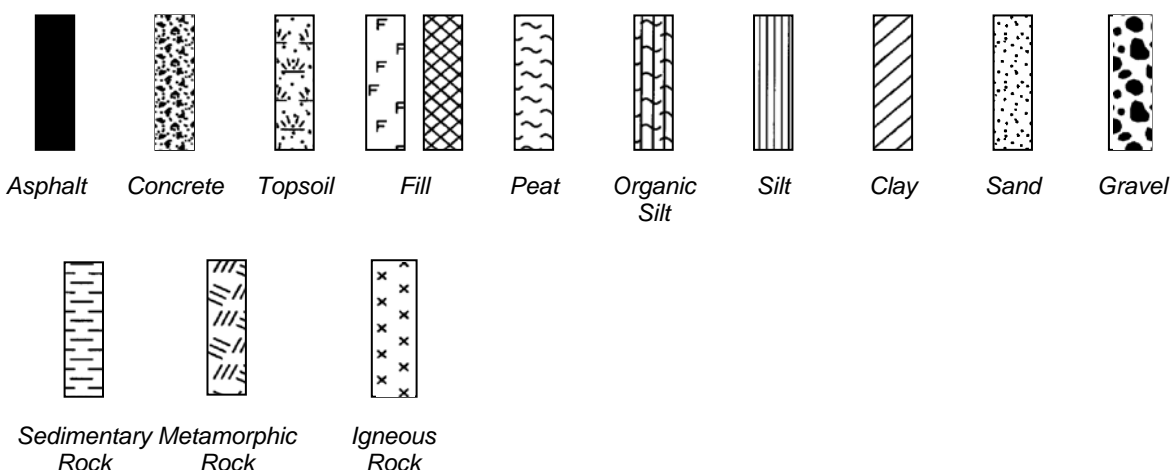
The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis. Standard Penetration Test 'N'-values* can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils.

Consistency	Undrained Shear Strength (kPa)	'N'-Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: **N'-VALUE- The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in millimeters (e.g. 50/75).

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

SAMPLE TYPE

SS	Split spoon sample (obtained from the Standard Penetration Test)	BS	Bulk sample
TW	Thin Wall Sample or Shelby Tube	WS	Wash sample
PS	Piston sample	HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits.
GS	Grab sample		
AS	Auger sample		
VT	Vane Test		

RECORD OF BOREHOLE No HC-06-01

1 OF 2

METRIC

W.P. 71-00-00 LOCATION Stn: 10+645 o/s: 24 m Lt, Twp of Blenheim ORIGINATED BY JL
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Mudrotary Casing, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.17.06 - 4.18.06 CHECKED BY GC

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								
289.7	Grass		1	SS	50/ 100 mm		20	40	60	80	100					
0.0 289.4	300 mm TOPSOIL, wet															
0.3	Silty SAND, some silt, some organics, trace rootlets, moist Compact to loose Dark brown (FILL)		2	SS	11											
	- with silt, trace clay loose		3	SS	3											0 67 23 10
			4	SS	7											
286.7																
3.0	SAND (SP), with gravel, trace silt, wet Compact Grey		5	SS	19											
	- some gravel		6	SS	22											11 84 (5)
			7	SS	24											
	- some silt, trace gravel, saturated		8	SS	22											
			9	SS	12											
			10	SS	19											
			11	SS	13											0 81 (19)

Continued Next Page

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

METRIC






✖³, ✕³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No HC-06-02

1 OF 2

METRIC

W.P. 71-00-00 LOCATION Stn: 10+620 o/s: 22 m Lt, Twp of Blenheim ORIGINATED BY JP
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Mudrotary Casing, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.19.06 - 4.19.06 CHECKED BY GC

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								
289.3	Grass						20	40	60	80	100					GR SA SI CL
0.0	SAND (FILL), some silt, trace gravel, with organics, trace rootlets, saturated Loose Brown		1	SS	7		289									Organic Content: 1.6%
288.4							288									
0.9	SAND (SP), some silt, with organics, wet Very loose to loose Dark brown - saturated, trace organics		2	SS	5		287									
	- peat lenses and seams		3	SS	2											
			4	SS	3											
286.1			5a	SS			286									
3.2	PEAT, wet Loose Black		5b	SS	4											
285.5																
3.8	SAND (SP), some silt, saturated Compact to very dense Grey		6	SS	13		285									
	- some gravel		7	SS	18		284									
	- casing grinding on possible cobble or boulder, damp		8	SS	10	283										
	- wet		9	SS	25	282										
	- saturated dense		10	SS	41	281										
						280										
						279										
						278										
						277										
						276										
						275										

Continued Next Page

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

2 OF 2

METRIC

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

RECORD OF BOREHOLE No HC-06-03

1 OF 3

METRIC

W.P. 71-00-00 LOCATION Stn: 10+620 o/s: 27 m Rt, Twp of Blenheim ORIGINATED BY JP
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Mudrotary Casing, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.20.06 - 4.20.06 CHECKED BY GC

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						✕ LAB VANE	WATER CONTENT (%)
289.2	Grass						20	40	60	80	100						
0.0	SAND (FILL), some silt, some gravel, with organics, wet Compact Dark brown		1	SS	11	▽	289										
288.3							288										
0.9	SAND (SP), trace organics, wet Loose to very loose Brown		2	SS	9												
	- peat lenses and seams, trace clay, saturated		3	SS	6												
	- trace silt, trace organics very loose		4	SS	3												
	- with organics Loose		5	SS	5			286									
285.7	PEAT, wet Loose to compact Black		6	SS	4			285									Organic Content: 41%
3.5								284									
284.5	SAND (SP), trace silt, trace gravel, saturated Compact to dense Grey		7	SS	13			283									
	- casing grinding on possible cobble or boulder from 7.6 m to 12.8 m		8	SS	15			282									
							281										
							280										
			9	SS	11		279									1 97 (2)	
							278										
							277										
	- some silt, saturated		10	SS	23		276										
							275										

Continued Next Page

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

ONTARIO MOT 1009213.01_HC.GPJ ONTARIO MOT.GDT 2/22/08

2 OF 3

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			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 (%)				
							20 40 60 80 100		○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE		w _P w w _L					
	SAND (SP), trace silt, trace gravel, saturated Compact to dense Grey (continued)															
	- some silt, some gravel, dense (SP-SM)		11	SS	36									13 70 (17)		
271.7 17.5	Sandy SILT (ML), wet Compacy to very dense Grey															
			12	SS	58											
	- some clay Compact		13	SS	14											
	- trace clay Very dense		14	SS	50/ 125 mm											
263.1 26.1	Clayey SILT (CL-ML), trace sand, wet Hard Grey															
			15	SS	65									0 4 72 24		
			16	SS	50/ 125 mm											
259.2			17	SS	50/											

Continued Next Page

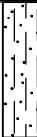
✕³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

RECORD OF BOREHOLE No HC-06-03

3 OF 3

METRIC

W.P. 71-00-00 LOCATION Stn: 10+620 o/s: 27 m Rt, Twp of Blenheim ORIGINATED BY JP
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Mudrotary Casing, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.20.06 - 4.20.06 CHECKED BY GC

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 ○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE															
					5 mm		20 40 60 80 100				10 20 30				
30.0	Silty SAND (SM), saturated Very Dense Grey						259								
257.8			18	SS	50/ 125 mm		258								
31.4	END OF BOREHOLE at approximately 31.4 m Groundwater measured at a depth of 3.5 m (Elev. 285.7 m) on completion of drilling														

RECORD OF BOREHOLE No HC-06-04

1 OF 2

METRIC

W.P. 71-00-00 LOCATION Stn: 10+645 o/s: 26 m Rt, Twp of Blenheim ORIGINATED BY MW
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.19.06 - 4.19.06 CHECKED BY GC

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 × LAB VANE							
289.9	Grass														
289.0	100 mm TOPSOIL, wet Black		1	SS	16										
289.3	SAND and GRAVEL (FILL), trace silt, trace organics, moist Compact Dark brown		2	SS	11										
0.6	SAND (SP), some silt, trace clay, saturated Compact to very loose Grey Some organics in upper 300 mm - trace organics		3	SS	2										
			4	SS	7										
286.5	- trace silt		5	SS	2										
3.4	PEAT, wet Very loose Black														
285.8	GRAVEL, trace silt, trace sand, saturated Compact Grey		6	SS	17										
284.4	SAND (SP-SM), trace gravel, trace silt, trace clay, saturated Compact Grey		7	SS	16										
	- some gravel, some silt		8	SS	16										
			9	SS	15										
275.6	Sandy SILT (ML), trace clay, trace gravel, saturated Compact to very dense Grey														
14.3															

Continued Next Page

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

ONTARIO MOT 1009213.01_HC.GPJ ONTARIO MOT.GDT 2/22/08

2 OF 2

METRIC

W.P.	<u>71-00-00</u>	LOCATION	<u>Stn: 10+645 o/s: 26 m Rt, Twp of Blenheim</u>	ORIGINATED BY	<u>MW</u>
DIST	<u>London</u>	HWY	<u>401</u>	BOREHOLE TYPE	<u>Hollow Stem Auger, Split Spoon</u>
				COMPILED BY	<u>MW</u>
DATUM	<u>Geodetic</u>	DATE	<u>4.19.06 - 4.19.06</u>	CHECKED BY	<u>GC</u>

[illegible]

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

RECORD OF BOREHOLE No HC-06-05

1 OF 1

METRIC

W.P. 71-00-00 LOCATION Stn: 10+595 o/s: 17 m Lt, Twp of Blenheim ORIGINATED BY RM
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.20.06 - 4.20.06 CHECKED BY GC

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											
								20 40 60 80 100											
								○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
							WATER CONTENT (%)												
291.2	Gravel Shoulder																		
0.0	SAND and GRAVEL (FILL), trace silt, trace organics, moist Compact Brown		1	SS	13		291										34 59 (7)		
			2	SS	22		290												
			3	SS	11		289												
288.6	- loose	4	SS	6	288														
2.6	SAND (SP), some silt, some gravel, some organics, saturated Loose to compact Dark brown		5	SS	11		287											16 72 (12)	
	- trace organics, seams of peat		6	SS	5		286												
	- very loose		7	SS	1		285												
285.9	PEAT, wet Loose Black		8	SS	8		284												
5.3								283											Organic Content: 1.3%
283.9	SAND (SP), trace silt, saturated Very loose Grey		9	SS	0			282											
7.3																			
281.4	END OF BOREHOLE at approximately 9.8 m Groundwater measured at a depth of 7.6 m (Elev. 283.6 m) on completion of drilling	10	SS	1															
9.8																			

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No HC-06-06

1 OF 1

METRIC

W.P. 71-00-00 LOCATION Stn: 10+660 o/s: 17 m Rt, Twp of Blenheim ORIGINATED BY RM
 DIST London HWY 401 BOREHOLE TYPE Hollow Stem Auger, Split Spoon COMPILED BY MW
 DATUM Geodetic DATE 4.19.06 - 4.19.06 CHECKED BY GC

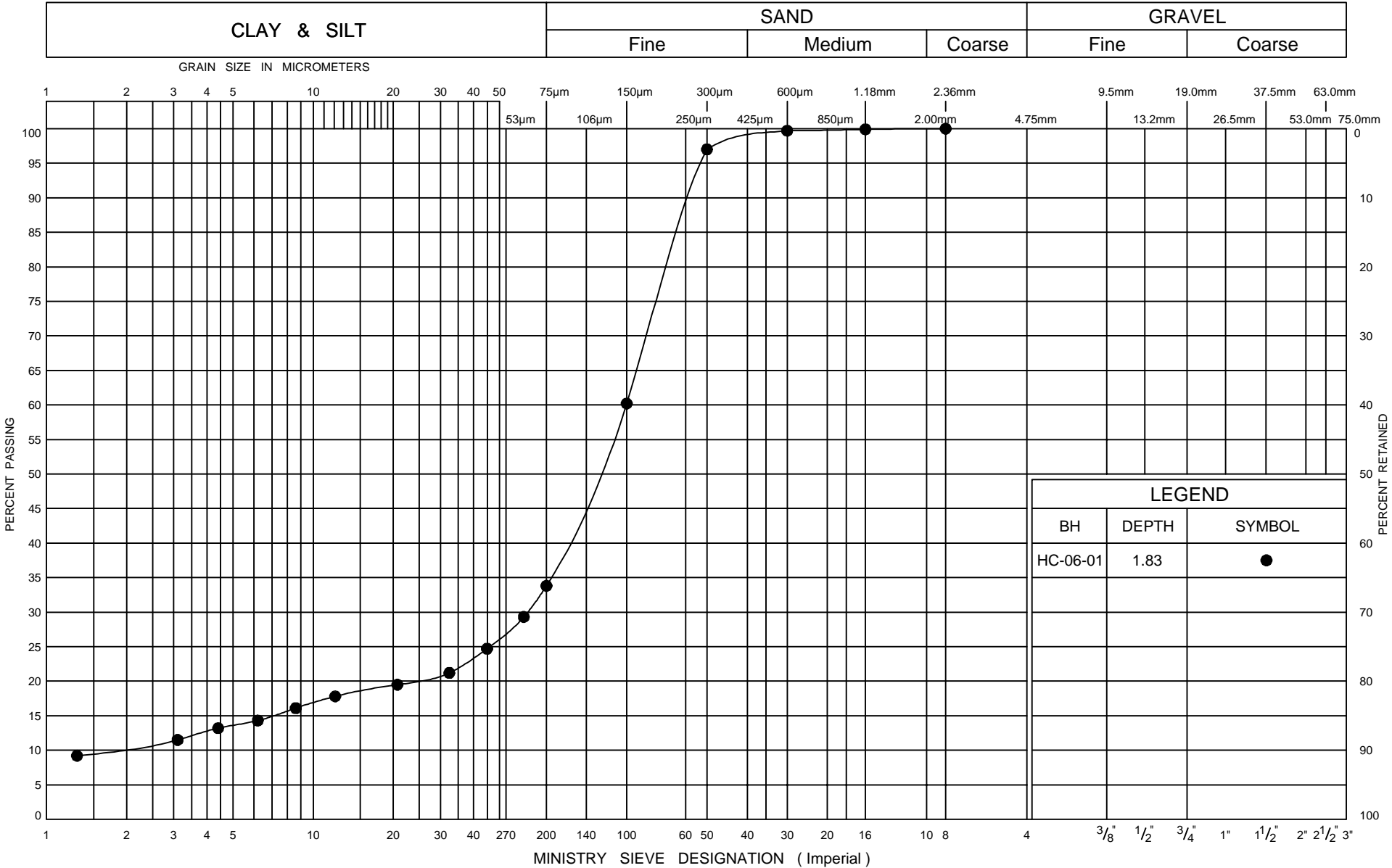
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
291.4	Gravel Shoulder							20	40	60	80	100					
0.0	SAND and GRAVEL (FILL), trace silt, trace organics, damp Compact to very dense Brown		1	SS	21		291						○				
			2	SS	44		290						○				54 41 (5)
			3	SS	29								○				
	- some silt		4	SS	77		289						○				
288.4																	
3.0	TOPSOIL, wet		5	SS	17		288								○		Organic Content: 4%
288.0	Loose, Black														○		
3.4	SAND (SW), some silt, wet Compact to loose Grey		6	SS	9		287										
			7	SS	4										○		
286.5							286										
4.9	PEAT, wet Loose, Black		8	SS	8		285										
284.2							284										
7.2	SAND (SW), some silt, trace gravel, wet Compact Grey		9	SS	27										○		12 76 (12)
283.2																	
8.2	END OF BOREHOLE at approximately 8.2 m Groundwater measured at a depth of 4.6 m (Elev. 286.8 m) on completion of drilling																

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

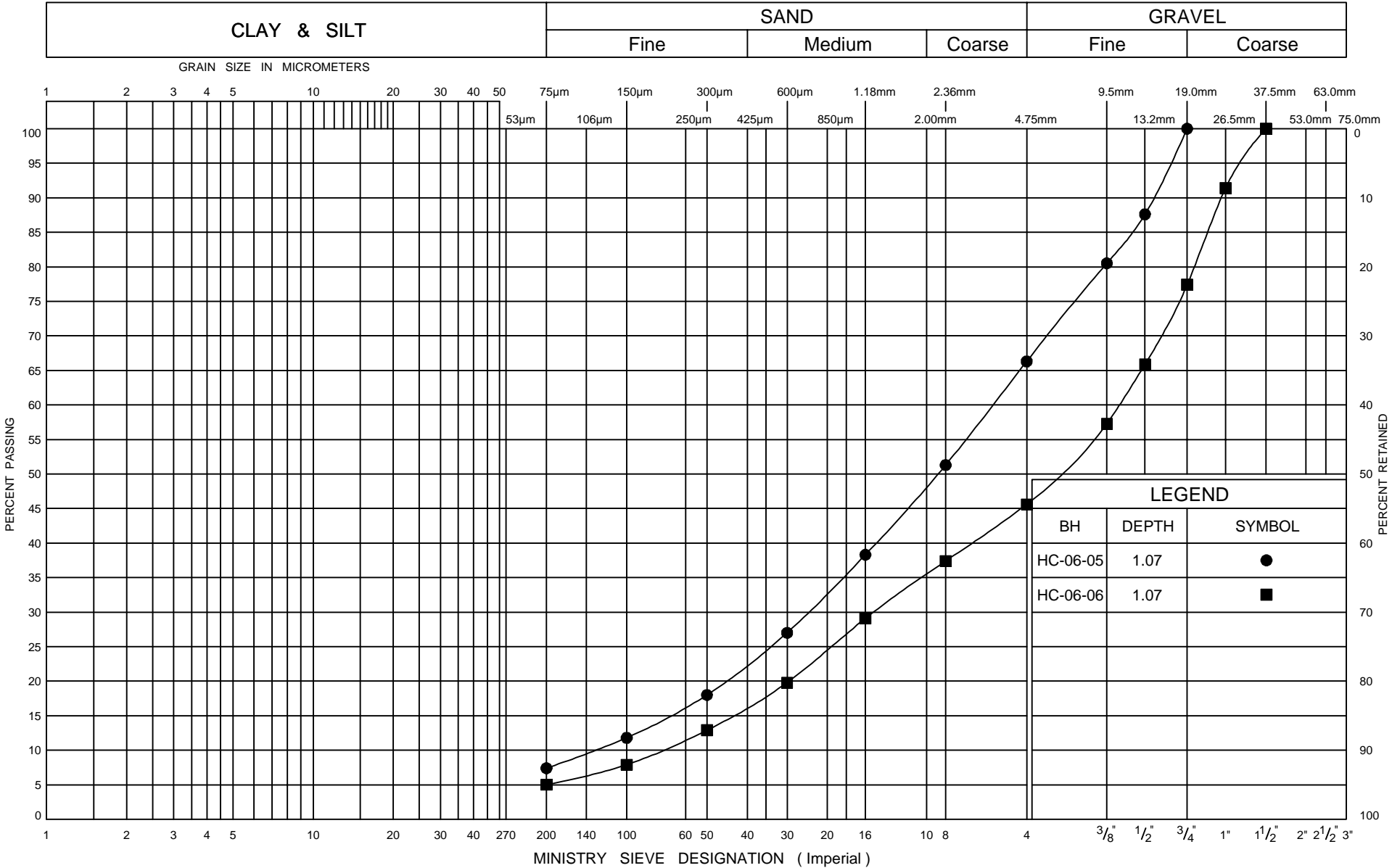
APPENDIX C

Geotechnical Laboratory Test Results

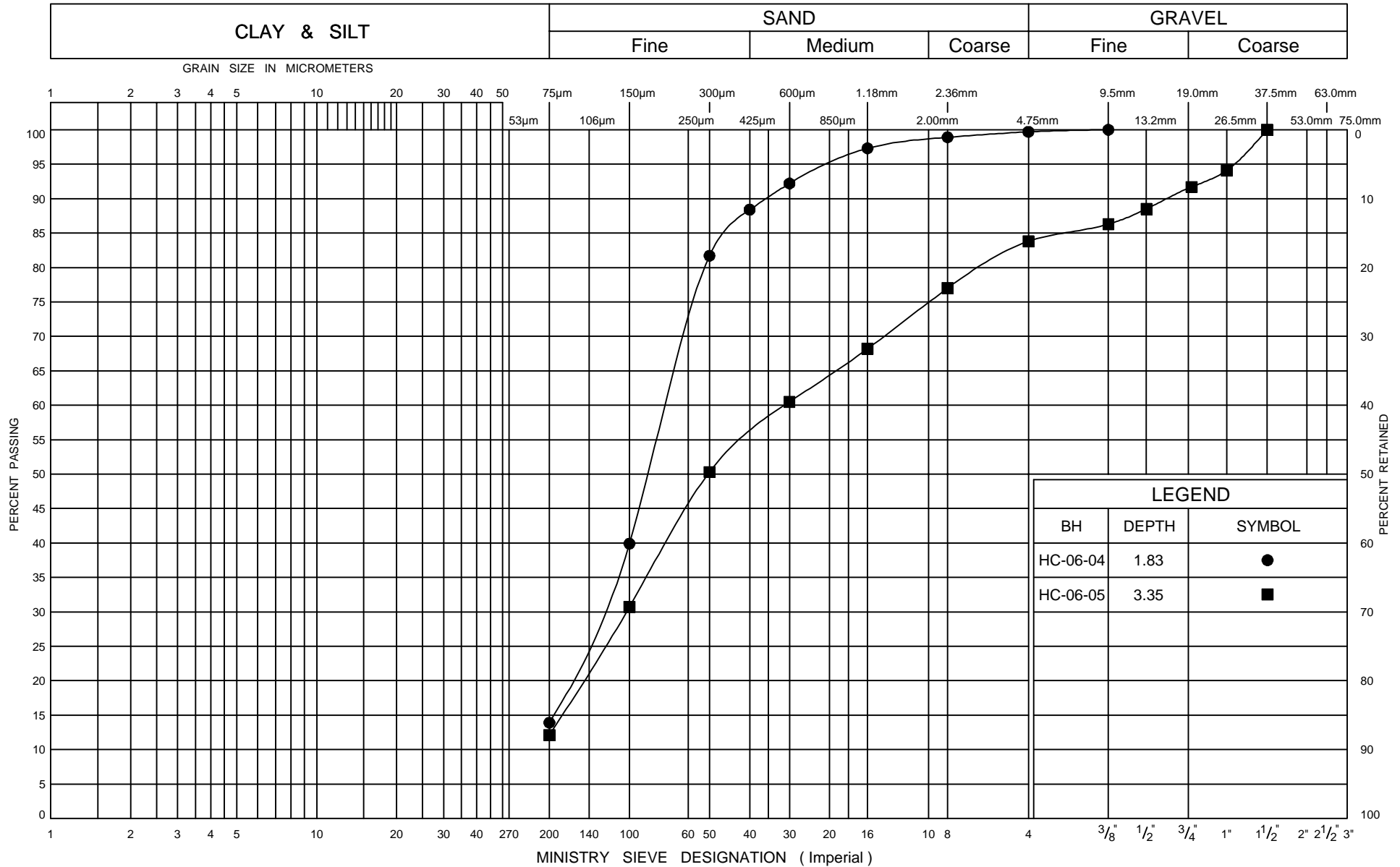
UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

UPPER SAND (SP)

FIG No 3

W P 71-00-00

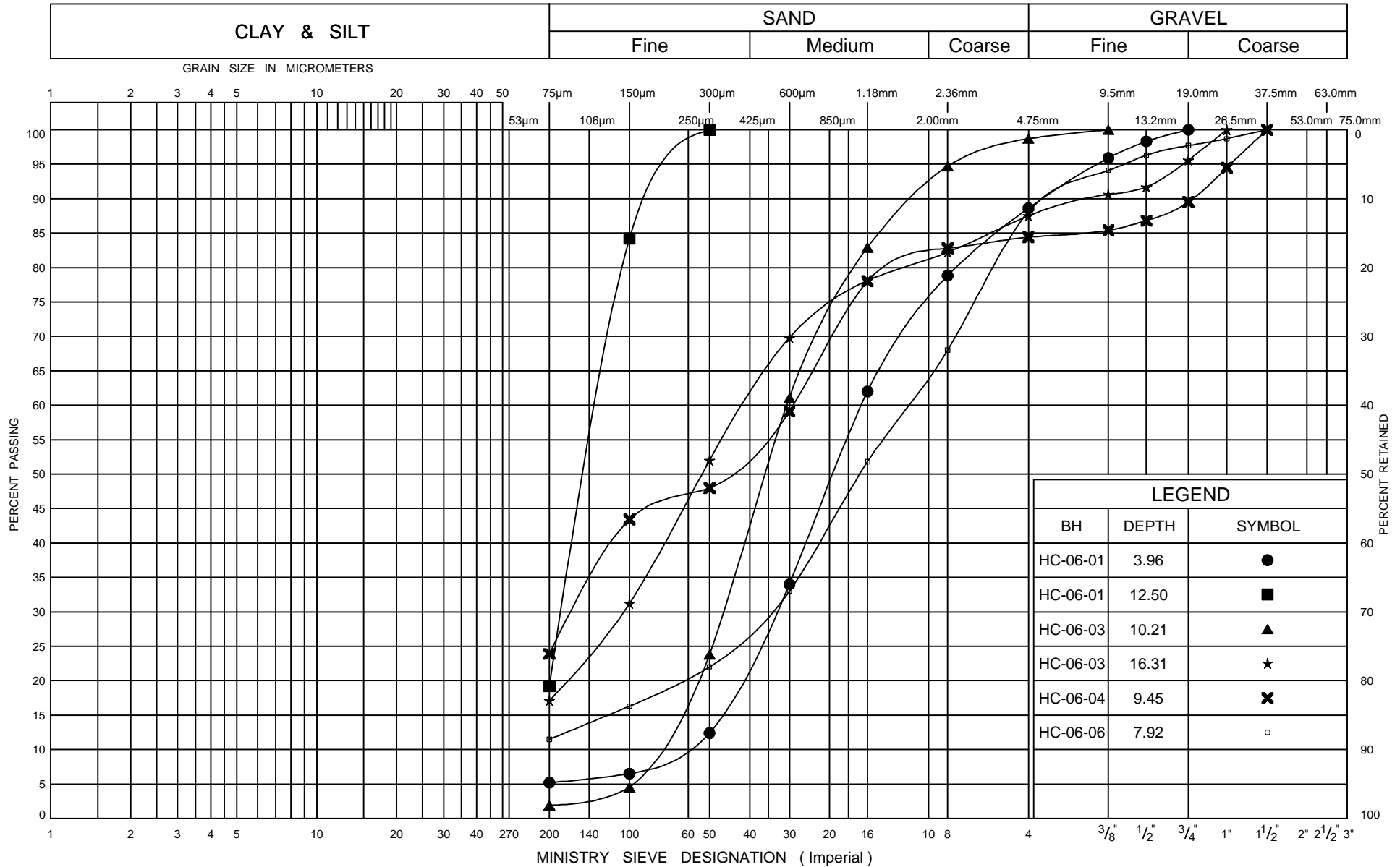
Horner Creek, Township of Blenheim



Ministry of
Transportation

Ontario

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

SAND (SW to SP - SM)

FIG No 4

W P 71-00-00

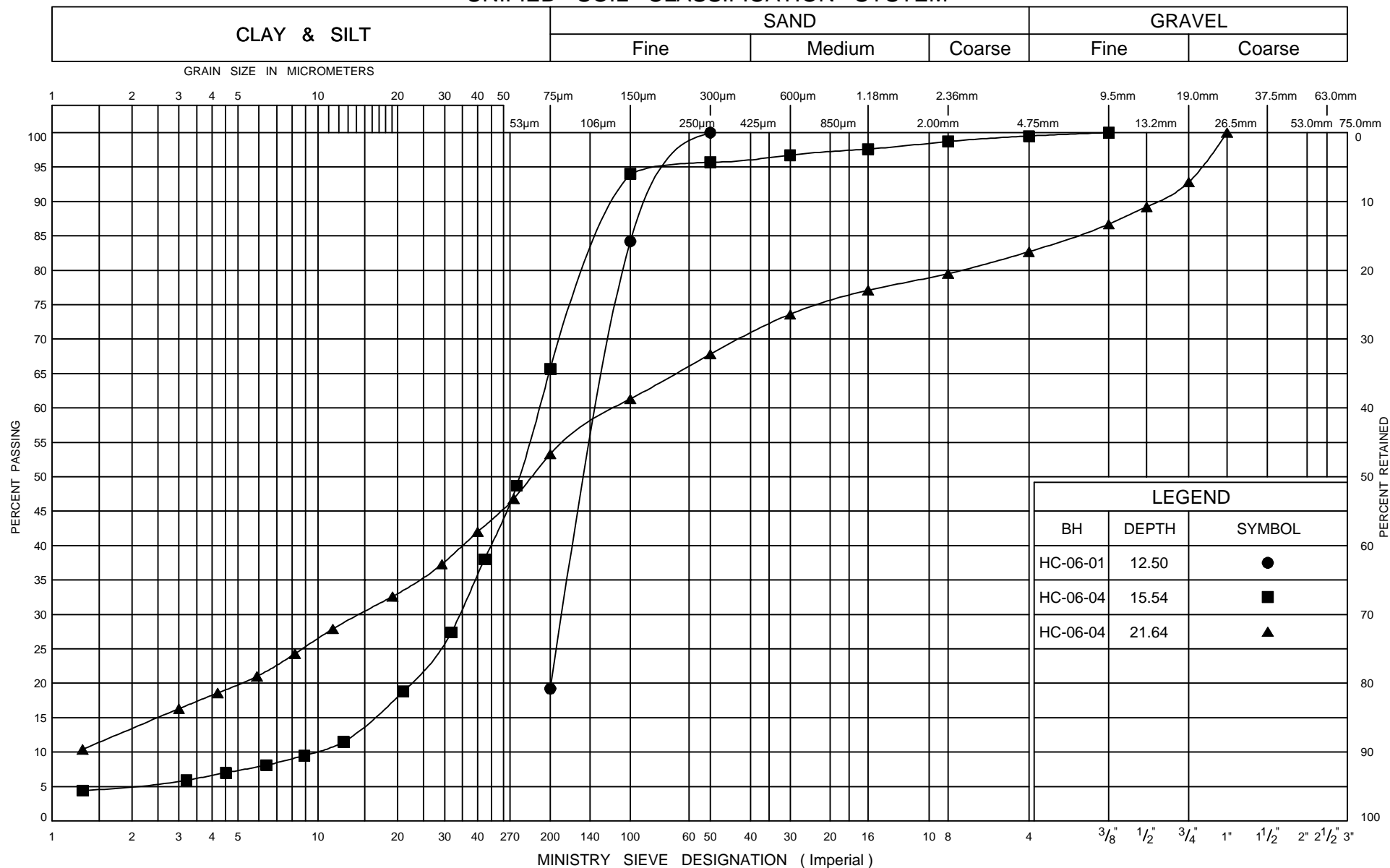
Horner Creek, Township of Blenheim



Ministry of
Transportation

Ontario

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

Sandy SILT (ML)

FIG No 5

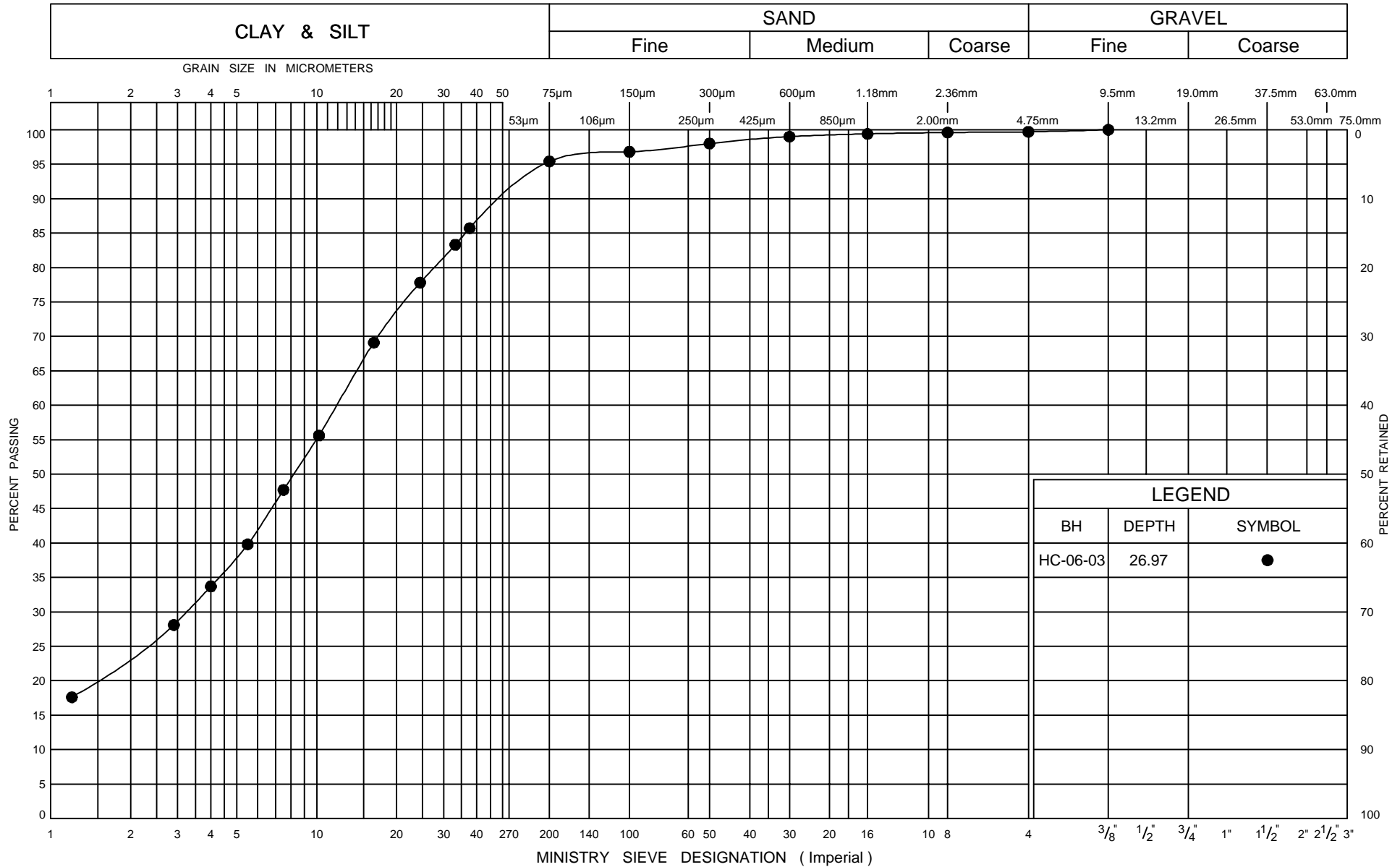
W P 71-00-00

Horner Creek, Township of Blenheim

Ministry of
Transportation

Ontario

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

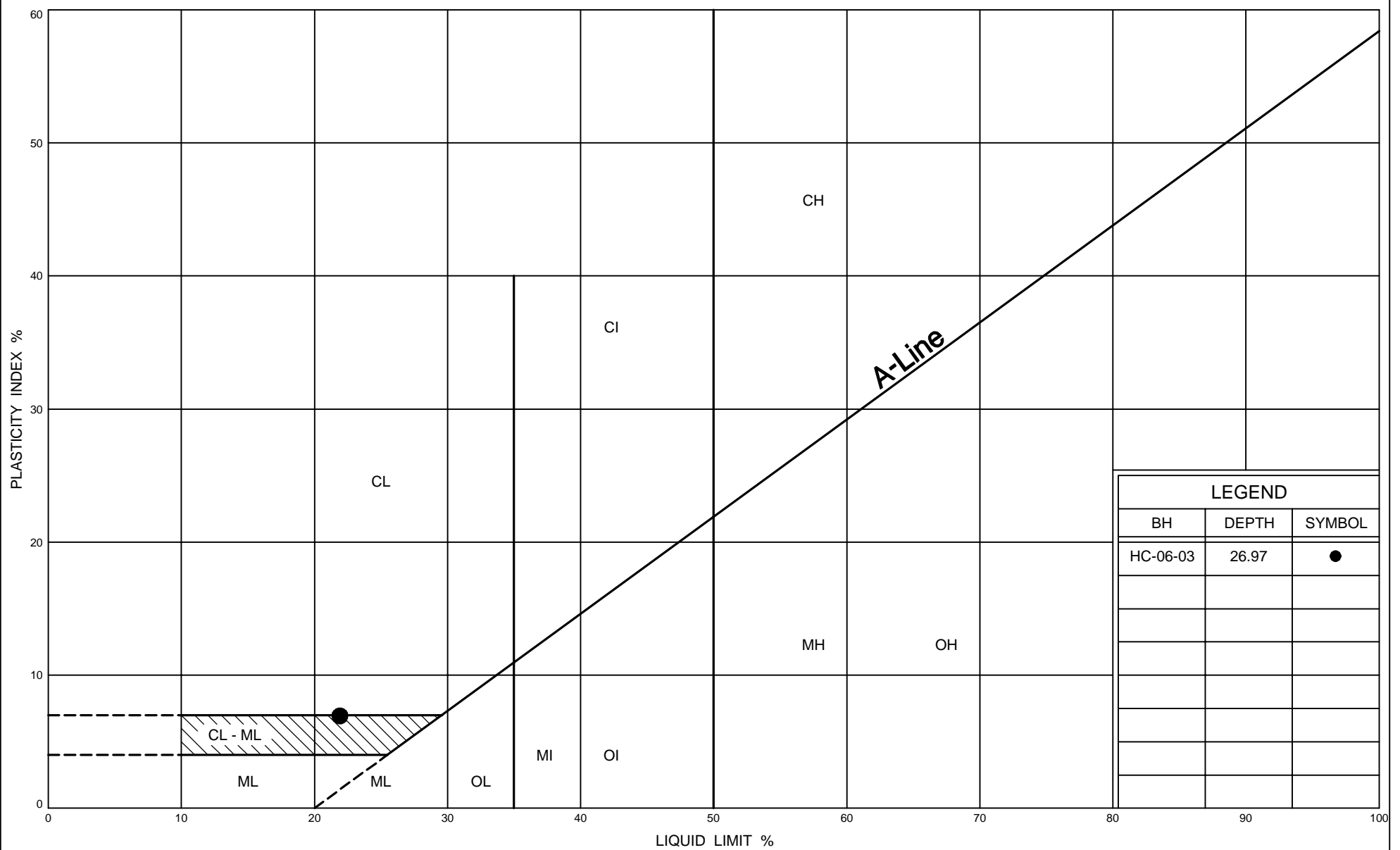
GRAIN SIZE DISTRIBUTION

Clayey SILT (CL - ML)

FIG No 6

W P 71-00-00

Horner Creek, Township of Blenheim



LEGEND		
BH	DEPTH	SYMBOL
HC-06-03	26.97	●



Ministry of
Transportation

PLASTICITY CHART

Clayey SILT (CL - ML)

FIG No 7

W P 71-00-00

Horner Creek, Township of Blenheim

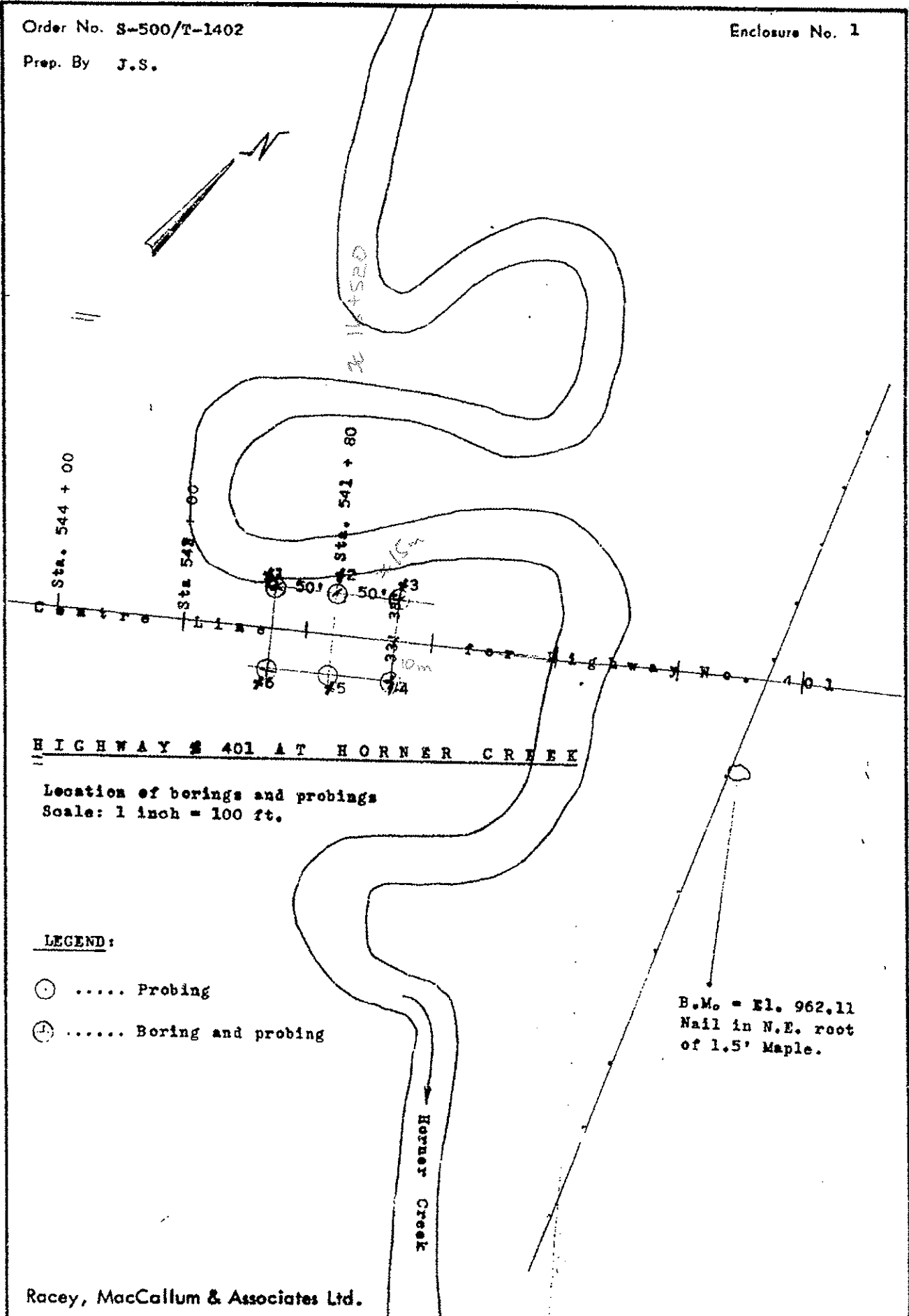
Appendix D

Drawings and Factual Record of Borehole Sheets
from Previous Investigations

Order No. S-500/T-1402

Enclosure No. 1

Prep. By J.S.



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: **1**Project: **Highway No. 401, Bridge No. 11.**Location: **Blenheim Township, Con. V, Lot 24.**Hole Location: **See Enclosure No. 1.**Hole Elevation and Datum: **948.0 Ft.**Field Supervisor: **A.H.** Prep.: **J.S.**Driller: **F.B.** Checked: **J.S.** Date: **26.9.58****LEGEND**

Shear Strength (C)

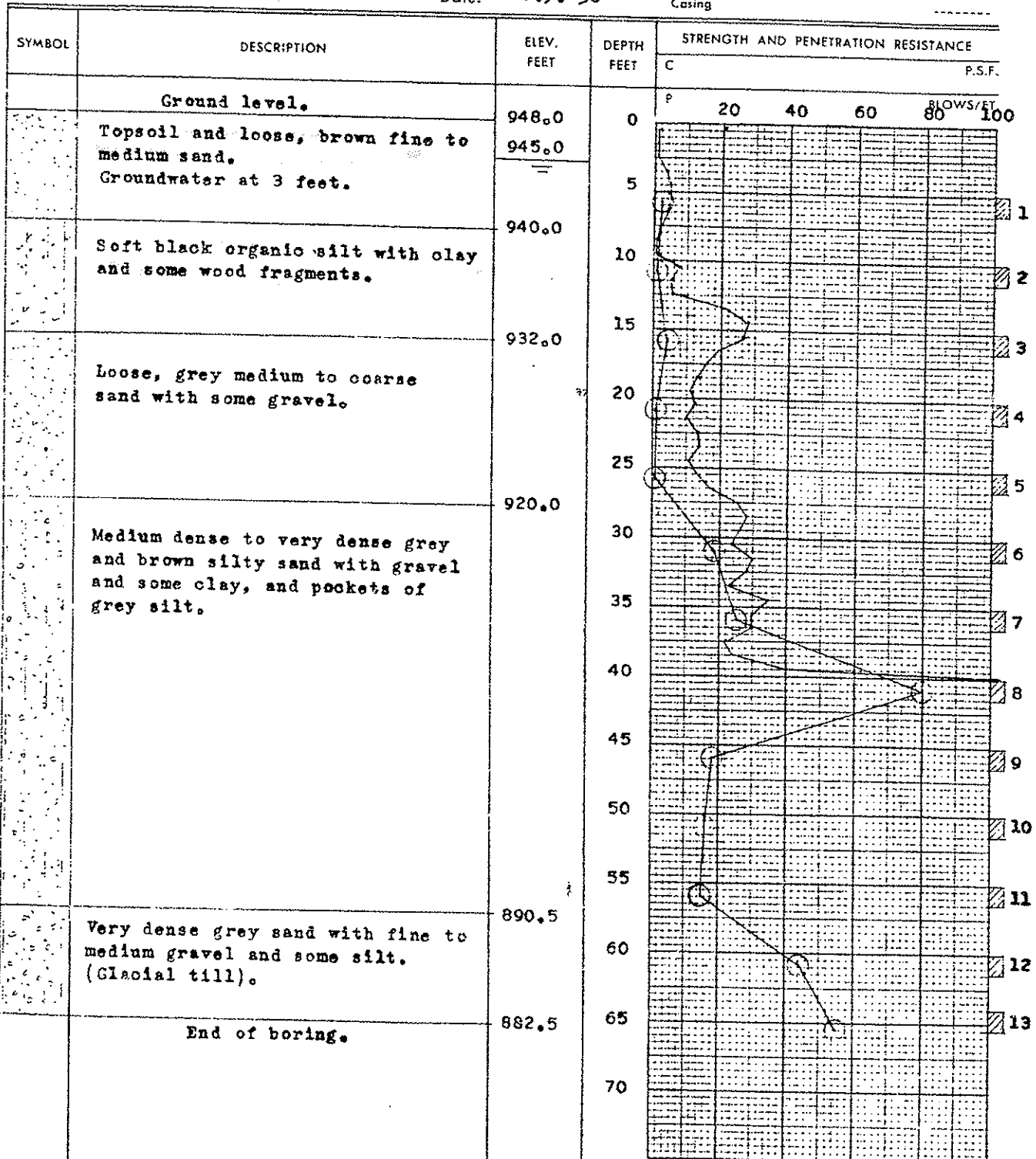
Unconfined compression
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

⊕
45

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Borehole~~ Probing : 2

Project: Highway No. 401, Bridge No. 11.
 Location: Blenheim Township, Con. V, Lot 24.

Hole Location: See Enclosure No. 1.

Hole Elevation and Datum: 948.8 Ft.

Field Supervisor: H.G. Prep.: J.S.

Driller: F.B. Checked: J.S. Date: 26.9.158

LEGEND

Shear Strength (C)

 Unconfined compression
 Vane test and sensitivity (S)
⊕
+s

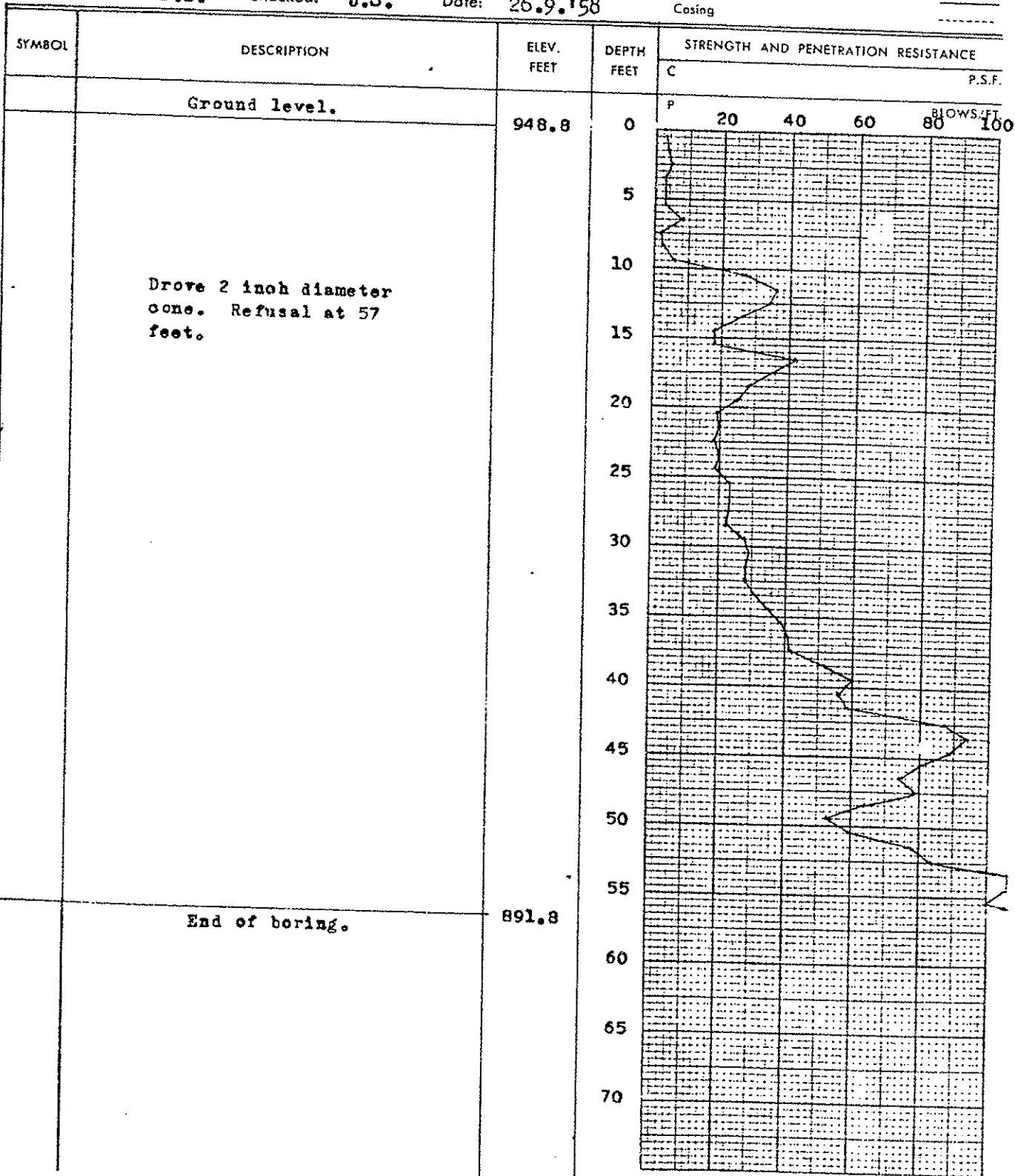
Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

⊕ ⊕ ⊕



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

Project: Highway No. 401, Bridge No. 11.
 Location: Blenheim Township, Con. V, Lot 24.
 Hole Location: See Enclosure No. 1.
 Hole Elevation and Datum: 948.7 Ft.
 Field Supervisor: H.G. Prep.: J.S.
 Driller: F.B. Checked: J.S. Date: 26.9. '58

LEGEND

Shear Strength (C)

Unconfined compression

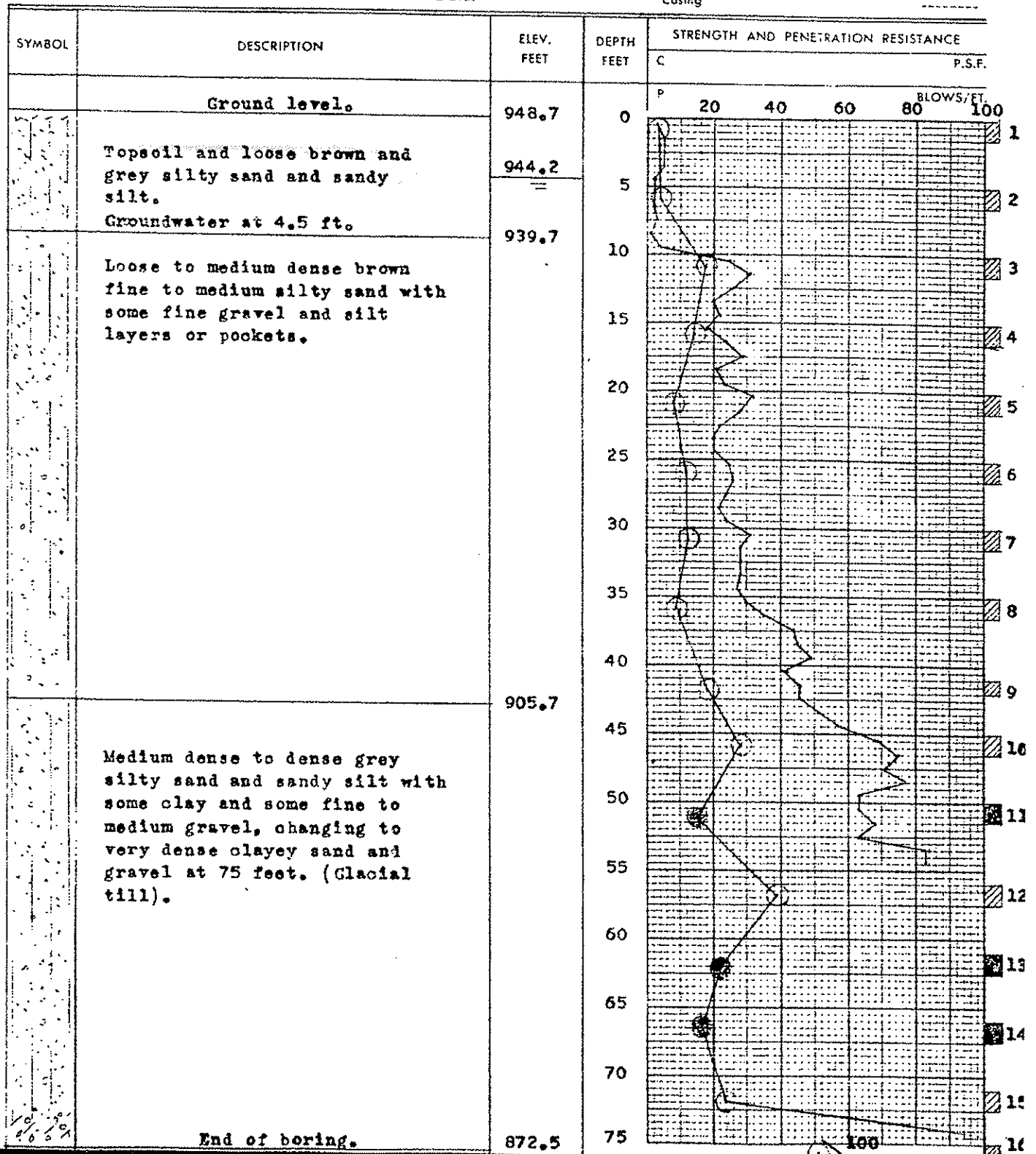
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

⊕
+S⊕
⊕
⊕

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 4

Project: Highway No. 401, Bridge No. 11.

Location: Blenheim Township, Con. V, Lot 24.

Hole Location: See Enclosure No. 1.

Hole Elevation and Datum: 947.9 Ft.

Field Supervisor: H.G. Prep.: J.S.

Driller: F.B. Checked: J.S. Date: 26.9.158

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

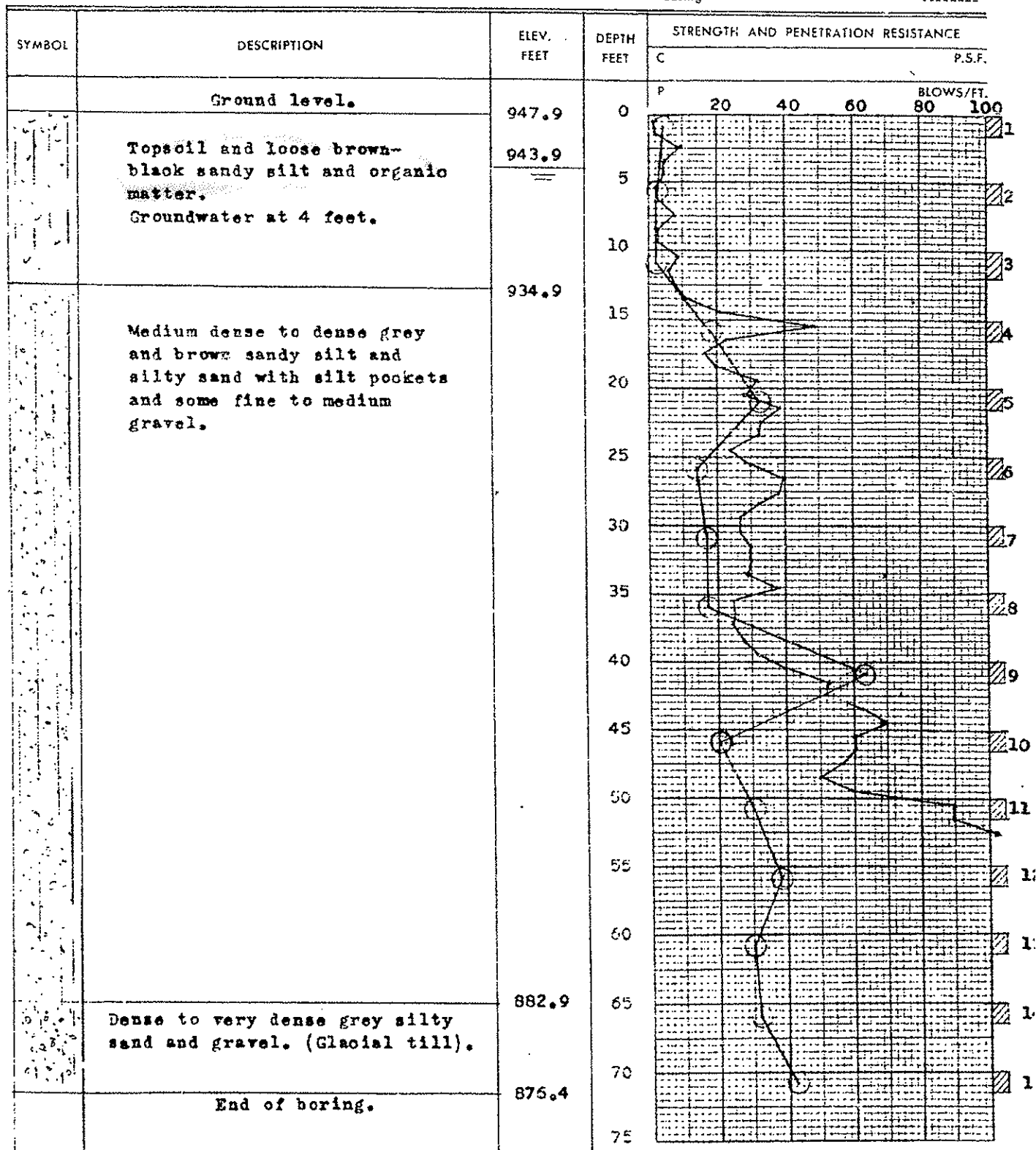
2" Split tube

2" Dia. Cone

Casing

⊕
+3

⊕ ⊕



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineer: G. Data Sheet for ~~Borehole~~ **Probing : 5**

Project: Highway No.101, Bridge No.11.
 Location: Blenheim Township, Con.V, Lot 24.
 Hole Location: See Enclosure No.1.
 Hole Elevation and Datum: 948.2 Ft.
 Field Supervisor: H.G. Prep.: J.S.
 Driller: F.B. Checked: J.S. Date: 26.9.'58

LEGEND

Shear Strength C

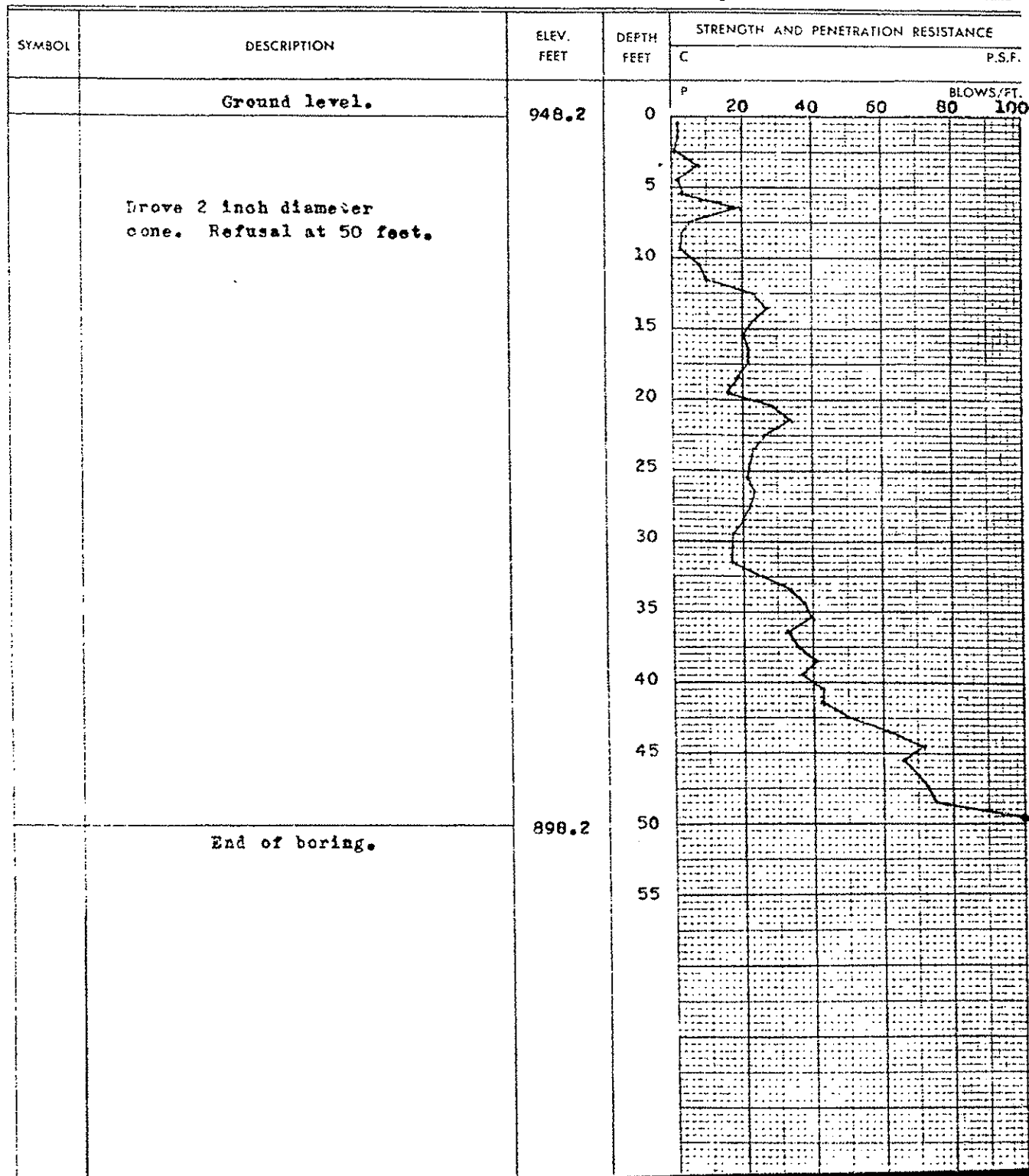
Unconfined compression
Vane test and sensitivity ψ

Penetration Resistance P

2" Split tube

2" Dia. Cone

Casing

⊕
+^s


RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 6

Project: Highway No. 401, Bridge No. 11.
 Location: Blenheim Township, Con. V, Lot 24.
 Hole Location: See Enclosure No. 1.
 Hole Elevation and Datum: 948.4 Ft.
 Field Supervisor: H.G. Prep.: J.S.
 Driller: F.B. Checked: J.S. Date: 26.9.'58

LEGEND

Shear Strength (C)

Unconfined compression
 Vane test and sensitivity (S)

Penetration Resistance (P)

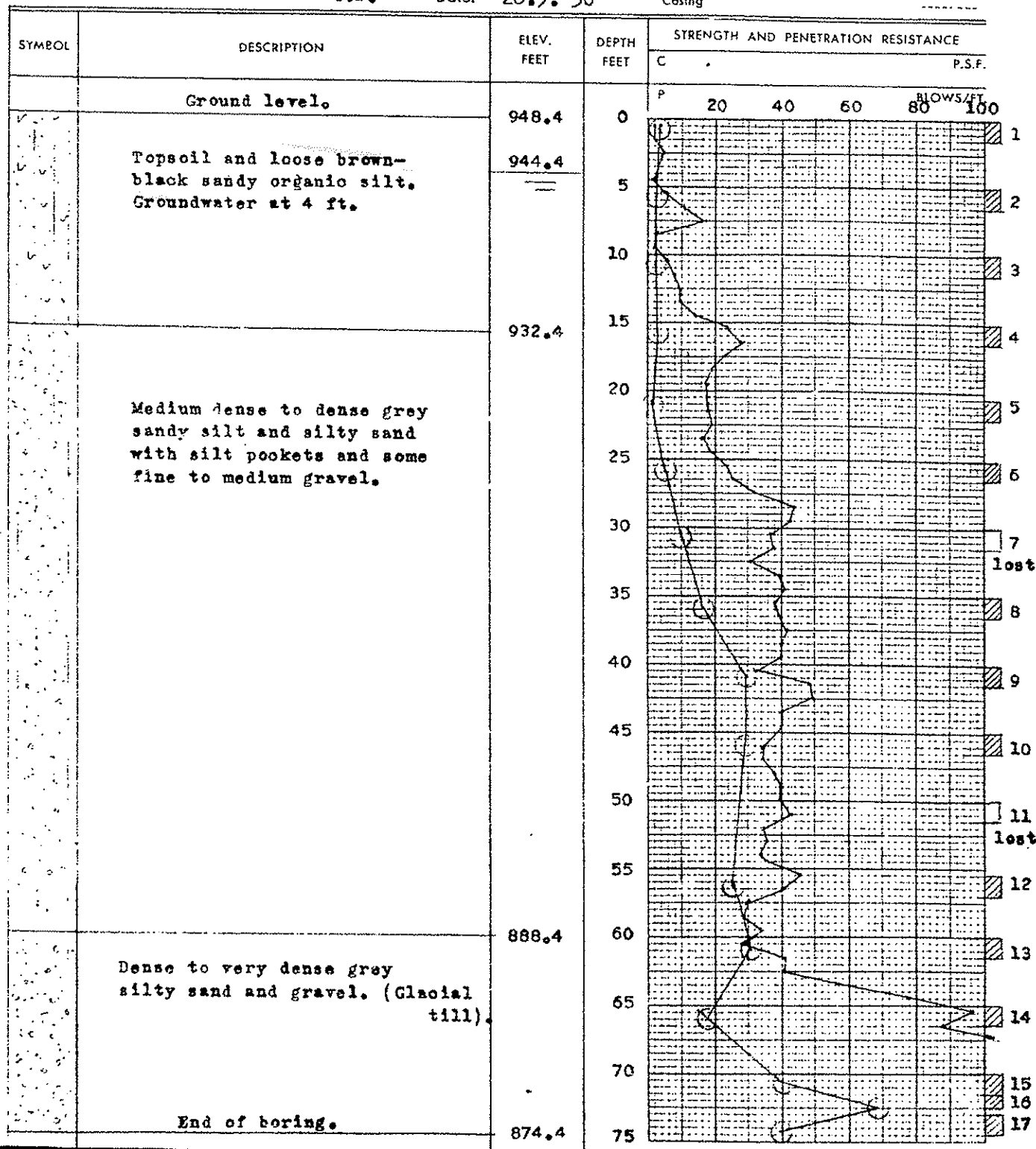
2" Split tube

2" Dia. Cone

Casing

⊕
+5

⊕ ⊕



RECORD OF BOREHOLE No 95-1

1 OF 2

METRIC

W.P. B17-93-01

LOCATION Sta. 10+608.9 o/s 0.0

ORIGINATED BY M.M.

DIST 31 HWY 401

BOREHOLE TYPE H.S. Auger

COMPILED BY T.H.

DATUM Geodetic

DATE 95 01 11

CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		UNIT WEIGHT γ kN/m ³	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 _L		
291.4	Ground Surface												
0.0	Sand and Gravel Trace Silt Occasional Cobbles and boulders (Fill)		1	SS	42		290						
288.5			2	SS	43								
2.9	Organics, Trace Sand Dark Brown		3	SS	28								
287.7			4	SS	7								
3.7	Sandy Silt trace Clay Very Loose		5	SS	5								
285.8			6	SS	3								
5.6	Organics, Dark Brown Wood Particles		7	SS	5								
284.3			8	SS	13								
7.1	Trace to Some Gravel Coarse Grained		9	SS	22								
			10	SS	14								
	Fine Grained		11	SS	18								
			12	SS	3								
	Fine to Medium coarse Silty Sand Very Loose to Dense		13	SS	32								
			14	SS	13								
	With Gravel Coarse Grained		15	SS	28								
			16	SS	23								
	Trace to Some Clay Fined Grained		17	SS	25								
			18	SS	23								
267.5			19	SS	27								
23.9	Clayey Silt Trace Sand												
	Stiff to Hard		20	SS	49								
260.9	Trace to Some Gravel [Glacial Till] Hard												

30.5

Continued

+3, x² Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 95-1 2 OF 2 METRIC

W.P. 817-93-01 LOCATION Sta. 10+608.9 o/s D.O. ORIGINATED BY M.M.
DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger COMPILED BY T.H.
DATUM Geodetic DATE 95 01 11 CHECKED BY T.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100	W _P	W	W _L		
280.2	Continued															
30.5	Trace to Some Gravel (Glacial Till) Hard		21	SS	115											
			22	SS	120											16 21 58 5
257.4	Trace Sand		23	SS	120											
34.0	End of Borehole • 'N' values within the Silty Sand deposit may be questionable due to blow up and disturbance during sampling.															

RECORD OF BOREHOLE No 95-2 1 OF 2 METRIC

W.P. 818-93-01 LOCATION Sta. 10+649.2 o/s 0.0 ORIGINATED BY M.M.
 DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger COMPILED BY M.M.
 DATUM Geodetic DATE 95.01.10 CHECKED BY T.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N' VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%) 20 40 60		
291.3	Ground Surface												
0.0	Sand and Gravel Trace Silt Occasional Cobbles and Boulders [Fill]		1	SS	64	/20cm	290	Cone Penetration Test commenced at 3 m from the surface beyond existing fill					
288.4			2	SS	35		288						
287.6	Organics, Trace Sand Dark Brown, Wood Particles		3	SS	41		286						
3.7	Sandy Silt trace Clay Loose to Compact		4	SS	17		284						
286.1			5	SS	13		282						
5.2	Organics, Dark Brown Wood Particles		6	SS	5		280						
284.2			7	SS	7		278						
7.1			8	SS	5		276						
	Fine to Medium Coarse Silty Sand Very Loose to Very Dense		9	SS	1		274						
			10	SS	10		272						0 89 10 1
			11	SS	5		270						
			12	SS	8		268						
			13	SS	32		266						0 87 12 1
			14	SS	56		264						
			15	SS	74		262						
			16	SS	31		260						
			17	SS	20		258						50 42 6 2
			18	SS	21		256						
269.0			19	SS	34		254						
22.3	Clayey Silt Trace Gravel Trace Sand (Glacial Till) Hard		20	SS	72		252						1 17 70 12
			21	SS	130		250						
264.4			22	SS	120	/13cm	248						
26.9	Silty Sand Some Gravel Trace Clay Very Dense		23	SS	50	/8cm	246						
261.4							244						
29.9							242						

Continued

-3, x 5, Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 95-2 2 OF 2 METRIC

W.P. 818-93-01 LOCATION Sta. 10+649.2 e/s 0.0 ORIGINATED BY M.M.
DIST 31 HWY 401 BOREHOLE TYPE H.S. Auger COMPILED BY M.M.
DATUM Geodetic DATE 95 01 10 CHECKED BY T.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100		
260.8	Continued												
30.5			25	SS	135								
257.3			26	SS	60								
34.0	End of Borehole												
	<ul style="list-style-type: none"> * 'N' Values within the Silty Sand deposit may be questionable due to blow up and disturbance during sampling. 												

Appendix E

List of Standard Drawings and Specifications

Standard Specification and Drawings:

The following is a list of Standard Specifications and Drawings referenced in the Foundation Report for the proposed Horner Creek Bridge Widening on Highway 401 in the Township of Blanford-Blenheim, Ontario.

Standard Drawings:

OPSD3000.100
OPSD3000.150
OPSD3101.150
OPSD3102.100
OPSD208.010

Standard Specifications; OPSS572

Special Provisions:

SP903S01
SS-103-11
SP902S01
SP206S03
SP105S19

