

REPORT

Foundation Investigation and Design
Report

GWP 71-00-00

Highway 401

Canadian Pacific Railway Overhead
Widening

Site 23-118

Township of Blanford-Blenheim,
Ontario

London District

Stantec Consulting Ltd.

PROJECT NO. 1009213.01

GEOCRES NO. 40P2-68

PROJECT NO. 1009213.01

REPORT TO

**Stantec Consulting Ltd.
1400 Rymal Road East
Hamilton, Ontario
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FOR

**Foundation Investigation and Design
Report**

ON

**CPR Overpass Widening
Site 23-118
Township of Blanford-Blenheim, Ontario
London District
GWP 71-00-00
Geocres No. 40P2-68**

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FOUNDATION INVESTIGATION REPORT

Canadian Pacific Railway (CPR) Overpass Widening
Site 23-118
Highway 401
Township of Blanford-Blenheim, Ontario
GWP 71-00-00
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 structure over the Canadian Pacific Railway, in the Township of Blanford-Blenheim, Ontario, GWP 71-00-00.

The work was carried out under Agreement No. 3005-E-0013. 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 foundation investigation was to assess the subsurface soil conditions at the site for the planned structure widening by using existing borehole data.

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 SITE DESCRIPTION

The site location is a section of Highway 401 at the Canadian Pacific Railway (CPR) Overpass in the Township of Blanford-Blenheim, Ontario, as shown on the Key Plan on Drawing No. 1 in **Appendix A**.

Highway 401 in the area of the CPR line is a four-lane road and carries east and west bound traffic. 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 railway location. The road profile is generally level and the pavement structure is typically constructed on embankments that are generally 8 m to 9 m higher than the surrounding lands. The lands adjacent to the highway are generally undulating.

The CPR Overpass (also known as Township Bridge No. 9) consists of 2 structures, one to convey the eastbound traffic and one to convey the westbound traffic of Highway 401 over the CPR right-of-way. The existing structures were reportedly constructed in 1960 and rehabilitated in 1998 when the bridge deck was reportedly widened. The existing structures are twin three-span reinforced concrete variable depth Tee beam structures. The approach spans are approximately 14 m each with a central span of approximately 19 m and are supported on 2 sets of intermediate supports.



3.0 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 soils.

Physiographic mapping in the site indicates drumlinized till plains.

4.0 INVESTIGATION PROGRAM

4.1 Scope of Work

The scope of work for the investigation was as follows:

- To assess the subsurface soil and groundwater conditions at eight borehole locations advanced by others; and,
- To prepare a Foundation Investigation and Design Report.

4.2 Field Investigation Procedures

Site specific field work was not carried out for this investigation. This report uses the factual results of the following two (2) original investigations:

- Department of Highways - Ontario report titled, "Proposed Crossing of Highway No. 401 Overpassing C.P.R. Right of Way, Township of Blenheim, Ontario" dated September 1958; and,
- Ministry of Transportation Ontario Structural Section Southwestern Region, London - Memorandum titled, "WP 819/820-93-01, Site 23-118/1/2, Hwy 401 and CPR Overhead EBL/WBL District 31, London, Southwestern Region" dated April 1995.

4.2.1 September 1958 Investigation

The fieldwork for the investigation was carried out in early September 1958. A total of 6 boreholes (Borehole 1 to 6) were advanced at the approximate locations shown on the Borehole Location Plan and Soil Strata Plot, Drawing No. 1 in **Appendix A**.

The ground surface elevation at the borehole locations were converted from imperial measurements to metric units. The boreholes for the foundation investigation were advanced to depths in the range of approximately 17.1 m to 19.4 m, elevations of about 281.2 m to 284.2 m. Soil samples were recovered from the boreholes at regular intervals.

The groundwater levels encountered in the boreholes were reported on the Record of Borehole sheets.

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.



4.2.2 April 1995 Investigation

The fieldwork for the investigation was carried out in early January 1995. A total of 2 boreholes (Borehole 95-1 and 95-2) were advanced: one at each abutment location atop the existing fill within the median. The station references for Borehole 95-1 and 95-2 are Sta. 13+680.8 and Sta. 13+620.9 respectively.

Boreholes 95-1 and 95-2 were advanced depths of approximately 24.8 m and 29.4 m, elevations of about 285.9 m and 281.6 m, respectively. Soil samples were recovered from the boreholes at regular intervals.

The groundwater levels encountered in the boreholes were reported on the Record of Borehole sheets.

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.

5.0 RESULT OF THE INVESTIGATION

5.1 Subsurface Conditions

The subsurface conditions reported in the boreholes carried out for the 1958 and 1995 geotechnical investigations are summarized on the Record of Borehole sheets provided in **Appendix B**.

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

5.2 Soil

5.2.1 Organic Soil (Topsoil)

Organic soil (topsoil) was encountered at the ground surface in Boreholes 1 to 6 advanced for the 1958 investigation. The thickness of the organic soils ranged from approximately 300 mm to 900 mm.

5.2.2 Fill (SM- SC)

Fill was encountered at the ground surface in Boreholes 95-1 and 95-2. The fill generally consisted of sand and silt that contained trace to some gravel and was interbedded with layers of clayey silt. The fill was approximately 9 m to 9.8 m thick and extended to elevations of approximately 301.2 m and 301.7 m

Laboratory testing performed on selected samples of the fill consisted of moisture content, grain size distribution and Atterberg Limits tests. The test results were as follows:

- Moisture Content: 11% and 9%
- Grain Size Distribution:
 - 1% to 11% gravel;
 - 47% to 73% sand;
 - 23% to 39% silt; and,
 - 3% and 14% clay.
- Atterberg Limits:



- Plastic limits: 12%
- Liquid limits: 15%
- Plasticity Indices: 3%

The results of the moisture content, grain size distribution and Atterberg Limits tests are shown on the Record of Borehole sheets in **Appendix B**.

5.2.3 Sand and Gravel

Sand and gravel was encountered in Boreholes 1 to 6 underlying the organic soil (topsoil) at depths of approximately 0.3 m to 0.9 m, elevations of about 298.7 m to 301.6 m. The sand and gravel ranged in total thickness from approximately 0.9 m to 2.7 m, and extended to elevations in the range of approximately 297.2 m to 300.3 m.

Based on the N-values obtained from the Standard Penetration Tests (SPTs), the compactness of the sand and gravel was considered to be compact to loose.

5.2.4 Sand (SP/SM/ML)

Brown sand was encountered in all the boreholes. The sand was encountered at elevations of approximately 297.2 m to 301.2 m. The sand ranged in total thickness from approximately 2.6 m to 6.2 m, and extended to elevations in the range of approximately 293.5 m to 294.7 m in Boreholes 1 to 6. Interpretation of the information on the logs for 95-1 and 95-2 suggests a corresponding boundary at an elevation of approximately 294 m and 292 m, respectively.

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

Laboratory testing reported for samples of the sand consisted of grain size distribution tests. The test results were as follows:

- Grain Size Distribution:
 - 0% to 17% gravel;
 - 47% to 92% sand;
 - 6% to 52% silt; and,
 - 1% to 5% clay.

The results of the grain size distribution tests are shown on the Record of Borehole sheets in **Appendix B**. In addition, the results of the grain size distribution tests are presented on the Figures included in **Appendix C**.

5.2.5 Sandy Silt / Silt with Sand (ML)

Various strata of sandy silt and silt with sand were encountered in the boreholes advanced in both 1958 and 1995 investigations at depths of approximately 2.7 m to 19 m below existing grade, elevations of about 292.5 m to 294.7 m. The thickness of the sandy silt and silt with sand ranged from approximately 1.2 m to 5.2 m, and the materials extended to elevations in the range of approximately 288.3 m to 291.4 m.

Based on the N-values obtained from the SPTs, the compactness of the sandy silt, silty sand and silt with sand was considered to be loose to very dense but was more typically loose to compact. The 1995 Record of Borehole sheets indicate that the N-values may not be reliable.



Laboratory testing performed on selected samples of the sandy silt and silt with sand consisted of grain size distribution tests. The test results were as follows:

- Grain Size Distribution:
 - 0% gravel;
 - 5% to 23% sand;
 - 73% to 83% silt; and,
 - 4% to 12% clay.

The results of the grain size distribution tests are shown on the Record of Borehole sheets in **Appendix B**. In addition, the results of the grain size distribution tests completed for the 1958 investigation are presented on the Figures included in **Appendix C**.

5.2.6 Glacial Till (ML-CL)

Glacial till, comprised of clayey silt till and silt with sand till was encountered in all boreholes at depths of approximately 11 m to 22.3 m below existing grade, elevations of about 288.3 m to 291.4 m. The boreholes were terminated in the glacial till at depths of approximately 17.1 m to 29.4 m below existing grade, elevations of about 281.2 m to 285.9 m.

Based on the N-values obtained from the SPTs, the glacial till was considered to be very dense / hard.

Laboratory testing performed on selected samples of the glacial till consisted of moisture content, grain size distribution and Atterberg Limits tests. The test results were as follows:

- Moisture Content: 14% and 15%
- Grain Size Distribution:
 - 0% to 10% gravel;
 - 5% to 26% sand;
 - 49% to 73% silt; and,
 - 15% to 32% clay.
- Atterberg Limits:
 - Plastic limits: 14% and 12%
 - Liquid limits: 14% and 22%
 - Plasticity Indices: 0% and 10%.

The results of the grain size distribution tests are shown on the Record of Borehole sheets in **Appendix B** and are presented on the figures included in **Appendix C**. The results of the moisture content and Atterberg Limits Tests are provided on the Record of Borehole sheets in **Appendix B**.

5.3 Groundwater

The groundwater conditions reported in the boreholes (Boreholes 1 to 6, 95-1 and 95-2) at the time of the field investigations are summarized in Table 5.2 below:



Groundwater Level Measurements

| Borehole | Depth*(m) in Borehole at the time of the investigation | Geodetic Elevation (m) |
|-----------------|---|-------------------------------|
| 1 | 0.9 | 298.7 |
| 2 | 1.8 | 299.0 |
| 3 | 1.8 | 299.5 |
| 4 | Not reported | Not reported |
| 5 | 2.4 | 299.3 |
| 6 | 2.1 | 299.2 |
| 95-1 | 11.7 | 299.0 |
| 95-2 | 15.4 | 295.6 |

Notes: *Reported from the ground surface

The groundwater levels noted above, are those that were encountered and reported at the time of the field investigations and will be subject to fluctuations due to seasonal effects and significant precipitation events.

6.0 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 groundwater 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.

Regards,

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



FOUNDATION DESIGN REPORT

Canadian Pacific Railway (CPR) Overpass Widening
Site 23-118
Highway 401
Township of Blanford-Blenheim, Ontario
G.W.P. 71-00-00
District – London

7.0 DISCUSSION

7.1 General

Highway 401 in the area of the CPR line is a four-lane road conveying 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 railway bridge location. The road profile is generally level and pavement is typically constructed on embankments that are generally 8 m to 9 m higher than the surrounding lands. The lands adjacent to the highway are generally undulating.

The CPR Overpass (also known as Township Bridge No. 9) consists of 2 structures, one to convey the eastbound traffic and one to convey the westbound traffic of Highway 401 over the CPR right-of-way. The existing structures were reportedly constructed in 1960 and rehabilitated in 1998 when the bridge deck was reportedly widened. The existing structures are twin three-span reinforced concrete variable depth Tee beam structures. The approach spans are approximately 14 m each with a central span of approximately 19 m.

A drawing titled “Blenheim Township Bridge #9 C.P.R. Overhead”, Drawing No. D4284-2 by the Department of Highways Ontario, dated February 23, 1959, indicates that the existing structures are supported on steel 10BP42 H-piles driven to the underlying very dense till at an elevation of about 289.4 m.

Another drawing titled “Blenheim Township Bridge #9 C.P.R. Overhead”, Drawing No. D4284-1 by the Department of Highways Ontario, dated February 23, 1959, indicates that the top elevation for the pile caps at the central piers are at an elevation of about 300.5 m and the central pier piles are approximately 12.1 m long. The pile caps at the abutments are perched in the embankment fill with the top of the pile cap at an elevation of about 307.9 m with the abutment piles approximately 18.2 m in length.

7.2 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-Blenheim, 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 eastbound and westbound lanes of the highway. It is understood that the approach fills will be adjusted by filling in the median ditch (3.5 m wide and 1 m deep) and by widening the roadway platform by 1.0 m to both the north and south.

The bridge deck widening will consist of a variable depth cast-in-place reinforced concrete solid slab, supported on widened abutments and 6 new additional concrete columns: 3 at each pier: one to the outside of each existing bridge and one between the existing bridges.

7.3 Subsurface Conditions

The subsurface conditions reported at the location of the CPR Overpass at the time of the original investigation in 1958, generally consist of organic soil (topsoil) at the ground surface underlain by loose to compact sand and gravel that extended to elevations in the range of about 294.0 m to 300.3 m. The sand and gravel was underlain by loose to very dense sand to elevations in the range of about 292.0 m to 294.7 m, which was underlain by loose to compact layers of sandy silt and silt with sand to elevations of about 288.3 m to 291.4 m.

Very dense sand with silt till was encountered underlying the sandy silt and silt with sand at elevations of about 288.3 m to 291.4 m. All boreholes were terminated in the very dense sand with silt till at elevations of about 281.2 m to 285.9 m.

Groundwater was reported in the boreholes at depths in the range of approximately 0.9 m to 15.4 m below existing grade, elevations of about 295.6 m to 299.5 m.

7.4 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 soils and the existing embankment fills. | Lowest cost. | Shoring would be required. Low bearing resistance available. | Low to medium | Differential movement between the existing structure and the widened area. |
| Spread footings on granular pads | Higher geotechnical resistance. | Excavations to construct the granular pads may impact the existing pile caps, pavement structure and railway tracks. Shoring would be required. Dewatering techniques would likely be required at the pier locations. | Medium to High | Difficulties at the pier locations to backfill. Differential movement between the existing structure and the widened area. Excavation and dewatering issues. |

| Foundation Option | Advantages | Disadvantages | Relative Cost | Risks/Consequences |
|---|--|---|---------------|---|
| End bearing piles driven to the glacial till. | 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. |
| Friction piles founded in the sands and silts above the glacial till. | 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. |

It is recommended that the pier and abutment extensions be supported on end bearing driven piles, which would be consistent with the foundation for the existing bridge structures.

8.0 RECOMMENDATIONS

A list of the standard drawings and provisions referenced in this report is provided in Appendix D.

8.1 Pile Foundations

8.1.1 Geotechnical Resistance

Given the conditions reported and that the existing bridge structure is founded on steel piles driven to the underlying till, it is recommended to found the bridge widening on piles driven to the underlying very dense till. This option would be consistent with the foundations for the existing structures.

The piers and abutments could be supported on end bearing HP310x110 piles driven to the underlying very dense till at depths of approximately 12.1 m to 18.2 m below the pier and abutment pile caps, elevations of approximately 287 m for the west abutment and pier, and 289 m for the east abutment and pier.

HP310 x 110 Steel H-Piles for the abutment and pier extensions driven to the underlying very dense stratum 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 dense till. The toe of the pile is expected to settle less than 20 mm at the SLS value for piles end bearing within the very dense till. It is noted that the structural engineer will need to evaluate the elastic compression of the pile.



8.1.2 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 some settlement of the underlying soils. However the settlement is anticipated to be about 25 mm or less. Down drag forces are therefore not anticipated to be significant at the abutment locations.

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

8.1.3 Lateral Forces

Lateral forces could be fully or partially 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 | OPSS Granular B Type II | Native Sand and Gravel | Native Sand Silty Sand / Sandy Silt and Silt With Sand | Glacial Till |
|---------------------------------------|-------------------------|------------------------|---|--------------|
| Bulk Unit Weight (kN/m ³) | 21 | 20 | 19 | 21 |
| Effective friction angle | 35° | 32° | 30° | 32° |
| Coefficient of passive earth pressure | 3.69 | 3.26 | 3.00 | 3.26 |
| Design Undrained Shear Strength (kPa) | - | - | - | - |

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.

8.1.4 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 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

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.

8.1.5 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.



8.1.6 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 | 12 | HP310X110 | 140 |
| | 18 | HP310X110 | 160 |

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

8.1.7 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 12 m to 18 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 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, and must be driven below elevation 287 m for the east abutment and pier and 289 for the west abutment and pier.”

The piles should supplied and installed in accordance with SP903S01.

8.2 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 materials with less than 5% fines (passing the 75um 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 | Native Sand and Gravel | Sand Silty Sand / Sandy Silt and Silt With Sand |
|--|-----------------|--------------------------|------------------------|---|
| Unit Weight (kN/m ³) | 22 | 21 | 20 | 19 |
| Angle of Internal Friction, ϕ | 35° | 35° | 32° | 30° |
| Horizontal Backslope | | | | |
| Coefficient of Active Earth Pressure, K_a | 0.27 | 0.27 | 0.31 | 0.33 |
| Coefficient of Passive Earth Pressure, K_p | 3.69 | 3.69 | 3.26 | 3.00 |
| Coefficient of Earth Pressure at Rest, K_o | 0.43 | 0.43 | 0.47 | 0.50 |
| 2H:1V Backslope | | | | |
| Coefficient of Active Earth Pressure, K_a | 0.39 | 0.39 | 0.47 | 0.54 |
| Coefficient of Passive Earth Pressure, K_p | 10.84 | 10.84 | 8.62 | 7.48 |

8.3 Seismic Design

8.3.1 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 II as defined in the CHBDC Section 4.4.6 be used in the seismic design of this site.

8.3.2 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 8.1 Combined Coefficients of Static and Seismic Earth Pressure

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

8.4 Liquefaction

Low SPT N-values were reported for the silt deposit in both the 1958 and 1995 investigations. It is noted that the 1995 Record of Borehole sheets suggest that the low values are not representative due to disturbance. Nonetheless 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.

8.5 Embankment Design and Construction

The existing embankments, which are 8 m to 9 m in height 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 soils. The placement of approximately 1 m of fill in the central median will induce approximately 15 mm to 25 mm of settlement of the underlying soils. Given the sandy nature of the underlying soils, it is anticipated that the settlement will likely occur during the construction period.

The placement of the fill soils on the widening to the north and south of the existing embankments will likely induce minimal settlements on the underlying soils.

It is recommended that the fill for the central median be placed as soon as practical, preferably at the start of Stage 2 of the construction, to allow the majority of the settlement to occur before the bridge widening is constructed.

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. In addition, a mid-slope bench is recommended for embankments more than 8 m in height.

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 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 groundwater 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.

Flatter side slopes will be required for open cut excavations in loose sand and silty deposits below the ground water level unless appropriate dewatering methods are employed.

Where stable slopes noted above cannot be maintained or instability is noted, the side slopes should be flattened or temporary shoring should be provided.

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.

9.2 Staging

It is understood that the widening will be constructed in 5 Stages and in general two lanes of traffic in both directions will be maintained at all times. Pre-stage 1 will involve constructing the embankment widening to the south for the east bound lanes of the highway. Stage 1 will consist of moving the

eastbound traffic to the south (south side of the existing structure) and constructing the median widening and lanes 1 and 2 of the highway. Stage 2 will consist of shifting the eastbound traffic to the median and completing lane 3 and the shoulder of the eastbound widening. Stage 3 will consist of shifting both the east and west bound lanes to the south and constructing the westbound widening. Stage 4 will consist of moving the traffic to the correct east and west bound lanes, then constructing the central median barrier wall. Stage 5 is the completion of the project with all lanes open to traffic.

9.3 Shoring

Shoring may be required to support Highway 401 during the installation of the piles and pile caps at the abutment locations. Shoring may also be required to support the railway tracks during the installation of the piles and pile caps at the pier locations.

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

The temporary shoring may be designed using the lateral earth pressure parameters provided in Section 8.2, entitled Earth Pressure Design.

Shoring should meet the requirements of Performance Level 2 as per SP105S19. In addition, if excavations should extend below the level of the existing railroad tracks, shoring should be provided in accordance with the requirements of American Railway Engineering and Maintenance of Way Association as well as the Canadian Pacific Railway.

9.4 Groundwater Control

Ground water was encountered during drilling at depths ranging from approximately 0.9 m to 15.4 m below existing grade, elevations ranging from about 295.6 m to 299.5 m. Excavations below depths and elevations noted above will likely encounter groundwater. In addition, excavations above these elevations will likely encounter perched groundwater within the fill materials.

Given the soil conditions, seepage above the elevations noted above is anticipated to be slow and therefore should be readily handled by conventional sumps and pumping techniques. Excavations below the elevations noted above may be difficult given the presence of wet silts and sands. Therefore, some form of dewatering, in addition to conventional sumps and pumping techniques, such as well points and cut off walls may be required. An NSSP alerting the contractor to this issue should be included in the contract documents.

The following table provides an estimate of the permeability of the various soil types, based on the grain size distribution tests carried out:

| Soil Type | Co-Efficient of Permeability (cm/sec) |
|--------------------------------------|--|
| Fill (sand and silt) | 10^{-3} to 10^{-5} |
| Silty sand/sandy silt/silt with sand | 10^{-4} to 10^{-6} |
| Glacial Till | 10^{-4} to 10^{-7} |

9.5 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.

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 placing 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.

9.6 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 1.3 m.

9.7 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.

10.0 CLOSURE

A soil investigation is a limited sampling of a site. This report is based on the boreholes advanced at the site in 1958 and 1995. The information is gathered at specific borehole locations and can only be extrapolated to an undefined limited area around the locations.

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.

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.

Regards,

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/ct



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.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

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.

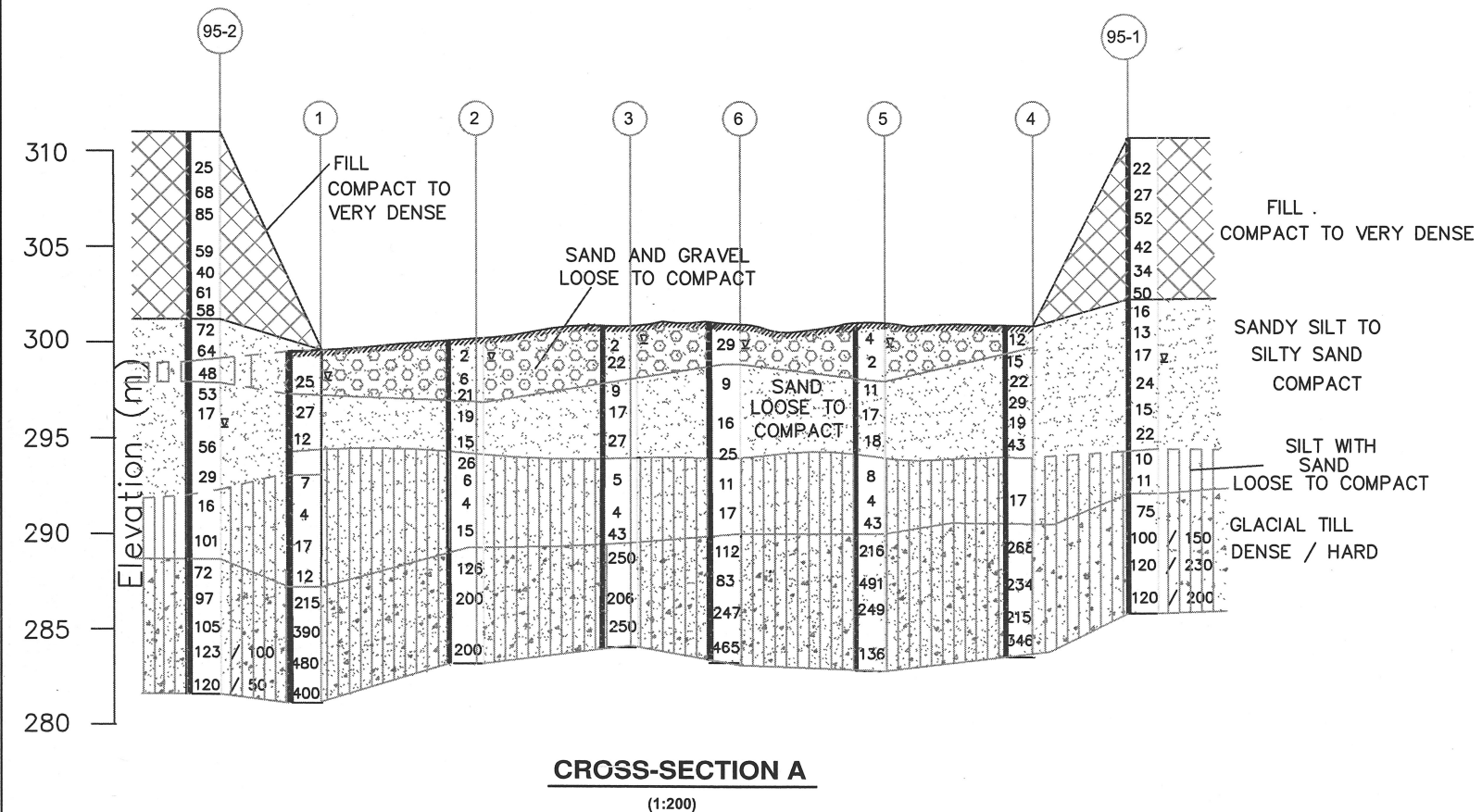
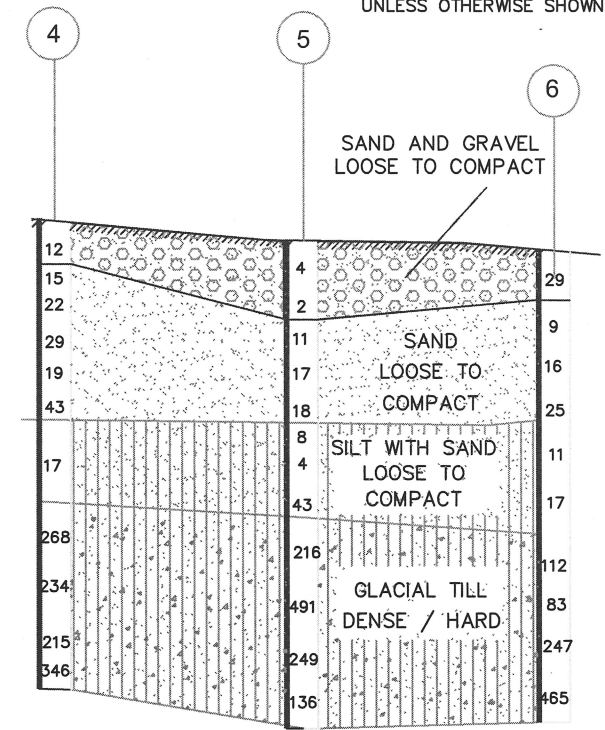
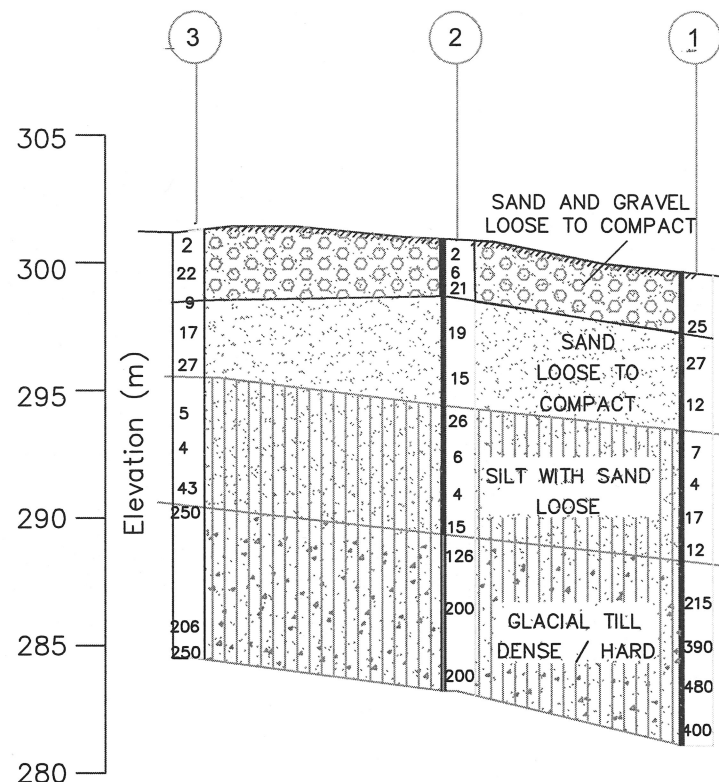
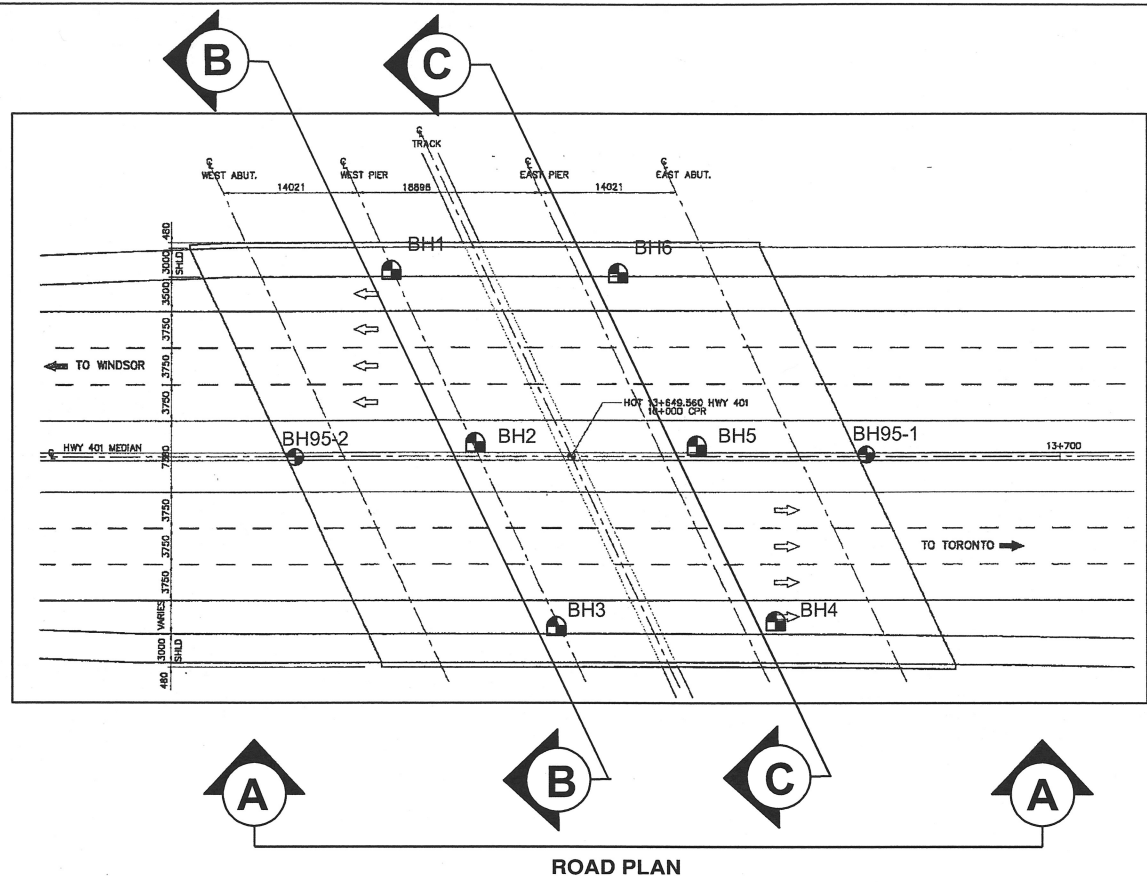
VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Jacques Whitford must be notified immediately to assess if the varying or unexpected conditions are substantial and if 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. 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.



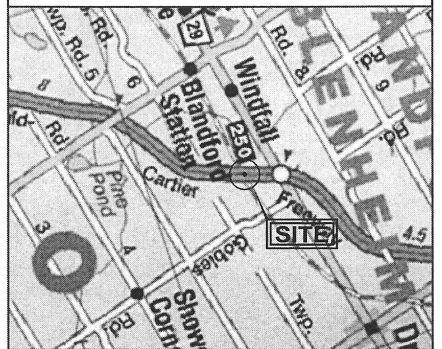
APPENDIX A

Drawings



CROSS-SECTION B & C
(1:200)

0 5 10 15m
SCALE 1:200



- LEGEND**
- BOREHOLE LOCATION
(BY OTHERS) 1958 INVESTIGATION
 - BOREHOLE LOCATION
(BY OTHERS) 1995 INVESTIGATION

| No | ELEVATION (m) | STATION | OFFSET |
|--------|---------------|----------|--------|
| BH1 | 299.6 | 13+629 | 20m LT |
| BH2 | 300.8 | 13+638 | 0 |
| BH3 | 301.3 | 13+647 | 18m RT |
| BH4 | 301.8 | 13+669 | 18m RT |
| BH5 | 301.7 | 13+661 | 0 |
| BH6 | 301.2 | 13+653 | 19m LT |
| BH95-1 | 310.7 | 13+680.8 | 0.0 |
| BH95-2 | 311.0 | 13+620.9 | 0.0 |

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.

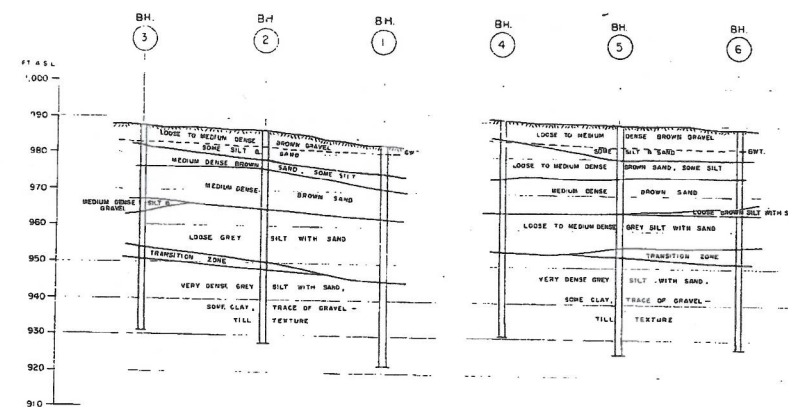
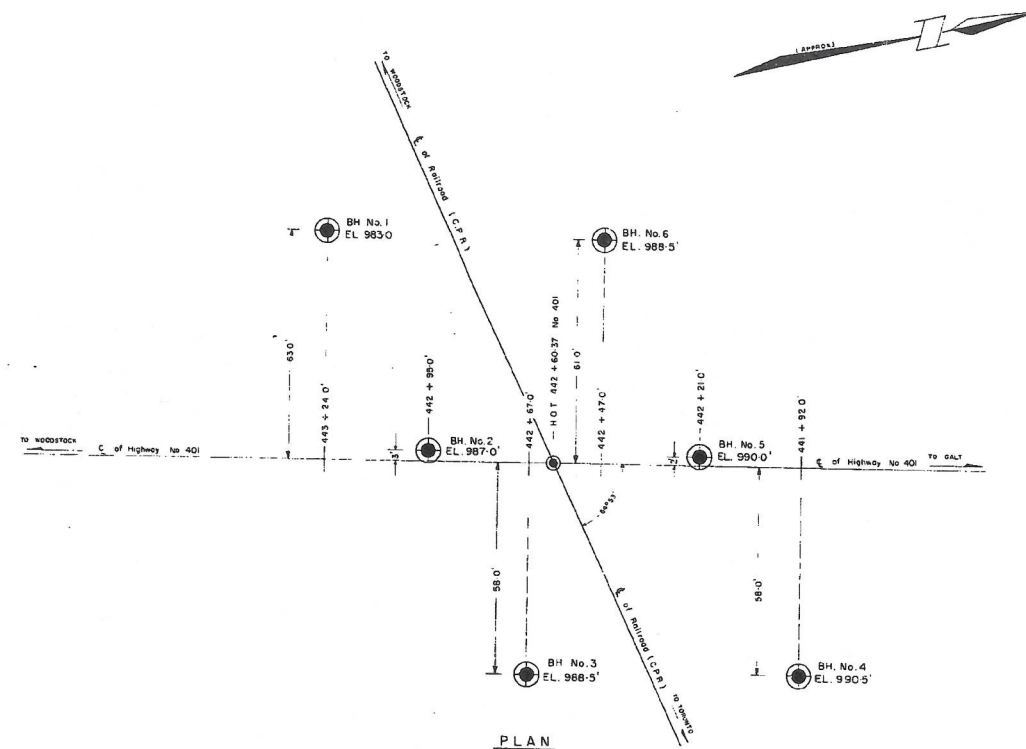
3) This Drawing is for subsurface information only. Surface details and features are for conceptual illustration only. The proposed structure location and features are shown for information only.

| DATE | BY | DESCRIPTION |
|------|----|-------------|
| | | |

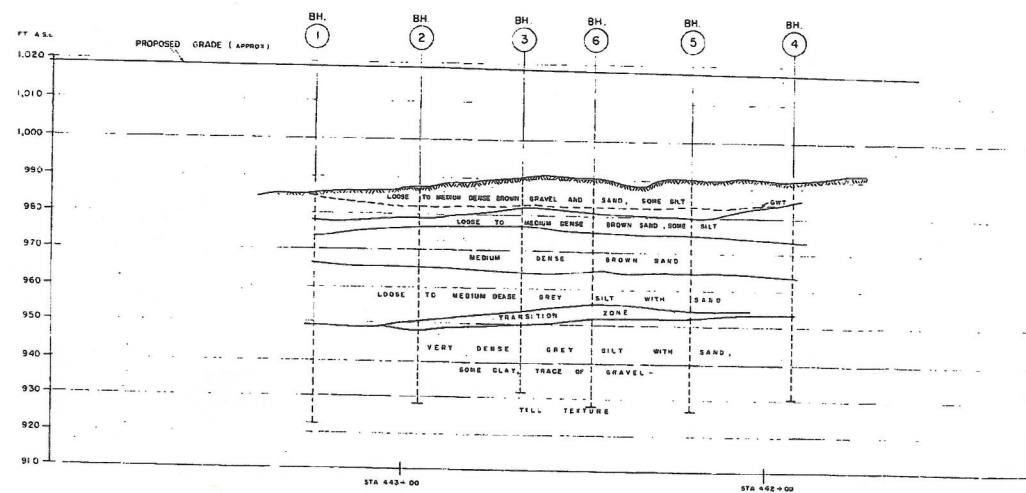
GEORES No 40P2 - 68

| HWY No 401 | SUBMITTAL | CHECKED | DATE | SITE |
|------------|-----------|---------|------------|------|
| | GC | | 2008-02-07 | |
| | HZ | | | |

DWG100821301GEO-058



SCALE - HOR. - 1" = 40'
VER. - 1" = 20'



| | | |
|--|-------------------|------------------|
| HUNTING TECHNICAL & EXPLORATION SERVICES LTD TORONTO | | |
| DEPARTMENT OF HIGHWAYS - ONTARIO | | |
| LOCATION OF BOREHOLES AND SUBSURFACE SOIL PROFILES FOR PROPOSED CROSSING OF HIGHWAY No. 401 OVERPASSING C.P.R. RAILROAD, BLENHEIM TWP. | | |
| BRIDGE SITE | | |
| SCALE: 1" = 20' H. (EXCEPT NOTED) | DRAWN BY - C.I.B. | DATE - SEPT 1998 |
| REFERENCE DRAWINGS - PROFILE F-3526-7 PLAN F-3526-9 | | |

APPENDIX B

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

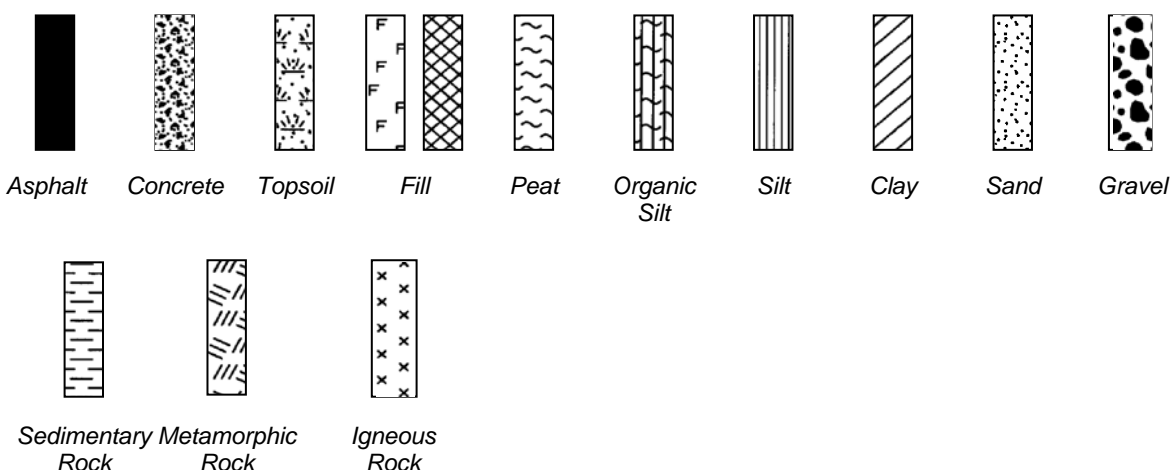
The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis. Standard Penetration Test 'N'-values* can also be used to provide 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 95-1

1 OF 1

METRIC

W.P. 819-93-01

LOCATION Sta. 13+680.8 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 11

CHECKED BY M.M.

| 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 | | 20 40 60 80 100 | 20 40 60 80 100 | | | | | |
| 310.7 | Ground Surface | | | | | | | | | | | | |
| 0.0 | | | | | | | | | | | | | |
| | Sand and Silt Trace to Some Gravel Interbedded layers of Clayey Silt (Fill) Compact to Very Dense | | 1 | SS | 22 | | | | | | | | 1 73 23 3 |
| | | | 2 | SS | 27 | | | | | | | | |
| | | | 3 | SS | 52 | | | | | | | | |
| | | | 4 | SS | 42 | | | | | | | | |
| | | | 5 | SS | 34 | | | | | | | | |
| 301.7 | Clayey Silt | | 6 | SS | 50 | | | | | | | | 1 48 39 12 |
| 9.0 | | | 7 | SS | 16 | | | | | | | | |
| | Sandy Silt to Silty Sand Trace Clay Compact | | 8 | SS | 13 | | | | | | | | |
| | | | 9 | SS | 17 | | | | | | | | 0 47 52 1 |
| | | | 10 | SS | 24 | | | | | | | | |
| | | | 11 | SS | 15 | | | | | | | | |
| | | | 12 | SS | 22 | | | | | | | | |
| | | | 13 | SS | 10 | | | | | | | | 0 23 73 4 |
| | | | 14 | SS | 11 | | | | | | | | |
| 291.4 | | | 15 | SS | 75 | | | | | | | | |
| 19.3 | Clayey Silt Trace Sand Trace Gravel (Glacial Till) Hard | | 16 | SS | 100 | /15cm | | | | | | | |
| | | | 17 | SS | 120 | /23cm | | | | | | | 2 7 73 18 |
| 285.9 | | | 18 | SS | 120 | /20cm | | | | | | | |
| 24.8 | End of Borehole * 'N' values within the Silty Sand to Sandy Silt deposit may be questionable due to blow up and disturbance during sampling. | | | | | | | | | | | | |

RECORD OF BOREHOLE No 95-2 1 OF 1 METRIC

W.P. 820-93-01 LOCATION Sta. 13+820.9 o/s 0.0 ORIGINATED BY M.M.
 DIST 31 HWY 401 BOREHOLE TYPE H. S. Augers COMPILED BY M.M.
 DATUM Geodetic DATE 95 01 16 CHECKED BY T.K.

| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL 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 (%) | | |
| 311.0 | Ground Surface | | | | | | | | | | | |
| 0.0 | Sand and Silt Trace to Some Gravel Interbedded layers of Clayey Silt [Fill] Compact to Very Dense | | 1 | SS | 25 | | | | | | | |
| | | | 2 | SS | 68 | | | | | | | |
| | | | 3 | SS | 85 | | | | | | | |
| | Clayey Silt | | 4 | SS | 59 | | | | | | | 11 47 28 14 |
| | | | 5 | SS | 40 | | | | | | | |
| | | | 6 | SS | 61 | | | | | | | 2 53 35 10 |
| 301.2 | Clayey Silt | | 7 | SS | 58 | | | | | | | |
| 9.8 | | | 8 | SS | 72 | | | | | | | |
| | | | 9 | SS | 64 | | | | | | | |
| | Some Gravel | | 10 | SS | 48 | | | | | | | 17 50 28 5 |
| | | | 11 | SS | 53 | | | | | | | |
| | Sandy Silt to Silty Sand Trace Clay Compact to Very Dense | | 12 | SS | 17 | | | | | | | |
| | | | 13 | SS | 56 | | | | | | | 0 92 6 2 |
| | | | 14 | SS | 29 | | | | | | | |
| | | | 15 | SS | 16 | | | | | | | |
| 288.7 | | | 16 | SS | 101 | | | | | | | |
| 22.3 | Clayey Silt Trace Sand Trace Gravel [Glacial Till] Hard | | 17 | SS | 72 | | | | | | | |
| | | | 18 | SS | 97 | | | | | | | 0 5 63 32 |
| | | | 19 | SS | 105 | | | | | | | |
| | | | 20 | SS | 123 | /10cm | | | | | | |
| 281.6 | | | 21 | SS | 120 | /5cm | | | | | | |
| 29.4 | End of Borehole * Water table not stabilized * 'N' values within the Sandy Silt to Silty Sand deposit may be questionable | | | | | | | | | | | |

+3, x5: Numbers refer to
Sensitivity

20
15-25 (%) STRAIN AT FAILURE
10

BONEHOLE No. 3

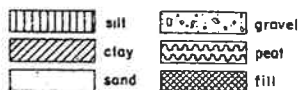
C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

SOME CLAY PRESENT AT THIS
ELEVATION

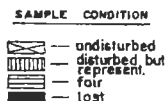
JOB No. H993/58 LOCATION NEAR ORUMBO - ONTARIO
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES CH. 442 + 95' 1 OFFSET 3' RT. OF
 ELEV. (surface) 982.1 (collar) _____ Datum D.H.O.
 BOREHOLE NUMBER 2
 DATE (started) _____ (finished) _____
 RIG No. 1 TYPE LONGYEAR JR. A.

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 2



x - standard penetr. 2 s.s.
 Δ - vane shear
 o - pocket penetrometer



S.S.W. - side slit
 S.S. - split spoon
 S.T. - Shelby tube
 T.W.P. - thin walled piston
 D.B. - diamond bit - rock core

C - consolidation test
 M - mechanical analysis
 T - triaxial shear
 K - permeability
 U - unconfined compression

| BORING LOG | | | | FIELD TESTS | | | | | | LABORATORY TESTS | | | |
|--------------|--------------|----------------------|---|---|---|-----|-------|-----------------------------|------|--|---|--------------------------------|---------|
| DEPTH FT. | ELEV. FT. | WATER OBSERVATION | LOG DESCRIPTION | SHEAR STRENGTH (TONS PER SQUARE FOOT) 1/2 1 1/2 | STANDARD PENETRATION TEST (BLOWS PER FOOT) 20 40 60 | No. | COND. | DEPTH FROM TO FT. FT. | TYPE | RECOVERY LENGTH REC. DIST. DRIV % | PENETRATION RESISTANCE (BLOWS PER FOOT) | ATTERBERG LIMITS WP X - C W | REMARKS |
| 0 | 987.1 | | ORGANIC MATERIAL AND SAND | | | 1 | | 10 | 1.5 | S.S. | 2 | | |
| 5 | | | LOOSE TO MEDIUM DENSE BROWN GRAVEL SOME SILT AND SAND | | | 2 | | 2.5 | 4.0 | S.S. | 6 | | |
| 8.0 | 979.1 | | | | | 3 | | 5.5 | 7.0 | S.S. | 15/18 | 21 | |
| 10 | | | MEDIUM DENSE BROWN SAND, SOME SILT | | | 4 | | 10.0 | 11.5 | S.S. | 15/18 | 19 | |
| 15 | | | MEDIUM DENSE BROWN SAND | | | 5 | | 15.0 | 16.5 | SSW | | 15 | |
| 20 | | | | | | 6 | | 20.0 | 21.5 | S.S. | 14/18 | 26 | |
| 25 | | | LOOSE TO MEDIUM DENSE GREY SILT WITH SAND | | | 7 | | 25.0 | 26.5 | S.S. | 14/18 | 6 | |
| 30 | | | | | | 8 | | 30.0 | 31.5 | S.S. | 8/15 | 4 | |
| 35 | 961.0 | | | | | 9 | | 35.0 | 36.5 | S.S. | 11/18 | 15 | |
| 38.5 | 948.6 | | TRANSITION ZONE | | | 10 | | 40.0 | 41.5 | S.S. | 8/18 | 126 | |
| 40 | | | VERY DENSE GREY | | | 11 | | 48.0 | 49.5 | S.S. | 13/18 | 200 | |
| 45 | | | SILT WITH SAND SOME CLAY | | | | | | | | | | |
| 50 | | | TRACE OF GRAVEL | | | | | | | | | | |
| 55 | | | - TILL TEXTURE | | | | | | | | | | |
| 58.4 | 928.7 | | END OF BORING | | | 12 | | 58.0 | 59.4 | S.S. | 5/5 | 200 | |

JOB No. H593/58 LOCATION NEAR DRUMBO - ONTARIO
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES CH. 442 + 67.0, OFFSET 58-0' OF 4
 ELEV. (surface) 988.6 (collar) _____ Datum R.H.O.
 BOREHOLE NUMBER 3
 DATE (started) _____ (finished) _____
 R/G No. 1 TYPE LONGYEAR J.N.A.

HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 3



x — standard penetr. z.s.s.
 Δ — vane shear
 o — pocket penetrometer

SAMPLE CONDITION








S.S.W. — side slit
 S.S. — split spoon
 S.T. — shelly tube
 T.W.P. — thin walled piston
 D.B. — diamond bit

C — consolidation test
 M — mechanical analysis
 T — triaxial shear
 K — permeability
 U — unconfined compression

| BORING LOG | | | | FIELD TESTS | | | | | | | | | | LABORATORY TESTS | | | |
|------------|-------|-------|-------------------|-------------|--|---|---|-------|---------|-------|------------|----------|--------|---------------------------------|---|--|---------|
| SCALE | DEPTH | ELEV | WATER OBSERVATION | LOG | DESCRIPTION | SHEAR STRENGTH (TONS PER SQUARE FOOT) | | | SAMPLES | | | | | ATTERBERG LIMITS WP X — O WL | | | REMARKS |
| FT | FT | FT | | | | 1/2 | 1 | 1 1/2 | N. | COND. | DEPTH FROM | DEPTH TO | TYPE | RECOVERY LENGTH REC. DIST. DRV. | PENETRATION RESISTANCE (BLOWS PER FOOT) | | |
| | | | | | | STANDARD PENETRATION TEST X (BLOWS PER FOOT) | | | | | FT. | FT. | | % | | | |
| 0 | 0 | 988.5 | | | | | | | | | | | | | | | |
| | 1-3 | 987.2 | | | DECOMPOSED ORGANIC MATERIAL | | | | 1 | | 0 | 1-5 | S.S. | 10/18 | 2 | | |
| | 5-5 | 983.0 | | | LOOSE TO MEDIUM DENSE BROWN GRAVEL, SOME SILT & SAND | | | | 2 | | 5-0 | 6-5 | S.S. | 10/18 | 22 | | |
| | 10 | 977.4 | | | MEDIUM DENSE TO LOOSE BROWN SAND, SOME SILT | | | | 3 | | 10-0 | 11-5 | S.S. | 18/18 | 9 | | |
| | 15 | | | | MEDIUM DENSE BROWN SAND | | | | 4 | | 15-0 | 16-5 | S.S.W. | | 17 | | |
| | 20 | 967.0 | | | MEDIUM DENSE SILT AND GRAVEL | | | | 5 | | 20-0 | 21-5 | S.S. | 8/18 | 27 | | |
| | 24-0 | 964.5 | | | LOOSE GREY SILT WITH SAND | | | | 6 | | 25-0 | 26-5 | S.S. | 16/18 | 5 | | |
| | 30 | | | | TRANSITION PHASE | | | | 7 | | 30-0 | 31-5 | S.S. | 15/18 | 4 | | |
| | 34-0 | 954.5 | | | VERY DENSE GREY SILT WITH SAND | | | | 8 | | 35-0 | 36-5 | S.S. | 14/18 | 43 | | |
| | 37-0 | 951.5 | | | SOME CLAY | | | | 9 | | 39-0 | 40-0 | S.S. | 10/12 | 250 | | |
| | 40 | | | | TRACE OF GRAVEL | | | | 10 | | 50-0 | 51-0 | S.S. | 6/12 | 206 | | |
| | 45 | | | | TILL TEXTURE | | | | 11 | | 55-0 | 56-2 | S.S. | 13/14 | 250 | | |
| | 50 | 932.3 | | | END OF BORING | | | | | | | | | | | | |
| | 56-2 | | | | | | | | | | | | | | | | |
| | 60 | | | | | | | | | | | | | | | | |
| | 63 | | | | | | | | | | | | | | | | |
| | 70 | | | | | | | | | | | | | | | | |
| | 75 | | | | | | | | | | | | | | | | |
| | 80 | | | | | | | | | | | | | | | | |

PENETRATION RESISTANCE = 12 BLOWS/FT.
 PENETRATION RESISTANCE = 27 BLOWS/FT.

BOREHOLE No. 4

| | | | |
|---|------|---|--------|
|  | sill |  | gravel |
|  | clay |  | peat |
|  | sand |  | fill |

x — standard penetr. 2 s.s.
 a — vane shear
 o — pocket penetrometer

| SAMPLE | CONDITION |
|---|----------------------------|
|  | — and disturbed |
|  | — disturbed but represent. |
|  | — fair |
|  | — lost |

S.S.W. - side slit
S.S. - split spoon
S.T. - Shelby tube
T.W.P. - thin walled piston
D.B. - diamond bit - rock core

C — consolidation test
M — mechanical analysis
T — triaxial shear
K — permeability
U — unconfined compression

BORING LOG

FIELD TESTS

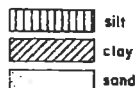
LABORATORY TESTS

| TEST LOG | | | | | | | | | | LABORATORY | | TESTS | | | | |
|--------------|--------------|--------------|----------------------|-----|--|--|-------|---------|-------|-------------|-----------|-------|--|---|---|---------|
| SCALE FT. | DEPTH FT. | ELEV. FT. | WATER OBSERVATION | LOG | DESCRIPTION | SHEAR STRENGTH (TONS PER SQUARE FOOT) | | SAMPLES | | | | | | | ATTENBERG LIMITS wp x - o wl ● - NATURAL WATER CONTENT | REMARKS |
| | | | | | | 1/2 | 1 1/2 | No. | COND. | DEPTH | | TYPE | RECOVERY LENGTH REC. DIST. DRV. % | PENETRATION RESISTANCE (BLOWS PER FOOT) | | |
| | | | | | | | | | | FROM FT. | TO FT. | | | | | |
| 0 | 0 | 990.5 | | | TOPSOIL | | | | | | | | | | | |
| 5 | 5.5 | 985.0 | | | MEDIUM DENSE BROWN SAND AND GRAVEL TRACE OF ORGANIC | | | 1 | III | 1-0 | 2-5 | S.S. | 17/18 | 12 | | |
| 10 | | | | | MEDIUM DENSE BROWN SAND, SOME SILT | | | 2 | III | 5-0 | 6-5 | S.S. | 12/18 | 15 | | |
| 15 | 16.0 | 974.5 | | | | | | 3 | III | 9-9 | 11-4 | S.S. | 12/18 | 22 | | |
| 20 | | | | | MEDIUM DENSE BROWN SAND | | | 4 | III | 15-1 | 16-6 | S.S. | 13/18 | 29 | GRAVEL | STRATUM |
| 25 | 25.8 | 964.7 | | | | | | 5 | III | 20-1 | 21-6 | SSW | | 19 | | |
| 30 | | | | | MEDIUM DENSE GREY SILT WITH SAND | | | 6 | III | 25-0 | 26-5 | S.S. | 14/18 | 43 | GRAVEL | STRATUM |
| 35 | 36-1 | 954.4 | | | | | | 7 | III | 30-0 | 31-5 | S.S. | 13/18 | 17 | | |
| 40 | | | | | VERY DENSE GREY SILT WITH SAND SOME CLAY TRACE OF GRAVEL - TILL TEXTURE | | | 8 | III | 35-0 | 36-5 | S.S. | 16/18 | | PENETRATION RESISTANCE = 28 BLOWS/FOOT PENETRATION RESISTANCE = 81 " " | |
| 45 | | | | | | | | 9 | III | 40-2 | 41-7 | S.S. | 14/18 | 268 | | |
| 50 | | | | | | | | 10 | III | 46-0 | 47-5 | S.S. | 14/18 | 234 | | |
| 55 | | | | | | | | 11 | III | 52-0 | 53-5 | S.S. | 13/18 | 215 | | |
| 60 | 59.5 | 931.0 | | | END OF BOREHOLE | | | 12 | III | 58-0 | 59-5 | S.S. | 12/18 | 346 | | |

JOB No. H583/58 LOCATION NEAR DRUMBO - ONTARIO
 CLIENT DEPARTMENT OF HIGHWAYS - ONTARIO
 COORDINATES CH. 442 + 210' ; OFFSET 20 FT. OF
 ELEV. (surface) 989.8 (collar) Datum D.M.D.
 BOREHOLE NUMBER 5
 DATE (started) 4 SEPT. 1958 (finished) 6 SEPT. 1958
 RIG No. 2 TYPE LONGYEAR JR. A

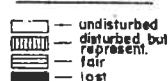
HUNTING TECHNICAL AND EXPLORATION SERVICES

BOREHOLE No. 5



x - standard penetr. 2 s.s.
 Δ - vane shear
 o - pocket penetrometer

SAMPLE CONDITION



S.S. - split spoon
 S.T. - Shelby tube
 T.W.P. - thin walled piston
 D.B. - diamond bit

C - consolidation test
 M - mechanical analysis
 T - triaxial shear
 K - permeability
 U - unconfined compressive

| BORING LOG | | | | | FIELD TESTS | | | | | | | | | | LABORATORY TESTS | | | |
|------------|-------|-------|-------------------|-----|---|---|-------|---------|-------|------------|------|------|----------------------------------|---|---------------------------|------------------|--|---------|
| SCALE | DEPTH | ELEV. | WATER OBSERVATION | LOG | DESCRIPTION | SHEAR STRENGTH (TONS PER SQUARE FOOT) | | SAMPLES | | | | | | | | ATTENBERG LIMITS | | REMARKS |
| FT | FT | FT | | | | 1/2 | 1 1/2 | N | COND. | DEPTH FROM | TO | TYPE | RECOVERY LENGTH REC. DIST. DRIV. | PENETRATION RESISTANCE (BLOWS PER FOOT) | C — NATURAL WATER CONTENT | | | |
| | | | | | | STANDARD PENETRATION TEST (BLOWS PER FOOT) | | | | | | | | | | | | |
| | | | | | | 20 | 40 | 60 | | | | | | | | | | |
| 0 | 0 | 989.8 | | | ORGANIC MATERIAL & SAND | | | | | | | | | | | | | |
| | 1.0 | 988.8 | | | | | | | 1 | | 1-0 | 2-5 | S.S. | 18/18 | 4 | | | |
| 5 | | | | | LOOSE BROWN SAND & GRAVEL | | | | 2 | | 5-0 | 6-5 | S.S. | 10/18 | 2 | | | |
| 10 | 9.7 | 980.1 | | | | | | | | | | | | | | | | |
| | | | | | MEDIUM DENSE BROWN SAND SOME SILT | | | | 3 | | 9-7 | 11-2 | S.S. | 15/18 | 11 | | | |
| 15 | 14.7 | 975.1 | | | | | | | | | | | | | | | | |
| | | | | | MEDIUM DENSE BROWN SAND | | | | 4 | | 14-7 | 16-2 | S.S. | 12/18 | 17 | | | |
| 20 | | | | | | | | | | | | | | | | GRAVEL STRATUM | | |
| | 24.7 | 965.1 | | | | | | | 5 | | 19-7 | 21-2 | S.S. | 13/18 | 18 | | | |
| 25 | | | | | | | | | | | | | | | | | | |
| | | | | | LOOSE GREY SILT WITH SAND | | | | 6 | | 24-7 | 26-2 | S.S. | 10/18 | 8 | | | |
| 30 | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | 7 | | 29-7 | 31-2 | S.S. | 17/18 | 4 | 0 34 63 3 | | |
| 35 | 34.1 | 955.7 | | | | | | | | | | | | | | | | |
| | 35.7 | 953.1 | | | TRANSITION ZONE | | | | 8 | | 34-7 | 36-2 | S.S. | 15/18 | 43 | | | |
| 40 | | | | | | | | | | | | | | | | | | |
| | | | | | VERY DENSE GREY SILT WITH SAND SOME CLAY TRACE OF GRAVEL TILL TEXTURE | | | | 9 | | 39-5 | 41-0 | S.S. | 14/18 | 216 | 7 27 53 13 | | |
| 45 | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | 10 | | 46-0 | 47-0 | S.S. | 18/18 | 491 | | | |
| 50 | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | |
| 55 | | | | | | | | | 11 | | 55-7 | 57-2 | S.S. | 17/18 | 249 | 1 18 50 25 | | |
| 60 | | | | | | | | | | | | | | | | | | |
| | 63.1 | 926.5 | | | END OF BORING | | | | 12 | | 61-8 | 63-2 | S.S. | 14/18 | 136 | | | |
| 65 | | | | | | | | | | | | | | | | | | |
| 70 | | | | | | | | | | | | | | | | | | |
| 75 | | | | | | | | | | | | | | | | | | |
| 80 | | | | | | | | | | | | | | | | | | |

0 34 63.3

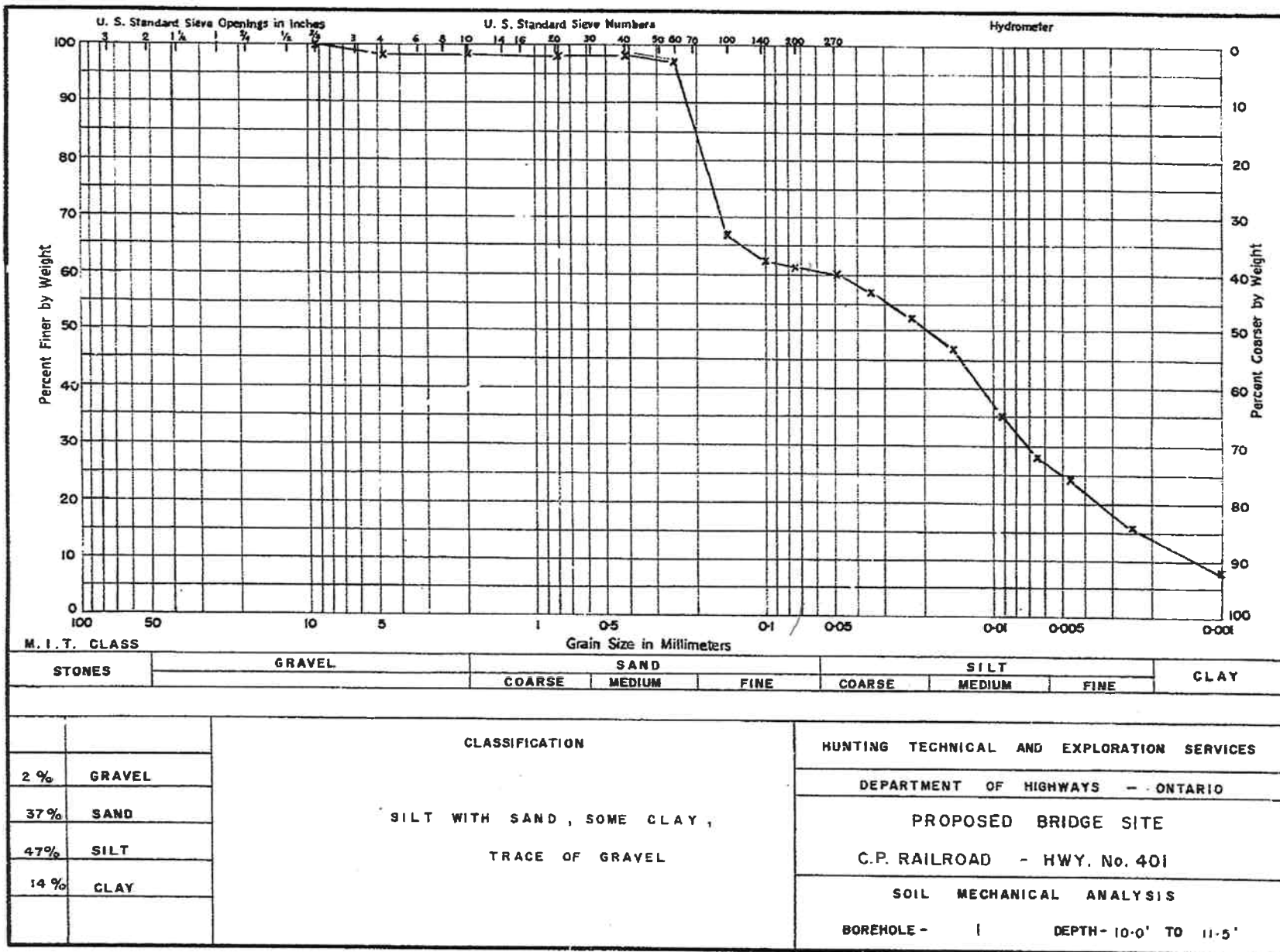
7 27 53.13

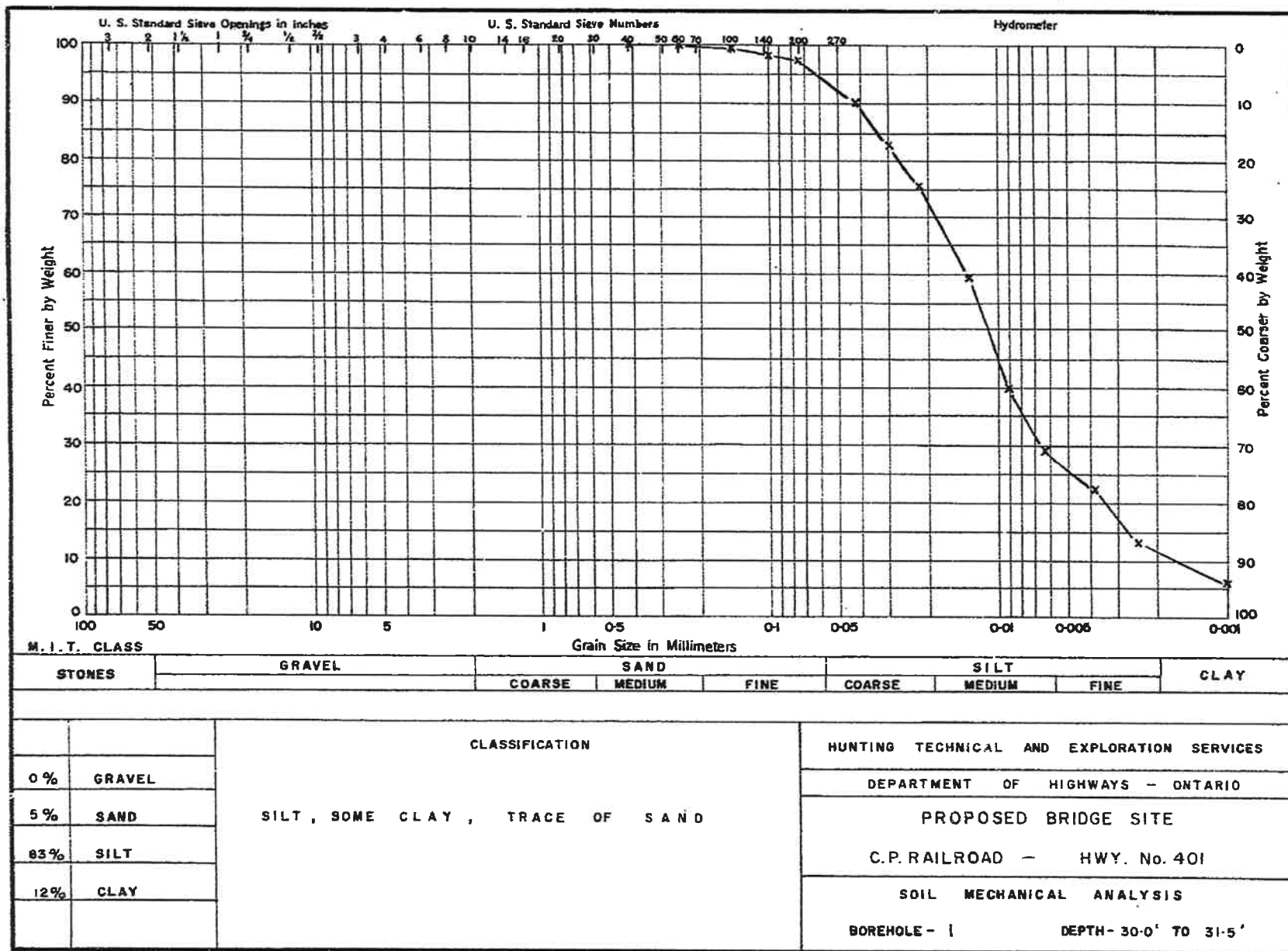
1 18.50.25

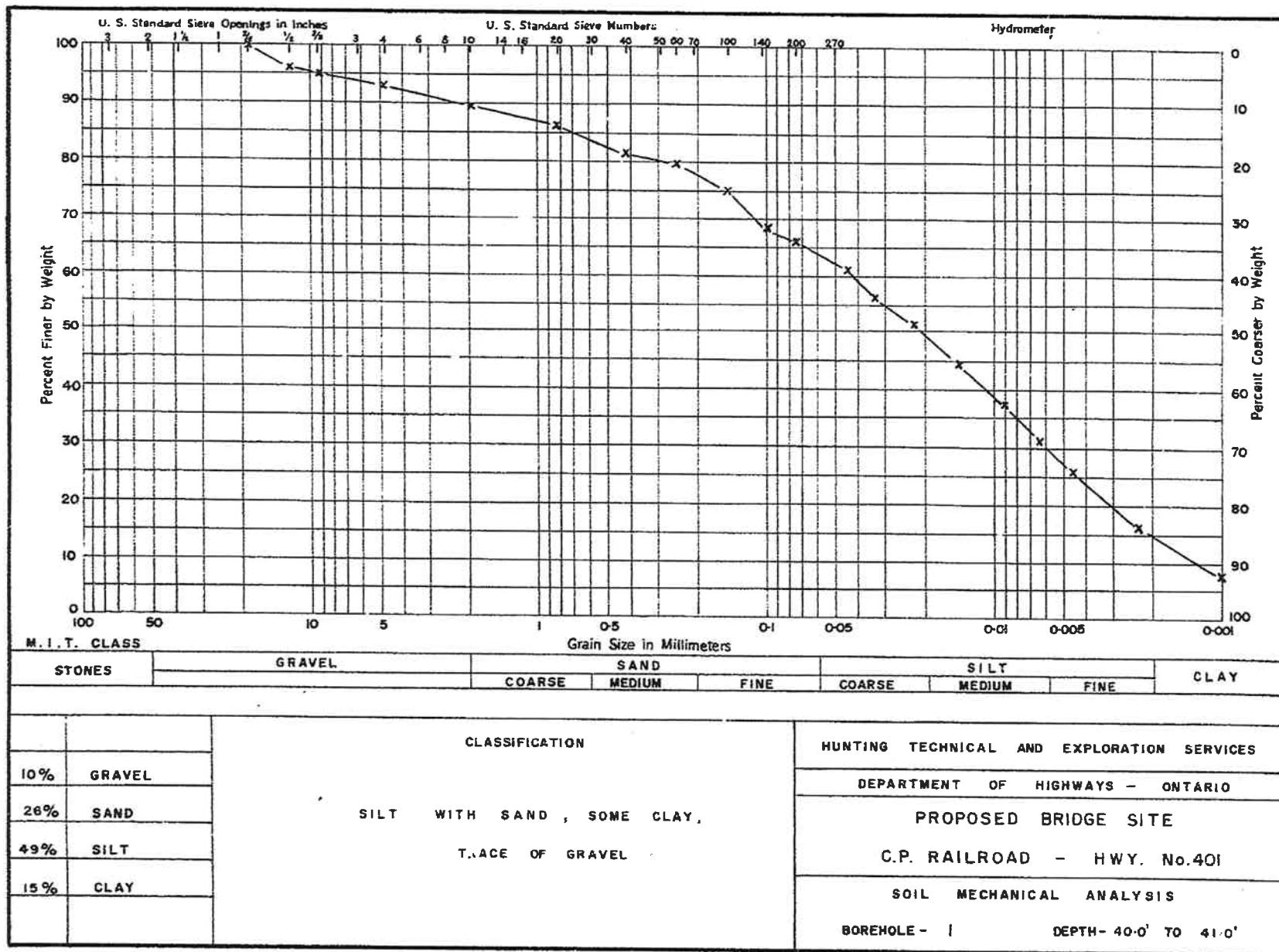
GRAVEL STRATUM

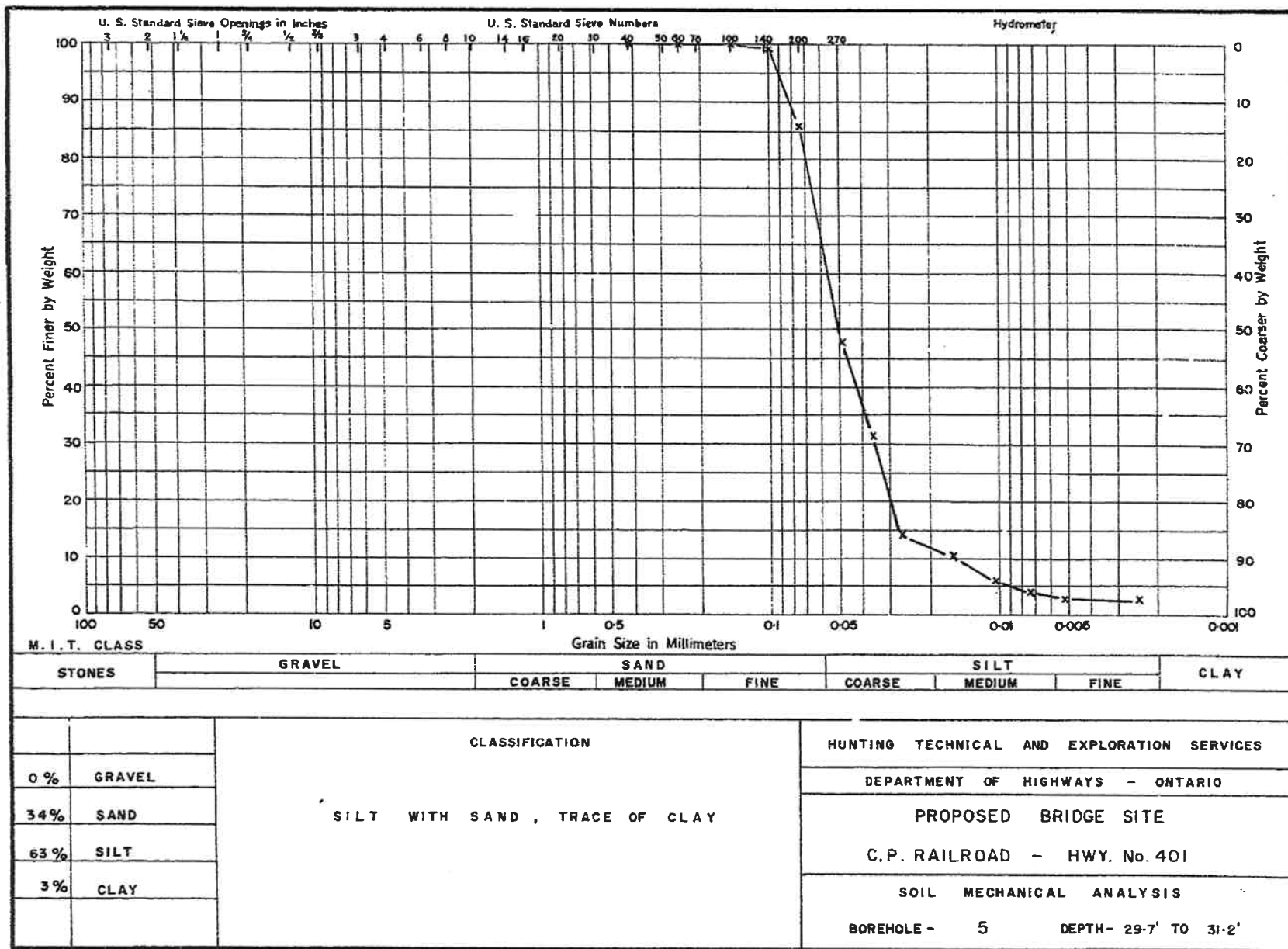
APPENDIX C

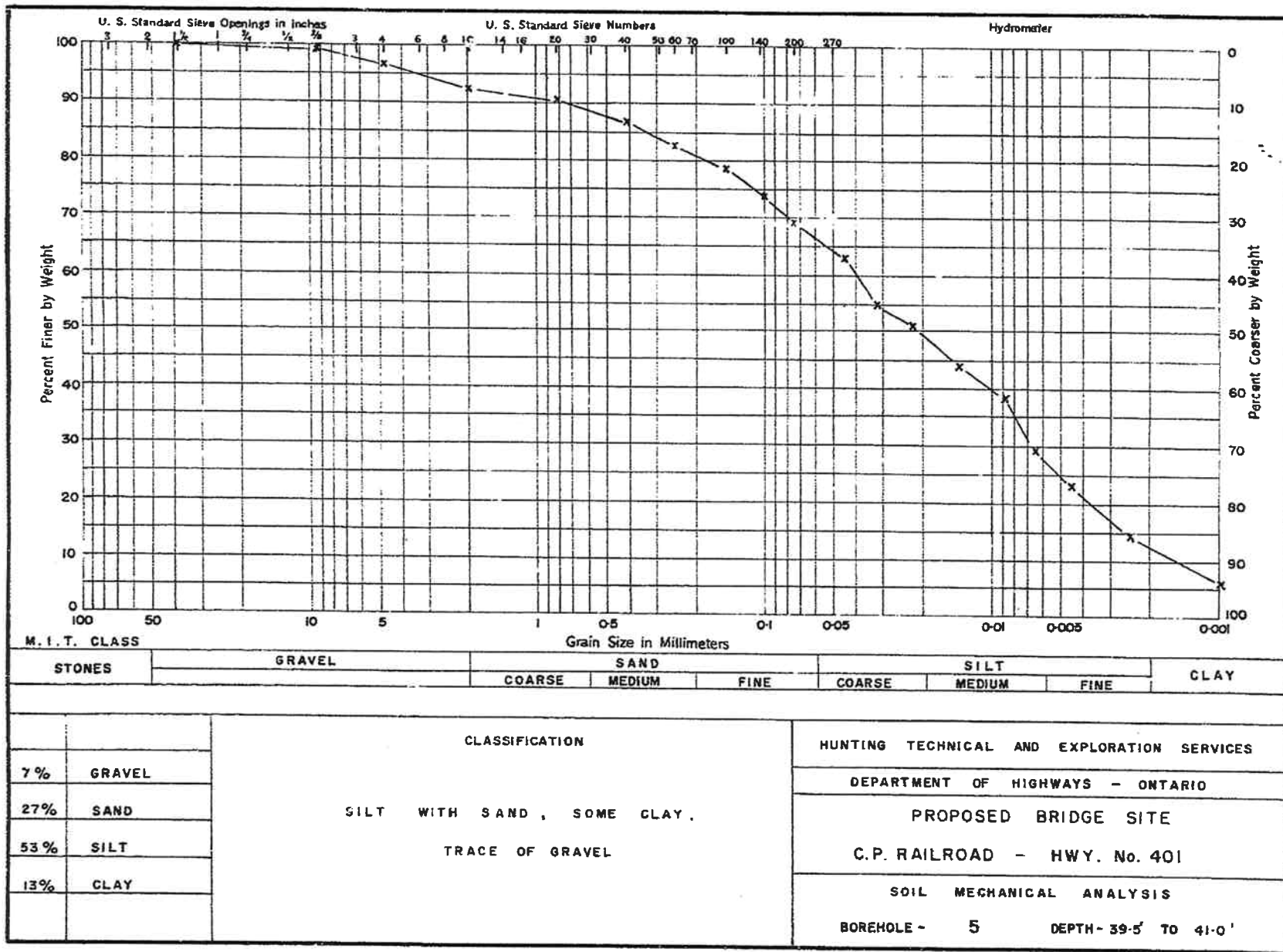
Laboratory Testing Results (1958 Investigation)

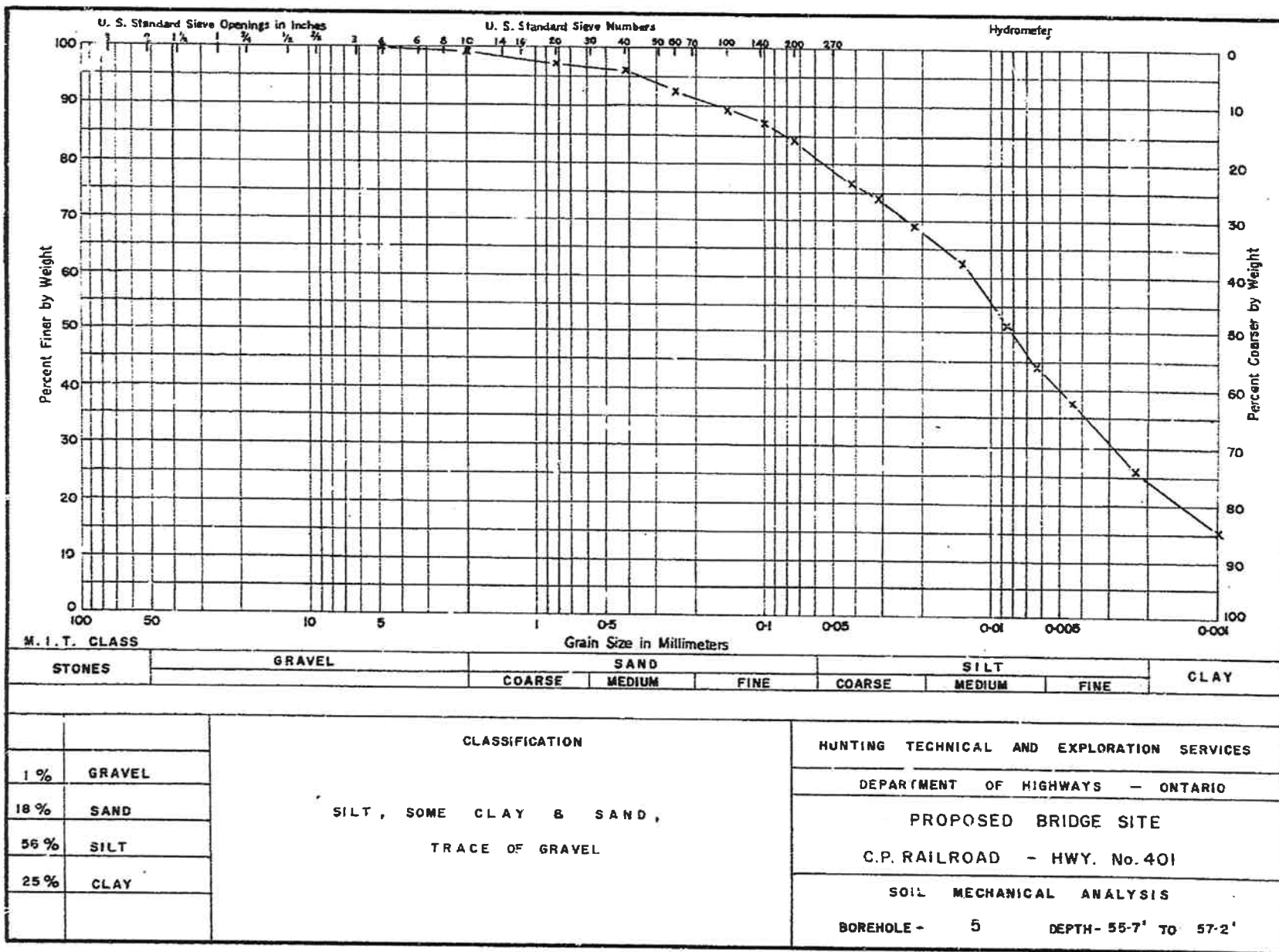












APPENDIX D

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 Canadian Pacific Railway Overhead Widening on Highway 401 in the Township of Blanford-Blenheim, Ontario.

Standard Drawings:

OPSD3000.100
OPSD3000.15
OPSD3101.150
OPSD3102.100
OPSD208.010

Standard Specifications; OPSS572

Special Provisions:

SS-103-11
SP902S01
SP206S03
SP105S19

