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# **FOUNDATION INVESTIGATION AND DESIGN REPORT**

W.P. 545-93-00  
HIGHWAY 60 –  
KEARNEY CREEK  
BRIDGE REPLACEMENT

McCormick Rankin Corporation

PROJECT NO. 1023332  
GEOCRE NO. 31F-273

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# **PROJECT NO. 1023332**

## **FOUNDATION INVESTIGATION AND DESIGN REPORT**

**TO**

**McCormick Rankin Corporation**  
**2655 North Sheridan Way**  
**Mississauga, Ontario**  
**L5K 2P8**

**ON**

**W.P. 545-93-00**  
**Highway 60 – Kearney Creek**  
**Bridge Replacement**  
**County of Nipissing**  
**District 43, Bancroft**  
**Ministry of Transportation**  
**Ontario**  
**Geocres No. 31F-273**

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**October 2007**

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# FOUNDATION INVESTIGATION REPORT

for

W.P. 545-93-00  
Highway 60 – Kearney Creek Bridge  
County of Nipissing  
District 43, Bancroft

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## 1.0 INTRODUCTION

This report was prepared as part of the Total Project Management (TPM) assignment for the Detailed Design of the Clarke Creek and Kearney Creek Bridge Replacements, Highway 60, G.W.P. 545-93-00.

This report presents the results of a foundation investigation carried out for the proposed replacement of the existing Kearney Creek Bridge on Highway 60 in Algonquin Park (Site No. 43-145).

The foundation investigation was carried out in general accordance with our proposal number 1019534 dated December 5, 2006. Authorization to proceed was provided by the Ministry of Transportation of Ontario (MTO) under Agreement Number 4006-E-0018 with McCormick Rankin Corporation (MRC), the Detailed Design Consultant for this project.

This report has been prepared specifically and solely for the project described herein. It contains factual information pertaining to the subsurface conditions which was obtained as part of this investigation.

It is noted that a Preliminary Foundation Investigation of this site was carried out by Jacques Whitford Limited. The relevant results from Report No. ONO11685 dated June 2006 have been included in the present report.

---

## 2.0 SITE DESCRIPTION AND GEOLOGY

The subject site is within the limits of MTO project W.P. 545-93-00 (Highway 60). The site location is shown on the Key Plan inset to Drawing No. 1 provided in Appendix A. It is noted that for project orientation purposes, Highway 60 will be assumed to run east-west at the Kearney Creek Bridge, with chainage increasing from west to east.

Physiographically, the Kearney Creek Crossing is located within the Algonquin Highlands. This region is characterized by rough rounded knobs and ridges with frequent outcrops of bare rock. The bedrock is generally shallow, however, the depth to bedrock varies greatly over short distances.

Many of the valleys are floored with outwash sand and gravel. There are frequent swamps and bogs.

Kearney Creek flows from north to south and is approximately 9 m in width at the centreline of the proposed realignment. Water depths were estimated to be less than 1 m at the time of the investigation. The observed water level at the time of the 2007 field investigation was approximately 392.0 m Geodetic. The high water level (100-year storm) is identified in the Structural Planning Report as being elevation 393.20 m.

The existing roadway embankments are approximately 3 m high at both the east and west abutments. The water level in Kearney Creek was approximately 3.5 m below the top of pavement on the existing bridge deck at the time of the 2007 investigation. The banks of the creek are steeply sloped for approximately 1 m above water level and then very gradually sloped upwards away from the creek. No indications of significant erosion were noted at the time of the site inspection. The ground surface within the highway right-of-way was vegetated with grass. Mature trees are present beyond the edges of the cleared right-of-way. Drainage in the area consisted of overland flow directed towards the creek.

A plan view and profile are shown on Drawing No. 1, provided in Appendix A.

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## 3.0 PROCEDURE

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### 3.1 Field Investigation

The preliminary investigation consisted of eight (8) boreholes designated as 05-1 through 05-8. The details concerning the field procedures for those boreholes is documented in the June 2006 Preliminary Foundation Investigation Report.

The site soil conditions were further investigated in 2007 with a borehole drilling investigation, piezocone (CPTu) investigation and laboratory testing program. The borehole drilling and CPTu testing was carried out using a track-mounted CME-55 drill rig between April 30 and May 10, 2007.

A total of two (2) boreholes, designated as 07-7 and 07-9 were put down during the field investigation. Boreholes 07-7 and 07-9 were advanced at the proposed west and east abutment locations, respectively, of the replacement bridge structure along the proposed permanent re-alignment.

The 2007 boreholes were advanced through the overburden using casing and drilling mud in order to balance the pressure within the borehole and minimize sand coming up the augers. Despite the use of casing and thick drilling mud, frequent problems were encountered with sand/silt coming up inside the casing.

The subsurface conditions were identified in the field by Jacques Whitford Limited (JW) personnel from samples obtained while carrying out Standard Penetration Tests (SPT) (ASTM D1586) at regular intervals. The boreholes were advanced through boulders and into bedrock by coring a minimum of 4.9 m using NQ-size coring equipment.

The recovered soil samples were stored in moisture proof containers and returned to our laboratory. The subsurface conditions encountered are described in detail in the Borehole Records presented in Appendix B.

Groundwater levels were measured in the open boreholes prior to backfilling.

Two CPTu test holes, designated as CPT 07-8 and CPT 07-10, were put down approximately 5 to 10 m behind the proposed west and east abutments, respectively. The piezocone was pushed through the native silt and sand materials using the hydraulic system on the drill rig until refusal. The testing technique is described in further detail in ASTM D3441. In this case, refusal was reached when the piezocone tip resistance was sufficient to cause the drill rig to start to lift up from the ground.

Prior to completing the investigation, the boreholes were grouted with a cement/bentonite mix.

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### 3.2 Survey

Borehole locations were established in the field by measurement by JW personnel relative to existing site features such as the existing bridge structure. The ground surface elevations at the borehole locations were surveyed relative to the top of asphalt on the deck of the existing Kearney Creek bridge structure adjacent to the west abutment. The top of pavement at this location has been identified as having a geodetic elevation of 395.3 m.

---

### 3.3 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual classification by a geotechnical engineer. Routine testing, consisting of moisture content testing and grain size distribution analysis, was carried out on representative samples. Two soil samples were submitted for pH, sulphate and resistivity testing to assess the potential for corrosion of buried steel and the potential for sulphate attack on buried concrete. Four samples had previously been analyzed as part of the preliminary investigation.

No complex testing was deemed to be necessary based on the soil conditions.

All soil samples will be stored for a period of one year after issuance of the final version of the preliminary foundation investigation report. Unless otherwise directed, the stored samples will be disposed of after this period.

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## 4.0 SUBSURFACE CONDITIONS

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### 4.1 Subsurface Profile

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided. The results of the CPTu testing are also presented in Appendix B along with an explanation of terminology used on CPTu/SCPTu Records.

Borehole Records from the preliminary foundation investigation report for this project have been included in this report for completeness.

In general, the subsurface profile beneath the proposed re-alignment (Boreholes 05-1 to 05-4, 07-7 and 07-9 and CPT 07-8 and 07-10) consists of a deep deposit of poorly-graded silt with sand, silty sand, silt and sandy silt over a thin layer of bouldery till over bedrock.

A Borehole Location Plan and Stratigraphic Section of the soils encountered within the boreholes are provided on Drawing No. 1 in Appendix A.

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#### 4.1.1 Fill: Silty Sand to Gravelly Sand with Silt

Granular fill was encountered beneath the asphalt in all of the boreholes located along the existing Highway 60 alignment (05-5 to 05-8). The composition of the fill ranged from gravelly sand, trace to some silt, to sand, some silt, trace gravel. Woody organic matter was observed in the fill deposit in Boreholes 05-4 and 05-6. The thickness of the fill, where present, varied from 3.6 m to 4.4 m. The underside of the fill was observed to range from elevation 390.9 m to 391.8 m. The upper portion of the fill was frozen to a depth of approximately 1.4 m at the time of the 2005 preliminary investigation. The moisture content of the 8 samples of fill tested ranged from 5% to 34% and averaged 18%. The results of three grain size analyses indicated that the samples contained 1% to 8% gravel, 83% to 85% sand and 8% to 16% fines. The gradation results are provided on Figure 1 in Appendix B.

The SPT 'N' values ranged from 2 to 55 (excluding the results within the upper frozen zone) with an average value of 12 indicating that the fill was generally compact. The asphalt surface overlying the fill was observed to be 75 mm to 120 mm thick at the borehole locations.

A 100 mm thick organic layer was observed beneath the fill in Borehole 05-8.

A 300 mm thick layer of loose sand with cobbles was observed beneath the topsoil in Borehole 07-9 which was located along the drainage ditch at the toe of the existing highway embankment.



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#### 4.1.2 Poorly-Graded Sand with Silt (SP-SM)

A layer of poorly-graded sand with silt was observed in Boreholes 07-7, 05-1, 05-2 and 05-3. The thickness of this layer ranged from 0.8 m to 4.5 m. Where encountered, this material was observed within the upper 13 m below ground surface (above elevation 381.5 m). SPT 'N' values ranged from 5 to 95 and averaged 34, indicating that the deposit varies from a loose to very dense state but is on average, dense. The results of three grain size analyses indicate that the deposit contained 0 % gravel and between 91 % and 94 % sand and 6 to 9% fines. The gradation results are provided on Figure 2 in Appendix B. This material ranges is classified as an SP-SM soil using the MTO Soil Classification System.

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#### 4.1.3 Silty Sand (SM)

A layer of silty sand was observed beneath the vegetation, fill or poorly-graded sand with silt deposits in all boreholes. In some cases, the silty sand deposit was interrupted by layers of silt (ML) or poorly-graded sand with silt (SP-SM). Where fully penetrated, the silty sand deposit ranged from 4.0 m thick to 31.0 m thick. The base of the unit varied from elevation 350.5 m to 380.1 m (geodetic). SPT 'N' values ranged from 1 to 107 and averaged 19, suggesting a generally compact state. The moisture content of the 26 samples tested ranged from 14% to 34% with an average of 23%. Grain size analysis of 18 samples indicated that this deposit contained 0% to 6% gravel, 47% to 87% sand and 13% to 50% silt and clay sized particles. The results of the grain size distribution testing are shown on Figure 3 in Appendix B. This material corresponds to SM soil using the MTO Soil Classification System.

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#### 4.1.4 Silt / Silt with Sand / Sandy Silt (ML)

A layer of silt, silt with sand or sandy silt was encountered within five of the ten boreholes at this site.

The thickness of the silty layers ranged from 3 m to 27.6 m. Where fully penetrated the base of the unit varied from elevation 349.8 m to 363.0 m (geodetic). SPT 'N' values ranged from 3 to 124 and averaged 28, suggesting a generally compact state. The moisture content of the 17 samples tested ranged from 16% to 22% with an average of 21%. Grain size analysis of the six samples tested indicated that they contained 0% gravel, 10% to 41% sand and 59% to 90% silt and clay sized particles. The results of the grain size distribution testing are shown on Figure 4 in Appendix B. These materials correspond to an ML soil using the MTO Soil Classification System.

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#### 4.1.5 Silty Sand with Gravel, Cobbles and Boulders (TILL)

A thin layer of bouldery glacial till was encountered just above the bedrock in Boreholes 07-7 and 07-9. The upper surface of the till deposit ranged from 41.3 m to 42.1 m below ground surface (elev. 350.5 m to 351.7 m). The thickness of the till ranged from 2.3 m to 3.8 m

Rock coring techniques were used to advance the holes through boulders within the till. The limited sample recovered from the only split spoon sample successfully driven within the till deposit consisted of silty sand with gravel. Based on the above, it is inferred that the till deposit likely consists of silty sand with gravel and frequent cobbles and boulders.

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#### 4.1.6 Bedrock

Bedrock was encountered in Borehole 07-7 and 07-9 at depths of 45.9 m and 43.6 m below ground surface respectively. These depths correspond to elevations 346.7 m and 349.4 m geodetic, respectively. The bedrock was penetrated 1.8 m and 2.6 m by coring with NQ-size coring equipment. The core recovery ranged from 88 to 100 %. The rock quality designation (RQD) ranged from 50 % to 95 %, indicating fair to excellent rock mass quality. The recovered rock core consisted of black, white and pink biotite gneiss. The rock generally had a fair to excellent rock mass quality and was moderately to slightly weathered with close to moderately spaced fractures with dip angles ranging from 30 to 45 degrees from horizontal. The unconfined compressive strength of four samples of the recovered rock core ranged from 94 MPa to 145 MPa, indicating strong to very strong rock.

A detailed description of the rock cores is provided in the Rock Core Summary Table in Appendix B.

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#### 4.2 Groundwater

Groundwater levels were measured in the standpipes installed during the preliminary investigation on February 10, 2005. The water levels at that time ranged from 1.2 m to 6.0 m below ground surface (elevation 392.1 m to 392.8 m). Groundwater levels were observed in the open boreholes at the time of drilling during the 2007 investigation. The water levels were approximately 0.6 m to 0.8 m below ground surface (elevation 392.0 to 392.2 m).

The water level in Kearney Creek on January 20, 2005, was surveyed to be at elevation 392.4 m. The groundwater levels measured in the boreholes are very close to the water level in the creek, as would be expected considering the permeable nature of the upper sandy deposits.

Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

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## 5.0 CLOSURE


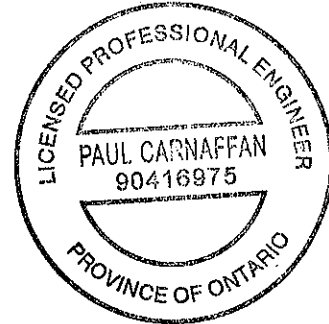
A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Yours very truly,

**JACQUES WHITFORD LIMITED**



Paul Carnaffan, M.Eng., P.Eng.



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



# FOUNDATION DESIGN REPORT

for

W.P. 545-93-00  
Highway 60 – Kearney Creek Bridge  
County of Nipissing  
District 43, Bancroft

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## 6.0 DISCUSSION

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### 6.1 Proposed Development

It is noted that, for project orientation purposes, Highway 60 will be assumed to run east-west at the Kearney Creek Bridge site, with chainage increasing from west to east.

It is understood that the Ministry of Transportation of Ontario (MTO) plans to replace the existing Kearney Creek Bridge (Site No. 43-145). Based on the Structural Planning Report, the existing bridge was constructed in 1939 and consists of an 18.6 m long three span slab-on-girder structure. It has a concrete deck and steel girders supported on timber piles. A concrete deck overlay was placed in 1981 as part of a bridge rehabilitation. The bridge provides a roadway width of 9.2 m between concrete curbs and a 0.45 m concrete curb on each side. The wingwalls at the abutments are approximately 1 m long.

The Structural Design Report for this site indicates that the Kearney Creek Structure will be reconstructed on a new alignment approximately 16 m to the north in order to improve highway geometry and facilitate single stage construction. The proposed replacement structure will likely consist of a 25 m single span CPCI 1400 girder bridge with integral abutments and wing walls. No retaining walls adjacent to the abutments are proposed. It is noted that this design will require raising the grade by approximately 1 m in the area of the structure in order to maintain a 1 m soffit clearance. The underside of the footings/pile caps for the bridge will be at approximately elevation 392.2 m (at or just above the adjacent creek level) in order to minimize dewatering requirements. Supplemental frost protection will be provided using rigid polystyrene insulation.

The approach embankments will be up to 3.75 m above existing grades along the proposed realignment.

## 6.2 Soil Summary

The native soil conditions at this site consist of a deep deposit of non-cohesive materials ranging from poorly-graded sand to silt. Although the SPT N-values suggest very loose to dense conditions, it is likely that the lower N-values observed are a reflection of the groundwater conditions. For design purposes, the soils will be considered to be compact to dense, with a design N-value of 15 blows/300 mm. To simplify the analyses and in recognition of the variable nature of the native non-cohesive soils at this site (silts and sands), the deposit has been considered to have a unit weight of 19.0 kN/m<sup>3</sup> and a minimum angle of internal friction of 29 degrees.

## 6.3 Foundation Options

### 6.3.1 Bridge Structure on New Alignment

The following table compares the available foundation options considered for the bridge structure on the new alignment:

**Table 6.1: Foundation Comparison for Replacement Structure**

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Spread Footings	<ul style="list-style-type: none"><li>• moderate geotechnical resistance</li><li>• allows for semi-integral abutment design</li></ul>	<ul style="list-style-type: none"><li>• incompatible with integral abutment design</li><li>• native soils easily disturbed when saturated</li><li>• increased susceptibility to scour</li></ul>	Low	<ul style="list-style-type: none"><li>▪ scour, erosion of foundation cover / loss of geotechnical resistance</li><li>▪ excavation below waterline / dewatering required or work in the wet</li></ul>
Spread Footings on Structural Fill Pad	<ul style="list-style-type: none"><li>• moderate geotechnical resistance but higher than spread footings on native soil</li><li>• allows for semi-integral abutment design</li></ul>	<ul style="list-style-type: none"><li>• incompatible with integral abutment design</li><li>• native soils easily disturbed when saturated</li><li>• may require excavation below water level</li></ul>	Low	<ul style="list-style-type: none"><li>▪ excavation below waterline / dewater or do work in the wet</li><li>▪ scour, erosion of foundation cover / loss of geotechnical resistance</li></ul>
Driven H-piles on bedrock	<ul style="list-style-type: none"><li>• readily incorporated into integral abutment design</li><li>• high geotechnical resistance on bedrock</li></ul>	<ul style="list-style-type: none"><li>• anticipated length of 40 to 45 m</li></ul>	Moderate	<ul style="list-style-type: none"><li>▪ piles do not penetrate boulders in till / damage to piles and/or slight reduction in resistance</li></ul>

**Table 6.1: Foundation Comparison for Replacement Structure**

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Driven H-piles in silt and sand (friction)	<ul style="list-style-type: none"> <li>• moderate geotechnical resistance (lower than piles on rock)</li> </ul>	<ul style="list-style-type: none"> <li>• anticipated length of 30 m</li> <li>• more difficult to incorporate into integral abutment design</li> </ul>	Moderate	<ul style="list-style-type: none"> <li>▪ design resistance not achieved at specified tip elevation / lengthen piles at extra cost</li> </ul>
Caissons	<ul style="list-style-type: none"> <li>• high geotechnical resistance on bedrock</li> <li>• allows for semi-integral abutment design</li> </ul>	<ul style="list-style-type: none"> <li>• require tremie concrete</li> <li>• require cased holes</li> <li>• incompatible with integral abutment design</li> </ul>	High	<ul style="list-style-type: none"> <li>▪ base instability in saturated sands may require use of drilling mud / extra cost</li> </ul>

Given the potential concerns with groundwater control at this site and the desire to incorporate an integral abutment, it is recommended that the replacement bridge be founded on H-piles end-bearing on bedrock.

## 7.0 RECOMMENDATIONS

### 7.1 Structure Foundations

#### 7.1.1 Pile Foundations

##### Axial Resistance

The bridge structure may be supported on steel H-piles driven to bedrock. The estimated pile tip elevation is between 346 m and 349 m for the west and east abutments, respectively. Due to the proposed embedment length (>40 m), and hard end-bearing conditions, it is recommended that a heavier pile section (HP 310x132) be used rather than the more commonly specified HP 310x110.

The following geotechnical parameters are recommended for the design of single piles:

**Table 7.1: Recommended Pile Design Parameters for HP 310 x 132 Piles**

Founding Material	Estimated Pile Tip Elevation (m)	Factored Axial Geotechnical Resistance at ULS (kN)	Unfactored Resistance at SLS (kN)
Bedrock	346.0 to 349.0	2,000	N/A

Previous experience in the Algonquin Highlands has consistently revealed high strength rock where the Geotechnical Resistance of the rock would exceed the Structural Resistance of the pile. The above factored axial resistance at ULS for piles on bedrock corresponds to the factored Structural Resistance of the pile

The pile tip for piles set on bedrock is not anticipated to settle. The unfactored geotechnical resistance at SLS (for 25 mm of settlement) will be higher than the factored geotechnical resistance at ULS. Therefore the geotechnical SLS value will not govern the design, however, the structural engineer will need to evaluate the elastic compression of the pile.

Downdrag forces are not anticipated at this site.

### Lateral Resistance

The passive lateral resistance for vertical piles should be calculated as per the non-cohesive approach of Section C6.8.7.1 (a) Static Analysis and C6.8.7.2 Static Analysis of the CHBDC using the following unfactored geotechnical soil parameters:

**Table 7.2: Recommended Lateral Pile Design Parameters**

Parameter	OPSS Granular B Type I	Native Silts and Sands
Bulk Unit Weight, kN/m <sup>3</sup>	21.2	19.0
Effective Friction Angle, degrees	35	29
Coefficient of Passive Earth Pressure	3.7	2.9

### Lateral Deflections

The coefficient of horizontal subgrade reaction, which may be used for deflection calculations, may be estimated for cohesionless soils using Terzaghi's method (1955) as follows:

$$k_s = n_h z/d$$

where

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

$n_h$  = coefficient related to soil compactness

$z$  = depth

$d$  = pile diameter

The soil compactness, based on the SPT N-values, is highly variable at this site but is generally compact within the upper soils (above elevation 382 m). Therefore, an  $n_h$  value of 4,000 kN/m<sup>3</sup> is recommended for design calculations for the upper soils. Below elevation 382 m the soil is compact to dense and an  $n_h$  value of 11,000 kN/m<sup>3</sup> is recommended.

## Group Effects on Lateral Deflections

As per section 6.8.9.2 of the CHBDC, the effects of interaction of the piles must be considered where the centre-to-centre spacing of the piles is less than 2.5 d (where d=pile width/diameter) or 750 mm. The interaction generally results in the lateral load at a specific deflection being 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

$\alpha$  = 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.



## 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. For this site, the soils primarily consist of a silt and sand mix and therefore the following parameters may be used for preliminary design purposes:

Submerged Unit Weight	9.2 kN/m <sup>3</sup>
Effective Friction Angle	29°
$\beta$ Coefficient	0.4
Resistance Factor	0.3

The following values have been calculated based on the above recommended parameters:

**Table 7.3: Recommended Tensile Pile Design Parameters**

Pile Type	Pile Tip Elevation (m)	Factored Geotechnical Resistance (Tension) at ULS (kN)
HP 310 x 132	349.0	1,300

The factored geotechnical resistance (tension) at ULS provided above does not include the weight of the pile.

### Pile Notes

Pile tips should be fitted with rock points as per OPSD-3000.201 and driven into bedrock in accordance with SP903S01.

Piles materials, splicing, installation and monitoring should be in accordance with SP 903S01 using an Ultimate Geotechnical Resistance of twice the maximum factored design load at ULS per pile.

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## 7.2 Earth Pressure Design

The abutments and retaining walls should be backfilled with free-draining material such as OPSS Granular B Type II or OPSS Granular A to prevent hydrostatic pressure build-up.

Computation of earth pressures should be in accordance with Section 6.9 of the CHBDC. For abutments or retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. For a structure with a horizontal backfill, the following unfactored soil parameters may be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

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

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

Where H is the height of the wall. Values for  $K_a$ ,  $K_p$  and  $\gamma$  are provided below. The thrust acts at a point one third up the height of the wall.

**Table 7.4: Recommended Lateral Earth Pressure Parameters**

Parameter	OPSS Granular B, Type I and II	OPSS Granular A and Granular B Type II
Total Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21.2	22.0
Effective Friction Angle	32 degrees	35 degrees
Coefficient of Active Earth Pressure ( $K_a$ )	0.31	0.27
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.47	0.43
Coefficient of Passive Earth Pressure ( $K_p$ )	3.2	3.7

Compaction of the granular backfill near the walls should be carried out using hand-operated equipment to prevent over-stressing the abutment walls.

Drainage should be provided behind vertical walls to prevent hydrostatic pressure build-up. Drainage should be provided by installing a subdrain as per OPSD 3102.100 with positive drainage to a frost-free outlet. Granular backfill should be designed as per OPSD 3101.150 using a depth of frost penetration,  $f$ , of 1.9 m.

## 7.3 Seismic Design Considerations

### 7.3.1 Zonal Acceleration Ratio

Table A3.1.1 of the CHBDC indicates that the Zonal Acceleration Ratio for Bancroft, which is 100 km southeast of the site, is 0.10. Reference is made to Section C4.6.4 of the CHBDC for the calculation of seismic forces on abutments and retaining walls. A seismic hazard calculation for the Kearney Creek site was obtained from Natural Resources Canada (copy provided in Appendix C). It indicates that the peak ground acceleration (PGA) value corresponding to a 10% probability of exceedance in 50 years is 0.078.

### 7.3.2 Soil Profile Type

It is recommended that Soil Profile 1 as defined in CHBDC Section 4.4.6 be used in the seismic design of this site.

---

### 7.3.3 Liquefaction of Foundation Soils

An assessment of the potential for liquefaction of the foundation soils was carried out using the Seed and Idriss (1971) simplified procedure outlined in the CHBDC, Section C4.6.2 Liquefaction of Foundation Soils.

The cyclic stress ratios (CSR) generated by the design earthquake were calculated based on the PGA value of 0.078 obtained from Natural Resources Canada. The profile of cyclic resistance ratios (CRR) available from the soil was calculated based on the CPTu tip resistance, actual fines content based on gradation results and a design earthquake magnitude of 6.0.

The results of the analysis indicate that liquefaction is not a concern at this site since the cyclic resistance ratios available from the soil are greater than the cyclic stresses that would be generated by the design earthquake, typically by a factor of at least three. The results are shown graphically as a profile of factor of safety against liquefaction versus elevation on the plots provided in Appendix C.

---

### 7.3.4 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

$\gamma$  = 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,  $A$  0.1

- Horizontal Acceleration Coefficient,  $k_h$  0.05
- Vertical Acceleration Coefficient,  $k_v$  0.033
- 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  $0^\circ$  to provide a conservative estimate.

**Table 7.5: Combined Coefficients of Static and Seismic Earth Pressure**

Parameter	OPSS Granular B Type I and II		OPSS Granular A & Granular B Type II	
	Horizontal Backslope	2H:1V Backslope	Horizontal Backslope	2H:1V Backslope
Total Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21.2	21.2	22.0	22.0
Effective Friction Angle	32 degrees	32 degrees	35 degrees	35 degrees
Active Earth Pressure ( $K_{AE}$ )	0.34	0.57	0.30	0.46
Height of application of $P_{AE}$ from base as ratio of wall height (H)	0.349	0.372	0.350	0.366
Passive Earth Pressure ( $K_{PE}$ )	3.16	-	3.59	-
Height of application of $P_{PE}$ from base as ratio of wall height (H)	0.316	-	0.316	-

It is noted that the combined coefficients of static and seismic earth pressure presented in Table 7.5 deviate only slightly from the static coefficients presented in Table 7.4. This is due to the low zonal acceleration ratio at this site.

#### 7.4 Embankment Design

The existing embankments are constructed at 2H:1V and exhibit no signs of instability.

---

#### 7.4.1 New Alignment

Embankment side slopes on the new alignment should be constructed no steeper than 2H:1V. Embankment fill should consist of OPSS Select Subgrade Material or clean granular fill such as OPSS Granular B. The use of rock fill could also be considered. Rockfill should be sloped to be no steeper than 1.25H:1V generally and 1.5H:1V where the fill extends below water level.

Settlement of the underlying soil has been estimated using elastic theory. Stress distribution was assessed based on a Boussinesq distribution. As much as 3.5 m of fill will be required at some locations to achieve design grades at the approaches on the new alignment. This will induce as much as 15 mm of settlement in the underlying native materials. Due to the non-cohesive nature of these materials, it is anticipated that settlement will occur rapidly. Post construction settlements of the underlying soils will be less than 5 mm. Self settlement of the embankment fill of as much as 10 mm for 3.5 m of fill will occur. This settlement will be complete at the completion of construction.

---

#### 7.4.2 Existing Alignment

The construction of the roadway embankment along the proposed new alignment will result in additional settlement of the existing embankment. It is estimated that settlement at the existing edge of shoulder will be less than 5 mm. Settlement of the existing embankment may result in similar settlement of the existing timber piles, depending on the length of the piles.

---

#### 7.5 Dewatering

It is anticipated that the underside of the pile cap will be set at elevation 392.2 m in order to minimize requirements for excavation below the water level.

The water level in Kearney Creek at the time of the investigation was 392.2 m. The Draft Structural Planning Report identifies the water level as elevation 392.1 m  $\pm$  and the high water level (100 year storm) as elevation 393.20 m.

Based on the proposed founding elevations and observed water levels, the deepest planned excavations will extend to at or just above the water level. Under these conditions, no extensive dewatering would be required, just localized periodic dewatering to remove surface water infiltration from rainfall. This type of dewatering could be carried out using conventional sump pumps.

It will likely be necessary to construct a working pad at the pile cap level since the soil at the base of the excavation will be wet and easily disturbed by construction activities. The working pad should consist of a minimum of 300 mm of OPSS Granular A.

Since the proposed underside of pile cap will be at or just above the observed creek levels but 1 m below the 100-year flood level, it is possible the abutment excavations would be within the Creek under high water levels. This would require stopping work and waiting for water levels to recede or provision of a cofferdam and dewatering system. Such a dewatering system would generally involve sheet piles embedded to a depth below the planned excavation depth with relief wells located inside the excavation. Design of shoring will need to account for basal heave due to flow of water around (i.e. beneath) the sheet piling.

It is recommended that the contract include a Non Standard Special Provision for dewatering to alert the contractor to the permeable soil conditions, water levels and potential need for dewatering.

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## 7.6 Erosion Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The creek slopes within 3 m of the structures should be surfaced with rip-rap at least 300 mm thick placed on a Class II non-woven filter fabric. At other locations, normal slope vegetation should be established as soon as possible after completion of the embankment fills in order to control surficial erosion.

The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site.

---

## 7.7 Frost Protection

Pile caps, retaining walls and spread footings should be provided with the equivalent of 1.9 m of earth cover for frost protection.

Alternatively and provided a minimum of 0.9 m of soil cover is available, rigid polystyrene insulation having a minimum thickness of 75 mm and extending beneath the pile cap and a minimum of 1.2 m laterally out from the edges of the pile cap would provide equivalent frost protection. A site specific detail will need to be developed once the structure and grading geometry have been confirmed. The detail will need to ensure that the insulation does not become displaced due to buoyant forces under flood conditions.

---

## 7.8 Other Construction Considerations

### Site Grading and Preparation

All organic soils and other deleterious materials must be removed from beneath the proposed foundation units. Where deleterious materials are encountered, the material should be excavated, wasted and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the foundations.

The influence zone includes all materials below an imaginary line drawn at an angle of 1 horizontal to 1 vertical downward and away from the vertical edges of the abutments.

Surficial vegetation, rootmat and topsoil should be removed beneath the approach embankments. Stripping of deleterious materials should be inspected by geotechnical personnel to ensure that all unsuitable materials are removed prior to placement of embankment fill.

Where required for grading purposes, fill should consist of Select Subgrade Material (SSM), placed in lifts and compacted in accordance with SP105S10.

Site preparation should be carried out in accordance with the requirements of *SP 902S01 Earth Excavation for Structure*.

### Excavation

Earth excavation should be carried out in accordance with OPSS-206.07.03. Side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations. The soils to be excavated for the proposed foundations should be considered as a Type 3 soil. Above the creek and ground water level, temporary cut slopes should be no steeper than 1 horizontal to 1 vertical from the base of the excavation. For excavations below creek and groundwater levels, shoring will be required.

Encroachment of excavations into the forward and side slopes of the existing structure will require special attention. Excavations will not be permitted within the influence zone of the existing abutments. The influence zone includes all materials below an imaginary line drawn at an angle of 1 horizontal to 1 vertical downward and away from the vertical edges of the abutments.

Shoring design should meet the requirements of Performance Level 2 as per SP105S19 and should consider sloping backfill and traffic loading. Protection systems would likely consist of a cantilevered steel sheet pile system or steel H-piles with timber lagging.

## Cement Type and Corrosion Protection

Six soil samples were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, resistivity, chloride and water soluble sulphate, in order to determine cement type and reinforcing steel protection requirements. The results are presented in the table below.

**Table 7.6: Chemical Analysis Results**

Location	Borehole	Sample	pH	Resistivity	Soluble Sulphate	Chloride
New Alignment West Abutment	05-2	SS-7	7.59	6,200 ohm.cm	5 µg/g	110 µg/g
New Alignment West Abutment	05-2	SS-14	7.06	37,000 ohm.cm	10 µg/g	5 µg/g
Existing Alignment East Abutment	05-7	SS-4	7.70	4,700 ohm.cm	55 µg/g	170 µg/g
Existing Alignment East Abutment	05-7	SS-11	7.30	18,000 ohm.cm	5 µg/g	30 µg/g
New Alignment West Abutment	07-7	SS-6	8.11	18,500 ohm.cm	35 µg/g	<5 µg/g
New Alignment West Abutment	07-7	SS-16	6.98	25,300 ohm.cm	30 µg/g	<5 µg/g

The soluble sulphate results indicate that a Type GU (General Use) Portland cement would be suitable for use in concrete mixtures at this site. The chloride, pH, and resistivity results should be considered by the structural designer when designing corrosion protection system.



## 8.0 CLOSURE

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Yours very truly,

**JACQUES WHITFORD LIMITED**



Paul Carnaffan, M.Eng., P.Eng.



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



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# **APPENDIX A**

## **Borehole Location Plan and Profile Plot**





CONT No -  
WP No 545-93-00

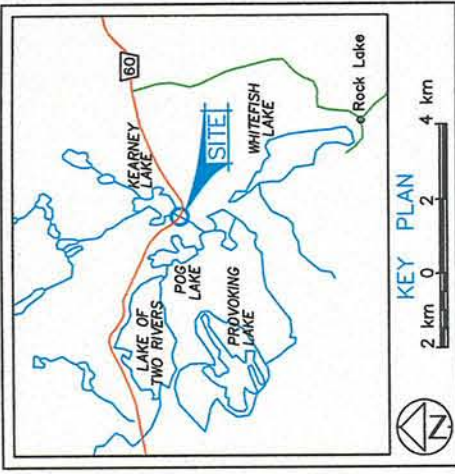
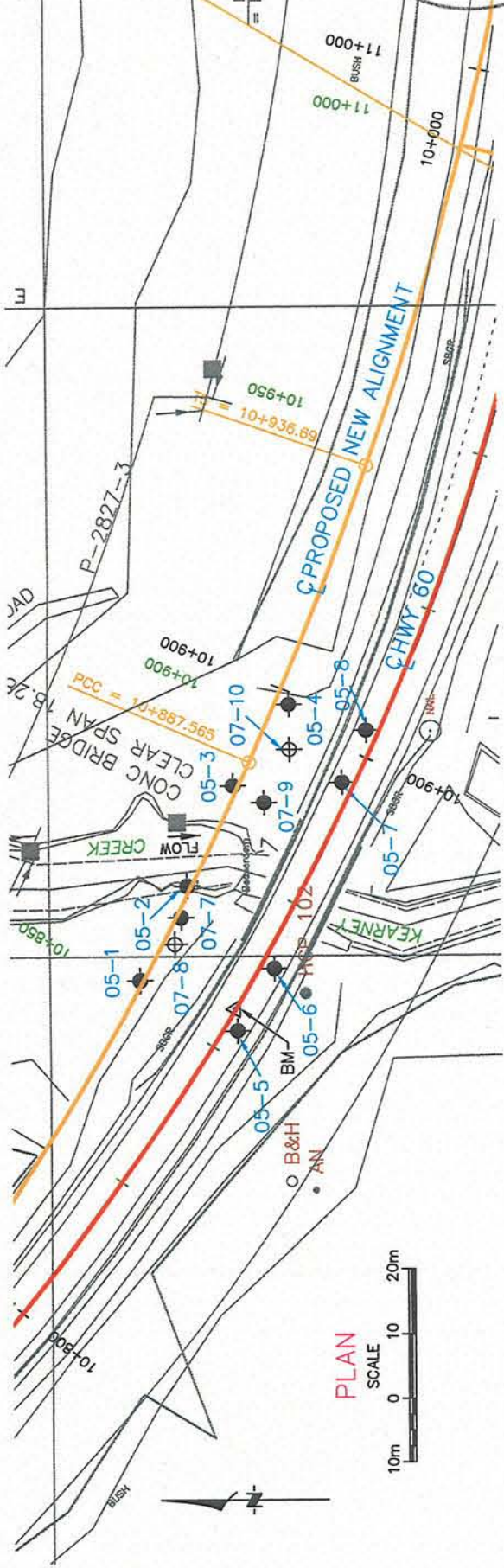
HWY 60 OVER KEARNEY CREEK  
BRIDGE REPLACEMENT

BOREHOLE LOCATIONS & SOIL STRATA

SHEET -

PROFESSIONAL ENGINEER  
PAUL CARMAFFAN  
90416975  
01/29/07  
PROVINCE OF ONTARIO

PROFESSIONAL ENGINEER  
F.J. GRIFFITHS  
07/10/09  
PROVINCE OF ONTARIO



LEGEND		
	Borehole	
	Dynamic Cone Penetration Test (Cone)	
	Borehole & Cone	
	N	
	Blows/0.3m (Std Pen Test, 475 J/blow)	
	Cone Blows/0.3m (60° Cone, 475 J/blow)	
	WL at time of investigation	
	WL in Piezometer	
	Piezometer	
	Benchmark (Top of Pavement)	
	BM	
Reference: TSH profile plate A-2		
No	ELEVATION	NORTHING
05-1	392.6	5 048 286.3
05-2	392.4	5 048 279.0
05-3	392.5	5 048 271.8
05-4	394.1	5 048 263.2
05-5	395.5	5 048 271.5
05-6	395.3	5 048 265.8
05-7	395.4	5 048 255.2
05-8	395.5	5 048 251.5
07-7	392.6	5 048 280.0
07-8	392.6	5 048 281.0
07-9	393.0	5 048 267.1
07-10	393.0	5 048 263.2
		387 496.4
		387 510.6
		387 526.0
		387 538.7
		387 488.7
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		387 526.5
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		387 505.8
		387 501.8
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		384 531.7

**NOTE**  
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

**NOTE**  
The complete foundation investigation and design report for this project is available for review at the Engineering Materials Office, Downsview, Ontario. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

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## **APPENDIX B**

Symbols and Terms Used on Borehole Records

Borehole Records

Terminology Used on SCPTu Records

SCPTu Records

Grain Size Distribution Test Results

Rock Core Summary Table

# 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 vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

### Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

### Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

### Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

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:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

### Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

### Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
<i>Very Soft</i>	<0.25	<12.5
<i>Soft</i>	0.25 - 0.5	12.5 - 25
<i>Firm</i>	0.5 - 1.0	25 - 50
<i>Stiff</i>	1.0 - 2.0	50 - 100
<i>Very Stiff</i>	2.0 - 4.0	100 - 200
<i>Hard</i>	>4.0	>200



## ROCK DESCRIPTION

### Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

### Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

### Terminology describing rock strength:

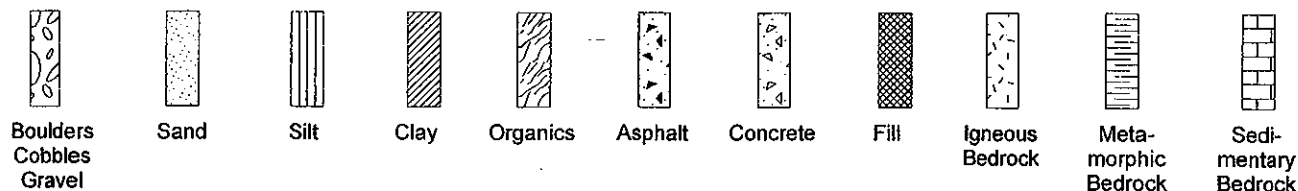
Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

### Terminology describing rock weathering:

Term	Description
<i>Fresh</i>	No visible signs of rock weathering. Slight discolouration along major discontinuities
<i>Slightly Weathered</i>	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
<i>Moderately Weathered</i>	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly Weathered</i>	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely Weathered</i>	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



## SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

## WATER LEVEL MEASUREMENT



## RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

## N-VALUE / RQD

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log. RQD is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery.

## DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability. Soil type may be inferred from adjacent boreholes and test pits.

## OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
$\gamma$	Unit weight
$G_s$	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
$Q_u$	Unconfined compression
$I_p$	Point Load Index ( $I_p$ on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer



RECORD OF BOREHOLE No 05-1

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge, N5046286.3 E387496.4 ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 05.01.27 - 05.01.27 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
392.6																	
0.0	Silty SAND, trace organics, very loose, brown (SM)		1	SS	3		392										
391.1			2	SS	7												
1.5	Silty SAND, loose to compact, brown (SM)		3	SS	12		390										80 (20)
			4	SS	7												
			5	SS	28		388										
	- becomes grey		6	SS	11		386										
385.2			7	SS	6		384										
7.4	Poorly-graded SAND with silt, loose, grey (SP-SM)		8	SS	11		382										
383.6			9	SS	7												
9.0	Silty SAND, loose to compact, grey (SM)		10	SS	21		380										78 (22)
379.8																	
12.8	End of Borehole																

×<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



# RECORD OF BOREHOLE No 05-2

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048279.0 E387510.6 ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 05.01.24 - 05.01.25 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								WATER CONTENT (%)										
392.4	Grass						20	40	60	80	100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	GR	SA	SI	CU
0.0	Organic Matter																	
391.6																		
0.8	Poorly-graded SAND with silt, loose to very dense, grey to brown (SP-SM)		1	SS	5													
			2	SS	17													
			3	SS	68													
			4	SS	95													
			5	SS	49													
			6	SS	45													
387.1																		
5.3	Silty SAND, compact to dense, grey (SM)		7	SS	10													
	- gravelly sand seam		8	SS	32													
			9	SS	11													
			10	SS	12													
380.6																		
11.8	Poorly-graded SAND with silt, very dense, grey (SP-SM)		11	SS	64													
379.1																		
13.3	Sandy SILT to SILT with sand, loose to dense, grey (ML)		12	SS	22													
			13	SS	8													
			14	SS	23													
			15	SS	30													
			16	SS	25													
			17	SS	27													
			18	SS	41													
			19	SS	44													

Continued Next Page

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

## METRIC

CHECKED BY PC

✕<sup>3</sup>, ✕<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

RECORD OF BOREHOLE No 05-3

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048271.8 E387526.0 ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 05.01.28 - 05.02.01 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W <sub>p</sub>	W	W <sub>L</sub>		
392.5							20	40	60	80	100					
0.0	Silty SAND, some organic material, loose, dark brown (SM)		1	SS	4											
391.0			2	SS	3											
1.5	Poorly-graded SAND with silt, trace organics, very loose, brown (SP-SM)		3	SS	32											
390.2			4	SS	11											
2.3	Silty SAND, compact to dense, brown (SM)		5	SS	42											
			6	SS	35											
386.4																
6.1	Silty SAND, loose to dense, grey (SM)		7	SS	23											
			8	SS	7											
			9	SS	27											
			10	SS	36											
			11	SS	39											
			12	SS	14											
377.3																
15.2	Sandy SILT to SILT, compact to dense, grey (ML)		13	SS	30											
	- sand rose up augers		14	SS	40											
			15	SS	23											
			16	SS	23											
			17	SS	3											
			18	SS	13											
			19	SS	25											

Continued Next Page

×<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No 05-3

2 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048271.8 E387526.0 ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 05.01.28 - 05.02.01 CHECKED BY PC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
						20	40	60	80	100			
	Sandy SILT to SILT, compact to dense, grey (ML) (continued)		20	SS	12								
			21	SS	24								
			22	SS	15								
			23	SS	24								
			24	SS	36								
			25	SS	28								
			26	SS	39								
			27	SS	31								
			28	SS	82								
349.8	End of Borehole		29	SS	100/75 mm								

ONTARIO MTO UPDATE 11685.GPJ ONTARIO MOT.GDT 07/10/25

RECORD OF BOREHOLE No 05-4										1 OF 1	METRIC						
W.P. 545-93-00		LOCATION Highway 60, Kearney Creek Bridge, N5048263.2 E 387536.7				ORIGINATED BY AB											
DIST Bancroft HWY 60		BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons				COMPILED BY JF											
DATUM Geodetic		DATE 05.01.28 - 05.01.28				CHECKED BY PC											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	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									WATER CONTENT (%)
394.1 0.0	Sand, with gravel, occasional cobbles, loose, brown (FILL)					V											
392.6 1.5	Sand, some gravel, trace silt, compact, brown (FILL)		1	SS	3												
391.8 2.3	Silty sand, occasional woody organic, compact to dense, brown (FILL)		2	SS	9												
			3	SS	30												
390.5 3.6	Silty SAND, compact, brown (SM)	4	SS	22													
		5	SS	15													
		6	SS	13													
387.4 6.7	Sandy SILT, occasional sand seams, loose to compact, grey (ML)		7	SS	6												
			8	SS	19												
			9	SS	11												
			10	SS	16												
			11	SS	20												
381.3 12.8	End of Borehole  Standpipe installed  (25 mm diameter flexible poly-tube)																

ONTARIO MTO UPDATE 11685.GPJ ONTARIO MOT.GDT 07/10/25

# RECORD OF BOREHOLE No 05-5

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 80, Kearney Creek Bridge, N5048271.5 E387488.7 ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers with SplitSpoons COMPILED BY JF  
 DATUM Geodetic DATE 05.01.22 - 05.01.22 CHECKED BY PC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	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	WATER CONTENT (%)					
						○ UNCONFINED    ✕ FIELD VANE ● QUICK TRIAXIAL    ✕ LAB VANE										
395.5	Asphalt															
395.0	120 mm Asphalt		1	GS												
394.4	Sand and gravel, some silt, very dense, brown (FILL)		2	SS	50/180mm											
1.1	Silty sand, very loose to compact, brown (FILL)		3	SS	7											
			4	SS	3											
391.8			5	SS	2											
3.7	Silty SAND, loose, brown (SM)		6	SS	4											
389.4			7	SS	6											
6.1	End of Borehole															

✕<sup>3</sup>, ✕<sup>3</sup>: Numbers refer to Sensitivity    ○ 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No 05-6

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048265.8 E387498.1 ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 05.02.02 - 05.02.02 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)
								○ UNCONFINED	● QUICK TRIAXIAL	✕ FIELD VANE	✕ LAB VANE						
395.3	Asphalt						20	40	60	80	100						
390.9	75 mm ASPHALT Gravelly sand, trace to some silt, brown (FILL)  - frozen to 1.5 m   - occasional wood		1	SS	100/ 125 mm												
			2	SS	16												
			3	SS	13												
			4	SS	18												
			5	SS	55												
390.9	Silty SAND, compact to dense, brown (SM)		6	SS	40												
4.4			7	SS	29												
			8	SS	32												
			9	SS	25												
			10	SS	24												
			11	SS	45												
			12	SS	15												
			13	SS	26												
			14	SS	107												
386.2	Silty SAND, compact to dense, grey (SM)																
9.1																	

ONTARIO MTO UPDATE 14685.GPJ ONTARIO MOT.GDT 07/10/25

# RECORD OF BOREHOLE No 05-7

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048255.2 387526.5 ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 05.01.21 - 05.01.21 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL						
395.4	Asphalt						20	40	60	80	100					
394.9	110 mm Asphalt															
393.9	Sand and gravel, some silt, brown (FILL)		1	SS	50/80mm											
393.9	- frozen to 1.4 m															
1.5	Silty sand, very loose to compact, brown ( FILL)		2	SS	9											
			3	SS	5											
			4	SS	2											
391.7	Silty SAND, occasional woody organic matter, loose to compact, brown (SM)		5	SS	4											
			6	SS	12											
389.4	Silty SAND, very loose to compact, grey (SM)		7	SS	4											
			8	SS	1											
			9	SS	5											
			10	SS	15											
384.1	Sandy SILT, compact to very dense, grey (ML)		11	SS	25											
			12	SS	20											
			13	SS	18											
			14	SS	111											
			15	SS	21											
373.4	Silty SAND, very dense, grey (SM)		16	SS	68											
22.0																

Continued Next Page

\*<sup>3</sup>, x<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No 05-7

2 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048255.2 387526.5 ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 05.01.21 - 05.01.21 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	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				W <sub>p</sub>	W	W <sub>L</sub>		
369.4	Silty SAND, very dense, grey (SM) (continued)						370									
26.0	Sandy SILT, dense, grey (ML)		17	SS	48		368									
							366									
			18	SS	36		364									
363.0	Silty SAND, very dense, grey (SM)						362									
32.4			19	SS	102											
361.3	End of Borehole															
34.1																

# RECORD OF BOREHOLE No 05-8

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048251.5 E387534.6 ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 05.01.22 - 05.01.22 CHECKED BY PC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
395.5	Asphalt												
395.0	110 mm Asphalt		1	GS									
394.2	Gravelly sand, some silt, very dense, brown (FILL)		2	SS									
394.2	- frozen to 1.3 m												
1.3	Sand, trace silt, trace gravel, very loose to compact, brown (FILL)		3	SS									
			4	SS									
392.4	Silty sand, loose, brown to dark brown (FILL)		5	SS									
3.1			6	SS									
391.2	Organic layer		7	SS									
391.2	Silty SAND, loose, brown (SM)		8	SS									
4.4													
389.4	End of Borehole												
6.1													

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

# RECORD OF BOREHOLE No 07-7

1 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 80, Kearney Creek Bridge, N5048280.0 E387505.8 ORIGINATED BY JF  
 DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
 DATUM Geodetic DATE 07.05.04 - 07.05.10 CHECKED BY PC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100					
392.8	Grass															
392.9	150 mm TOPSOIL															
0.2	Silty SAND, very loose to compact, brown to orangy brown (SM)		1	SS	1											
			2	SS	2											0 69 (31)
			3	SS	14											
			4	SS	18											
			5	SS	10											
			6	SS	12											
			7	SS	7											
387.3	Silty SAND, loose to compact, brownish gray (SM)		8	SS	11											
5.3			9	SS	9											0 82 (18)
385.0	Poorly-graded SAND with silt, compact, grey (SP-SM)		10	SS	14											
7.6			11	SS	14											0 91 (9)

Continued Next Page

× 3, × 3

Numbers refer to  
Sensitivity

○ 3% STRAIN AT FAILURE

## 2 OF 5

### METRIC

W.P.	545-93-00	LOCATION	Highway 60, Kearney Creek Bridge, N5048280.0 E387505.8	ORIGINATED BY	JF
DIST	43	HWY	60	BOREHOLE TYPE	HW casing, NW casing, Split Spoons, NQ Core
DATUM	Geodetic	DATE	07.05.04 - 07.05.10	CHECKED BY	PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED		× FIELD VANE						● QUICK TRIAXIAL		× LAB VANE	
							20	40	60	80	100								

✕<sup>3</sup>, ✕<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

# RECORD OF BOREHOLE No 07-7

3 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048280.0 E387505.8 ORIGINATED BY JF  
 DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
 DATUM Geodetic DATE 07.05.04 - 07.05.10 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
	Silty SAND, loose to dense, grey (SM) (continued)																
							372										
							371										
							370										
							369										
			17	SS	23		368										
							367										
							366										
							365										
			18	SS	24		364										
							363										

Continued Next Page

x<sup>3</sup> x<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 07-7

4 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 80, Kearney Creek Bridge, N5046280.0 E387505.8 ORIGINATED BY JF  
DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
DATUM Geodetic DATE 07.05.04 - 07.05.10 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	$w_p$	$w$	$w_L$		
	Silty SAND, loose to dense, grey (SM) (continued)																
			19	SS	9											0 58 42 0	
			20	SS	16												
			21	SS	22											0 50 49 1	

ONTARIO MTO 1023332.GPJ ONTARIO MOT.GDT 07/10/25

Continued Next Page

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

# RECORD OF BOREHOLE No 07-7

5 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048280.0 E387505.8 ORIGINATED BY JF  
 DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
 DATUM Geodetic DATE 07.05.04 - 07.05.10 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
								20 40 60 80 100					
						○ UNCONFINED	✱ FIELD VANE						
						● QUICK TRIAXIAL	✱ LAB VANE						
						WATER CONTENT (%)							
						20 40 60 80 100							
						10 20 30							
350.5	Silty SAND, loose to dense, grey (SM) (continued)												
42.1	Silty sand, some gravel, frequent cobbles and boulders, dense, grey: TILL		22	SS	24								
			23	NQ									
			24	NQ									
			25	NQ									
346.7	Biotite GNEISS, black, white and pink, fair, moderate to slightly weathered, close to moderately spaced fractures, thin bedding, 30 to 45 degree dip		26	NQ									
45.9			27	NQ									
344.9	End of Borehole												
47.7													

ONTARIO MTO 1023332.GPJ ONTARIO MOT.GDT 07/10/25

RECORD OF BOREHOLE No 07-9

1 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 80, Kearney Creek Bridge, N5048267.1 E387523.4 ORIGINATED BY AB  
DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
DATUM Geodetic DATE 07.04.30 - 07.05.03 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
393.0	Grass																
389.8	150 mm TOPSOIL																
0.2	Sand, with cobbles, loose, brown: FILL																
392.5																	
0.5	Silty SAND, loose to compact, brown (SM)																
			1	SS	15		392										
			2	SS	6		391										
			3	SS	12		390										6 87 (27)
			4	SS	23		389										
			5	SS	9		388										
			6	SS	15		387										
387.8			7	SS	6		386										
5.2	Silty SAND, loose to compact, grey (SM)						385										0 87 (13)
			8	SS	6		384										
			9	SS	16												

Continued Next Page

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



RECORD OF BOREHOLE No 07-9										2 OF 5		METRIC				
W.P. 545-93-00		LOCATION Highway 60, Kearney Creek Bridge, N5048287.1 E387523.4				ORIGINATED BY AB										
DIST 43 HWY 60		BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core				COMPILED BY JF										
DATUM Geodetic		DATE 07.04.30 - 07.05.03				CHECKED BY PC										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED    × FIELD VANE ● QUICK TRIAXIAL    × LAB VANE								
	Silty SAND, loose to compact, grey (SM) (continued)		10	SS	4		382									0 70 (30)
			11	SS	5		381									
			12	SS	7		380									
378.5	SILT, loose, grey (ML)		13	SS	8		379									
14.5			14	SS	6		378									
			15	SS	12		377									
375.5	Silty SAND, loose to compact, grey (SM)						376									0 12 83 5
17.5							375									
							374									

Continued Next Page

×<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MTO 1023332.GPJ ONTARIO MTO GDT 07/10/25

RECORD OF BOREHOLE No 07-9

3 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048267.1 E387523.4 ORIGINATED BY AB  
DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
DATUM Geodetic DATE 07.04.30 - 07.05.03 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
	Silty SAND, loose to compact, grey (SM) (continued)																
			16	SS	8												
			17	SS	15												
366.5 26.5	SILT with sand, loose to dense, grey (ML)																
			18	SS	15												

ONTARIO MTO 1023332.GPJ ONTARIO MTO GDT 07/10/25

Continued Next Page

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

# RECORD OF BOREHOLE No 07-9

4 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Creek Bridge, N5048267.1 E387523.4 ORIGINATED BY AB  
 DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
 DATUM Geodetic DATE 07.04.30 - 07.05.03 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	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 X LAB VANE					WATER CONTENT (%) W <sub>p</sub> W W <sub>L</sub>				
	SILT with sand, loose to dense, grey (ML) (continued)																
			19	SS	22		362										
							361										
							360										
			20	SS	5		359									0 25 74 1	
							358										
							357										
			21	SS	34		356										
							355										
							354										
			22	SS	13												

Continued Next Page

\*<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 07-9

5 OF 5

METRIC

W.P. 545-93-00 LOCATION Highway 80, Kearney Creek Bridge, N5048287.1 E387523.4 ORIGINATED BY AB  
DIST 43 HWY 60 BOREHOLE TYPE HW casing, NW casing, Split Spoons, NQ Core COMPILED BY JF  
DATUM Geodetic DATE 07.04.30 - 07.05.03 CHECKED BY PC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
351.7	SILT with sand, loose to dense, grey (ML) (continued)					352										
41.3	Silty sand, some gravel, frequent cobbles and boulders, dense, grey: TILL		23	NQ		351										
						350										
349.4																
43.6	Biotite GNEISS, black, white and pink, fair to excellent, moderate to slightly weathered, close to moderately spaced fractures, thin bedding, 30 to 40 degree dip		24	NQ		349										TCR = 88% RQD = 87%
			25	NQ												TCR = 96% RQD = 92%
						348										TCR = 100% RQD = 95%
			26	NQ		347										
346.8																
46.2	End of Borehole															

ONTARIO MTO 1023332.GPJ ONTARIO MOT.GDT 07/10/25

## Terminology Used on SCPTu Records

### Key Terminology and Principles

#### **SCPTu:**

- Seismic Piezocone (SCPTu);
- A piezocone (CPTu) is an enhanced cone penetration test (CPT) probe that is able to measure porewater pressure ( $u$ );
- A seismic piezocone (SCPTu) is further enhanced to measure surface generated compression and shear waves at depth; used to define the shear wave velocity of soils.

#### **Equipment Type and Governing Standard:**

- 10 cm<sup>2</sup> seismic piezocone;
- 150 cm<sup>2</sup> friction sleeve;
- manufactured by Applied Research Associates, Inc.;
- ASTM Specification D3441.

#### **PCPT Investigation Objectives:**

- evaluate soil type and soil stratigraphy;
- estimate the relative density of granular soils and in situ undrained shear strength of cohesive soils.

#### **Soil Behaviour Type (SBT):**

- The SBT is selected based on a soil's response to cone penetration, which is different from an explicit soil type defined by specified laboratory testing procedures, but is normally what the geotechnical engineer requires for design purposes.
- The SBT can be classified on the basis of the soil friction ratio,  $f_s$ ; ratio between the side shear on the friction sleeve and cone tip resistance.
- The SBT can also be classified on the basis of the normalized pore pressure,  $B_q$ ; a function of the pore water response and the cone tip resistance.
- The "CPT Soil Behaviour Type Legend" used for this project is attached.

#### **Canadian Foundation Engineering Manual (3<sup>rd</sup> Edition) Statement on the CPT**

- "The most significant advantage that the electric cone penetrometers offer is their repeatability and accuracy."
- "One of the most important applications of the cone penetration test is to accurately determine the soil profile."

#### Key References:

T. Lunne, P.K. Robertson, and J.J.M. Powell (1997). "Cone Penetration Testing in Geotechnical Practice"; Spon Press.

P.W. Mayne (1986). "CPT indexing of in situ OCR in Clays"; Proceedings of the ASCE Specialty Conference In Situ '86: Use of In Situ Tests in Geotechnical Engineering, Blacksburg, 780-93, ASCE.

P.K. Robertson and R.G. Campanella (1988). "Guidelines for geotechnical design using CPT and CPTU"; University of British Columbia, Vancouver, Department of Civil Engineering, Soil Mechanics Series 120.



### Terminology and Key Engineering Relationships

Parameter	Description	Symbol/Equation	Reference
Depth	Depth of the centroid of the sensor		
Elevation	Elevation of centroid of the sensor	Ground Surface – Depth	
Sleeve Stress	Sleeve Stress – interpolated to the depth of the tip	$f_s$	
Tip Stress, Uncorrected	Measured Tip Stress	$q_c$	
Tip Stress COR	Tip Stress, corrected for probe geometry	$q_t = q_c + u_2 x(1 - a)$	
Ratio COR	Friction Ratio	$R_f = \frac{f_s}{q_t} \times 100\%$	
Pore Pressure	Measured Pore Pressure	$u_2$	
Soil Behaviour Type	Soil Behaviour Type	$SBT'$	Lunne, Robertson and Powell, 1997
Overburden Stress		$\sigma_{vo} = \sum_{i=1}^n \gamma_i x h_i$	
Effective Overburden Stress		$\sigma'_{vo} = \sigma_{vo} - u_o$	
Normalized Tip Stress		$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$	Lunne, Robertson and Powell, 1997
Normalized Friction Ratio		$F_r = \frac{f_s}{q_t - \sigma_{vo}}$	Lunne, Robertson and Powell, 1997
Normalized Pore Pressure		$B_q = \frac{\Delta u}{q_t - \sigma_{vo}}$ where $\Delta u = u_2 - u_o$	Lunne, Robertson and Powell, 1997

K:\Divisions\GeoMaterials\CPT\CPT

Tools\Terminology

Used

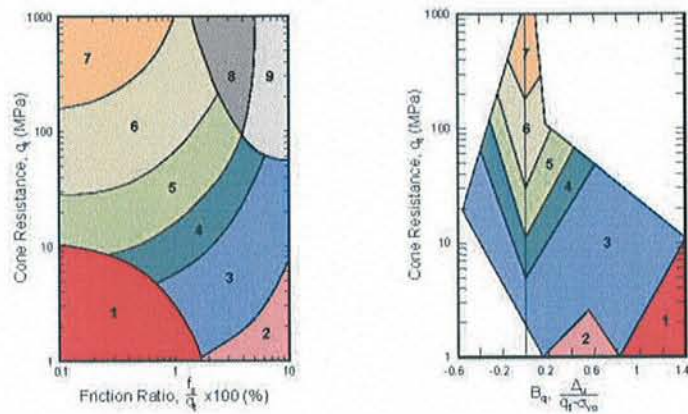
on

SCPTu

Records.doc



# CPT Soil Behavior Type Legend (Robertson et al. 1990)



Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Soils-Peats
3	Clays; Clay to Silty Clay
4	Silt Mixtures; Clayey Silt to Silty Clay
5	Sand Mixtures; Silty Sand to Sandy Silt
6	Sands; Clean Sands to Silty Sands
7	Gravelly Sand to Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine Grained*

\*Overconsolidated or Cemented.

# Jacques Whitford

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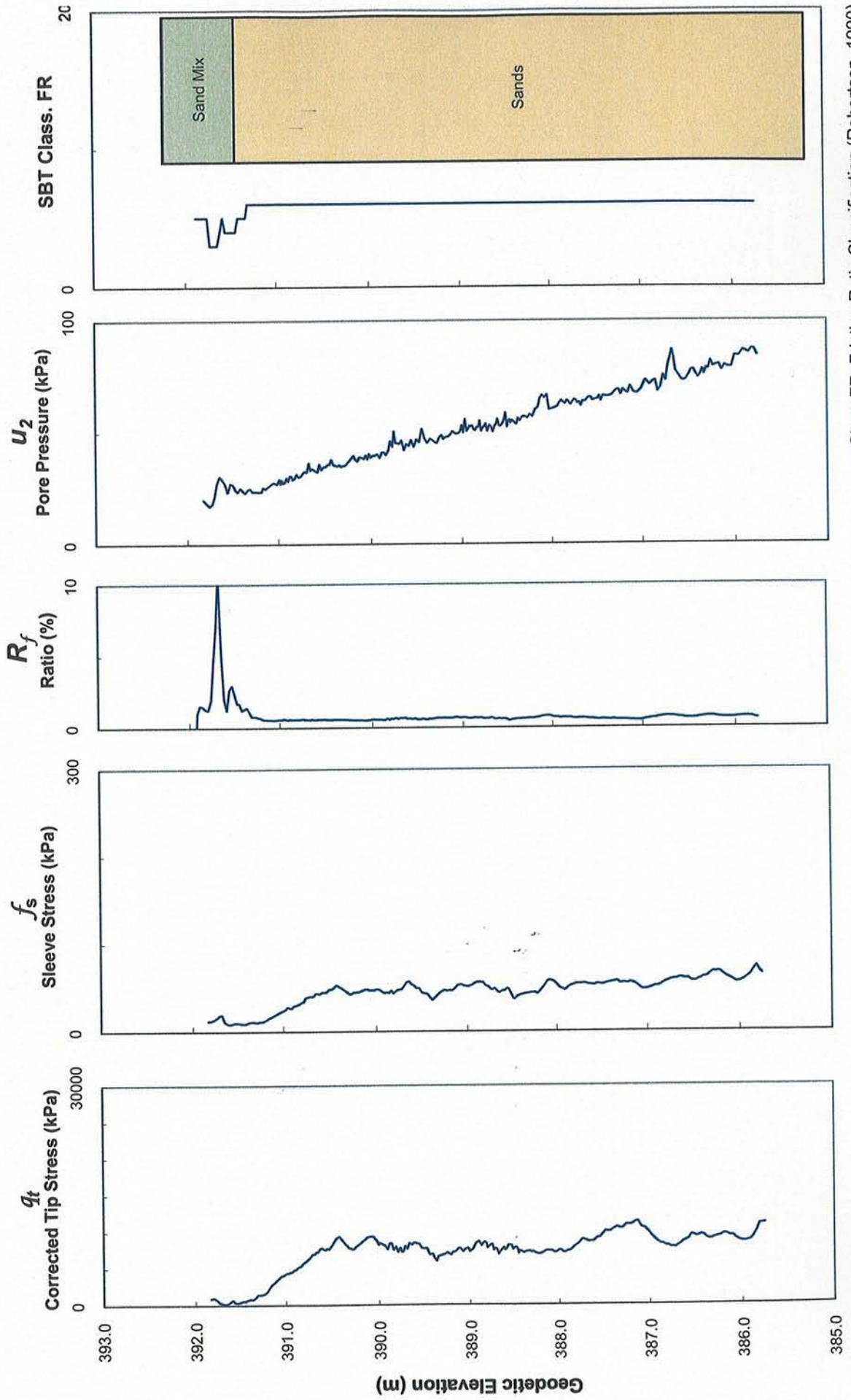
Ground Surface Elevation: 392.6 m  
SCPTu Start Elevation: 392.0 m  
Groundwater Elevation: 392.0 m

Client: McCormick Rankin Corporation

Project: MTO WP 545-93-00, Kearney Creek Bridge Replacement

Test Date: April 26, 2006  
Project No. 1023332  
N5048281.0 E387501.8

CPT 07-8







# Jacques Whitford

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Tel: (613) 738-0708 Fax: (613) 738-0721

Elevation: 393.0 m

SCPTu Start Elevation: 391.5 m

Groundwater Elevation: 392.0 m

Client: McCormick Rankin Corporation

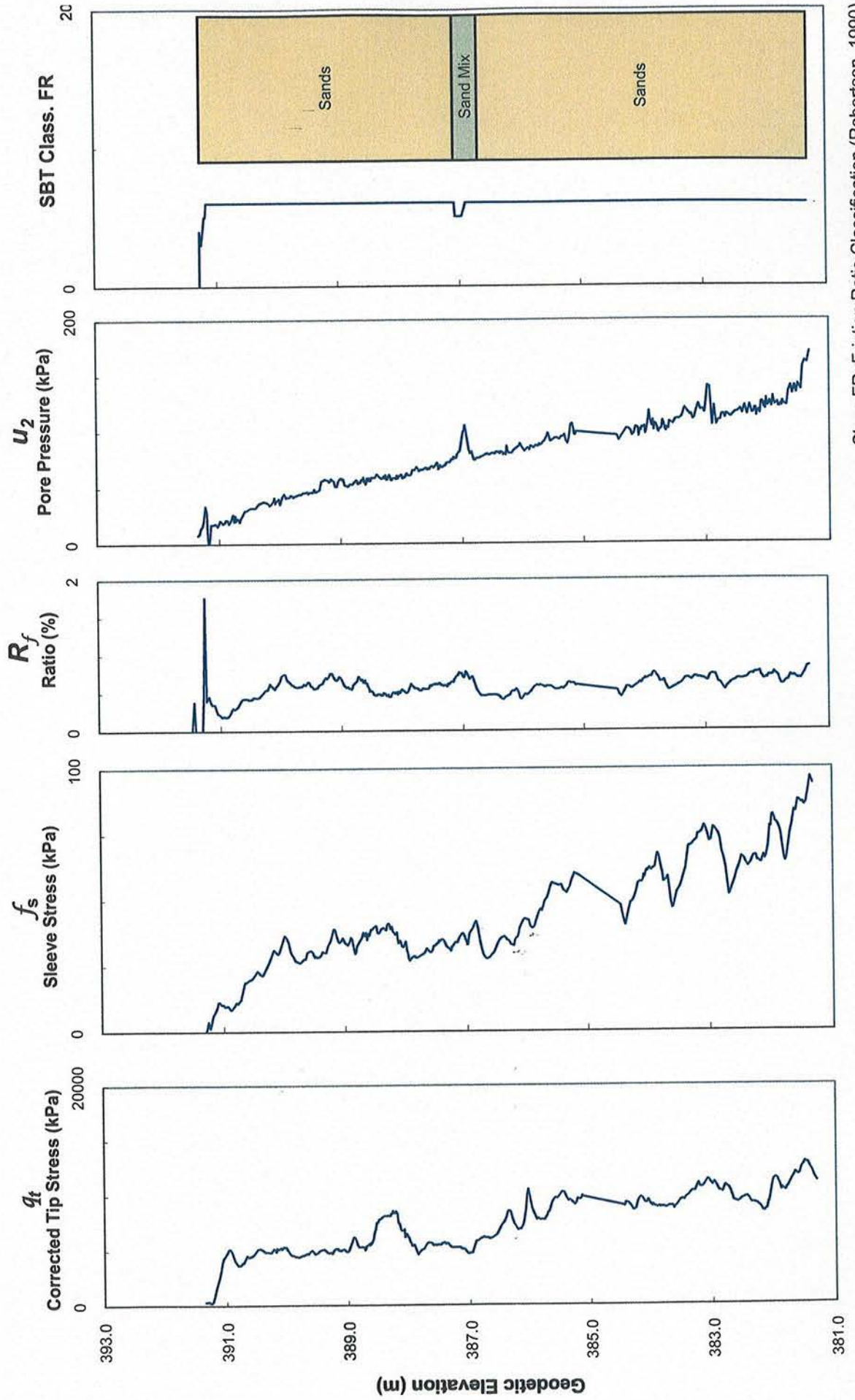
Project: MTO WP 545-93-00, Kearney Creek Bridge Replacement

Test Date: April 26, 2006

Project No. 1023332

N5048263.2 E384531.7

CPT 07-10



Class FR: Friction Ratio Classification (Robertson, 1990)

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

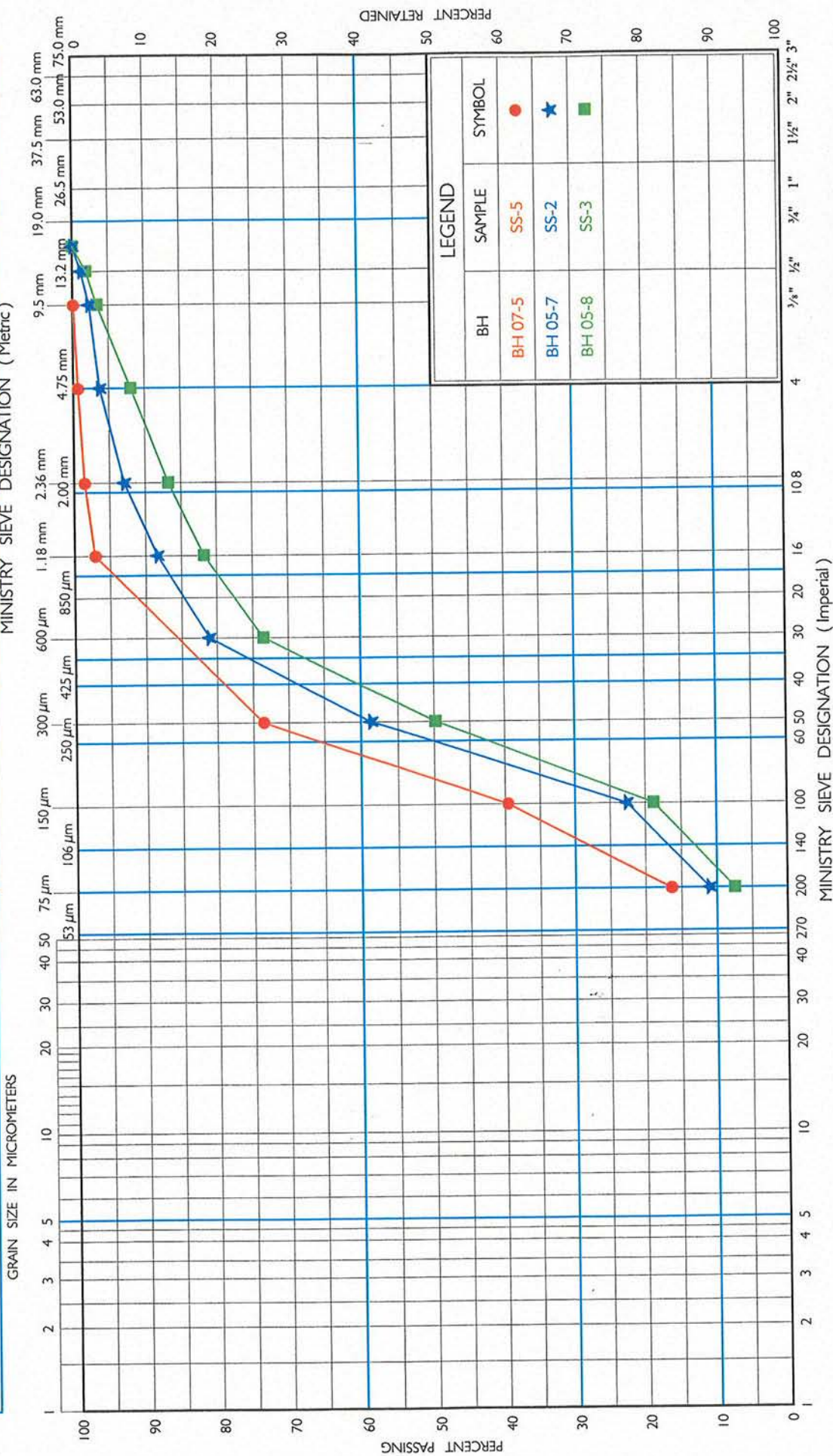


FIG No 1

GRAIN SIZE DISTRIBUTION

POORLY-GRADED SAND WITH SILT TO SILTY SAND



# UNIFIED SOIL CLASSIFICATION SYSTEM

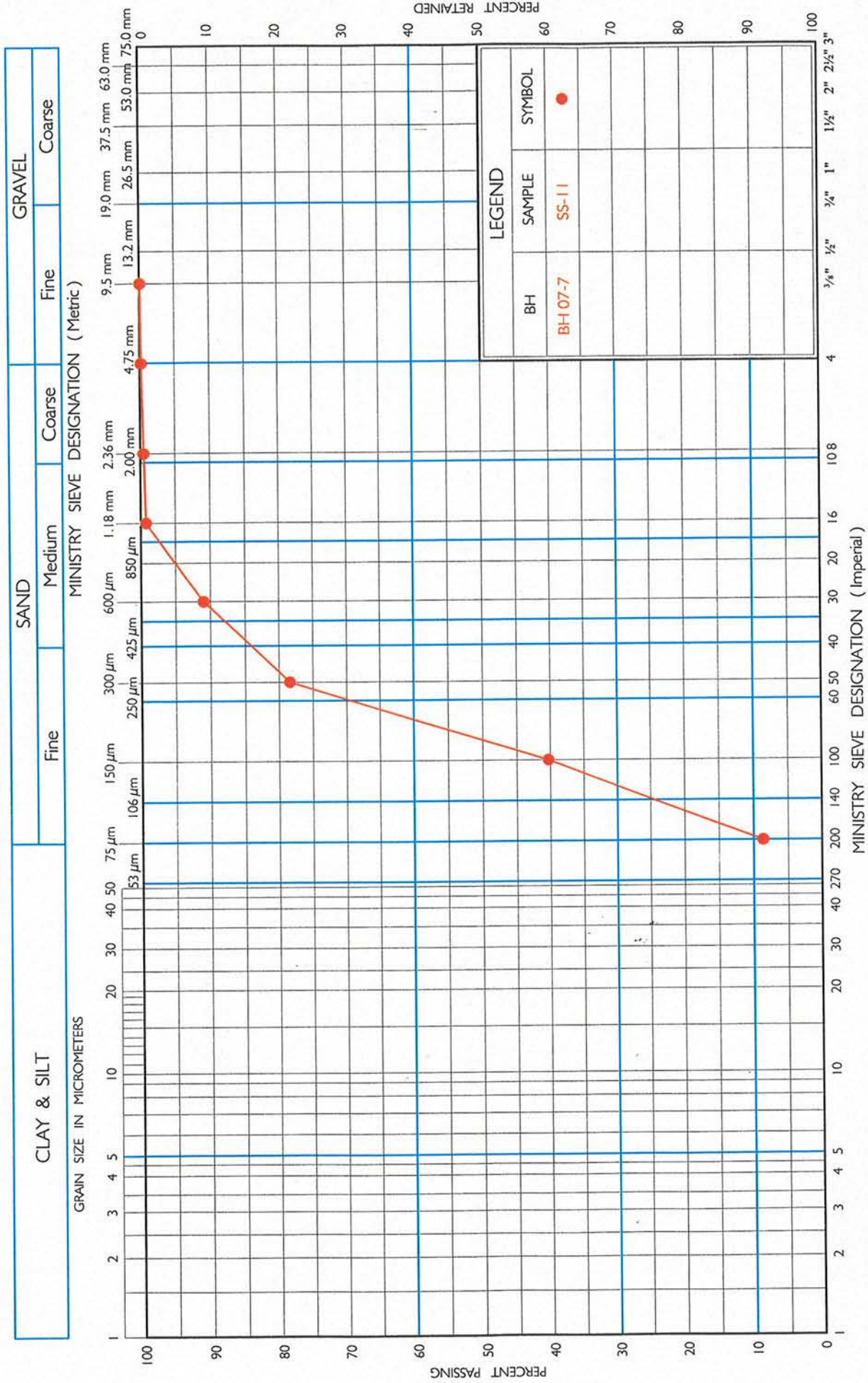


FIG No 2

## GRAIN SIZE DISTRIBUTION

POORLY-GRADED SAND WITH SILT (SP-SM)

GWP 545-93-00



# UNIFIED SOIL CLASSIFICATION SYSTEM

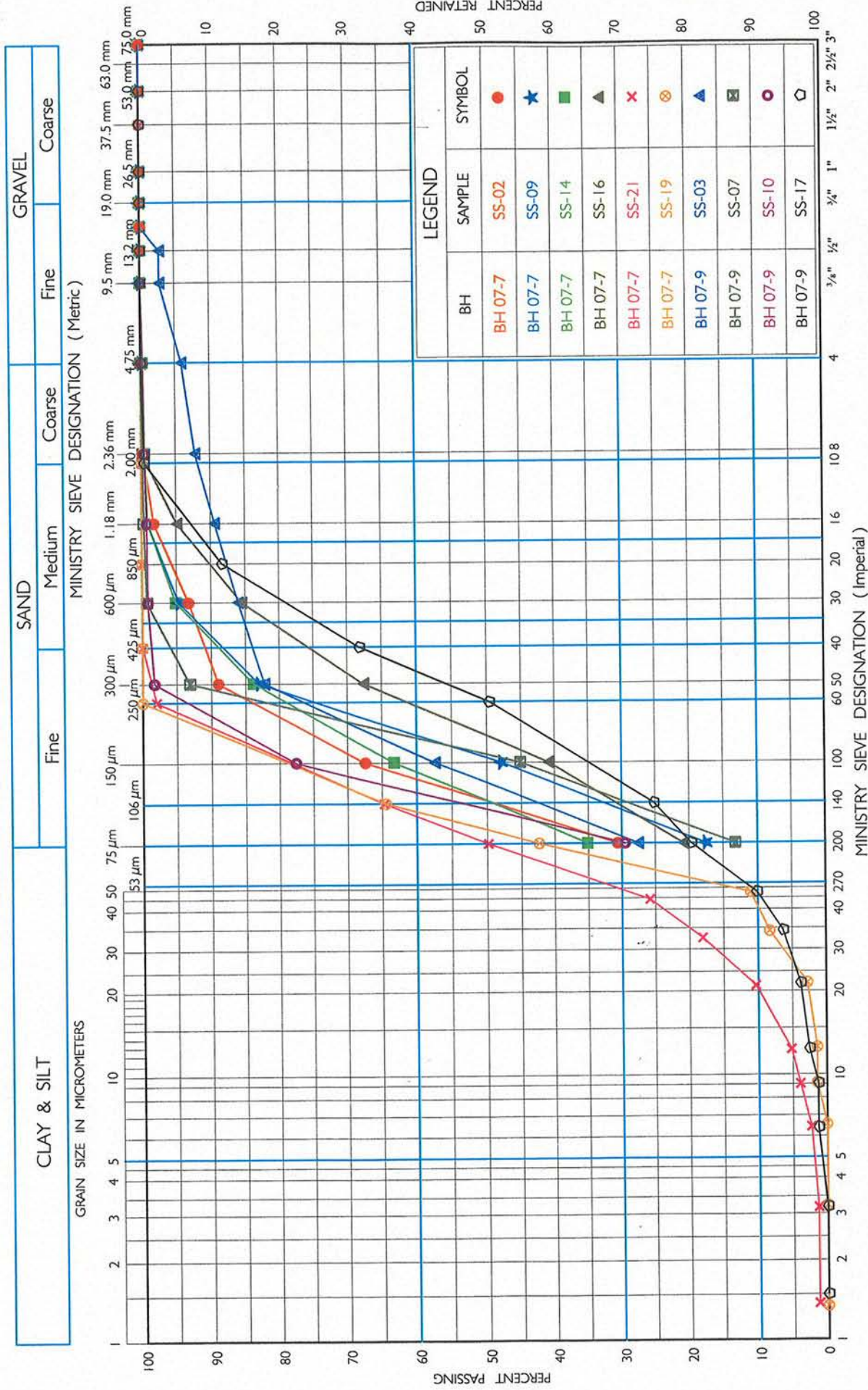
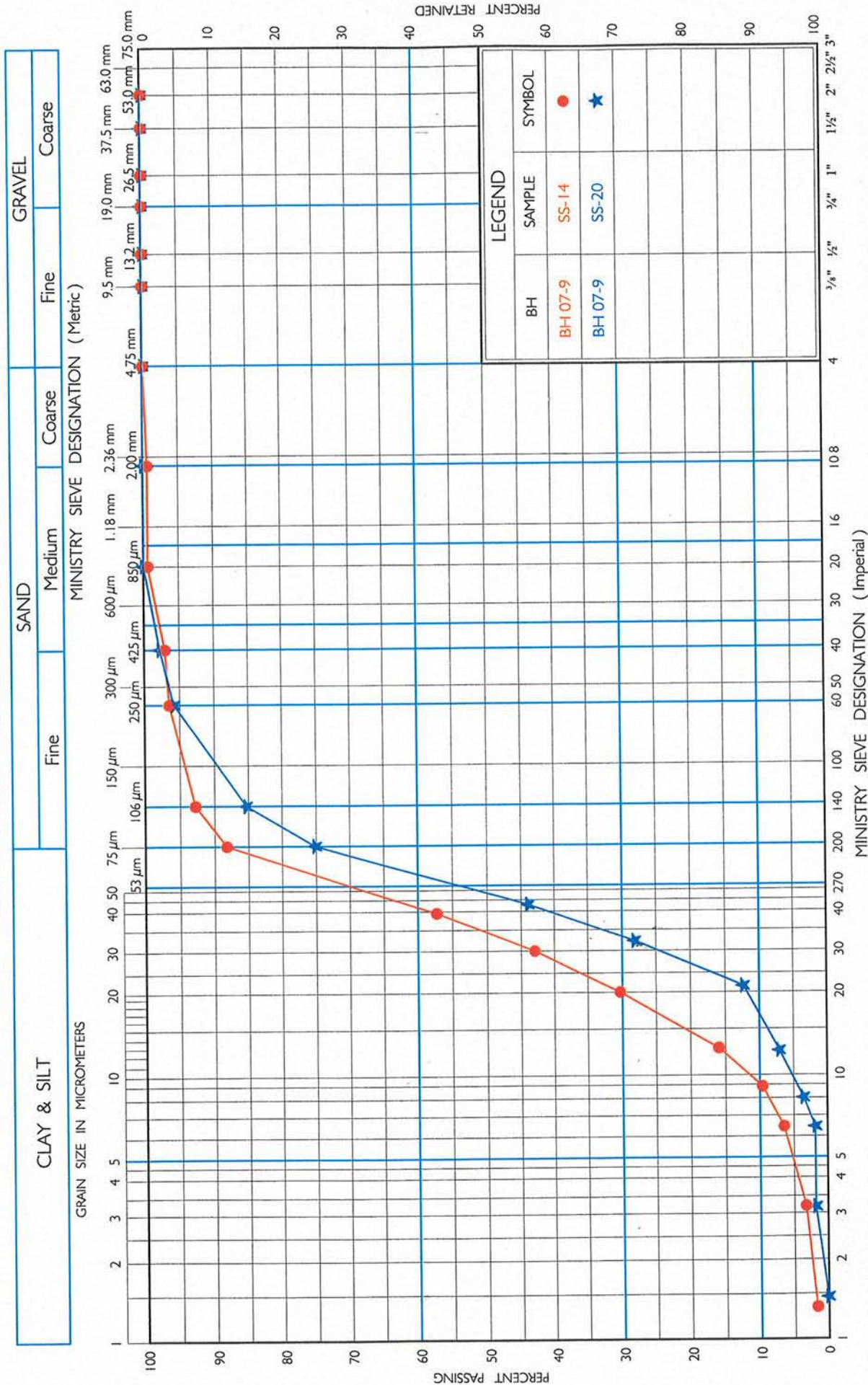


FIG No 3  
GRAIN SIZE DISTRIBUTION  
SILTY SAND (SM)

GWP 545-93-00



# UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION  
SILT / SILT WITH SAND (ML)

FIG No 4

---

GWP 545-93-00

**Rock Core Summary Table**  
**W.P. 302-89-00**

Borehole #	Sample #	Recovery (%)	R.Q.D. (%)	Unconfined Compressive Strength (MPa)	Description
07-7	26	100	64	-	Biotite GNEISS, black, white and pink, fair, moderate to slightly weathered, close to moderately spaced fractures, 30 to 45 degree dip
	27	90	50	103, 107	
07-9	24	88	87	145	Biotite GNEISS, black, white and pink, fair to excellent, moderate to slightly weathered, close to moderately spaced fractures, 30 to 40 degree dip
	25	96	92	94	
	26	92	95	-	

P:\2007\1023332\Kearney\Reports\Rock Core Summary Table.xls

# **APPENDIX C**

**NRCAN Seismic Hazard Calculation  
Characterization of Liquefaction Resistance**





# 2005 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: Paul Carnaffan, Jacques Whitford Limited

June 15, 2007

Site Coordinates: 45.5694 North 78.4407 West

User File Reference: Hwy 60 - Kearney Creek

## National Building Code ground motions:

**2% probability of exceedance in 50 years (0.000404 per annum)**

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.284	0.155	0.072	0.024	0.162

**Notes.** Spectral and peak hazard values are determined for firm ground (NBCC 2005 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. *These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.*

## Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.054	0.130	0.188
Sa(0.5)	0.026	0.066	0.099
Sa(1.0)	0.010	0.030	0.046
Sa(2.0)	0.003	0.009	0.014
PGA	0.033	0.078	0.110

## References

**National Building Code of Canada 2005 NRCC no. 47666;** sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3

**Appendix C:** Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

**User's Guide - NBC 2005, Structural Commentaries NRCC no. 48192**

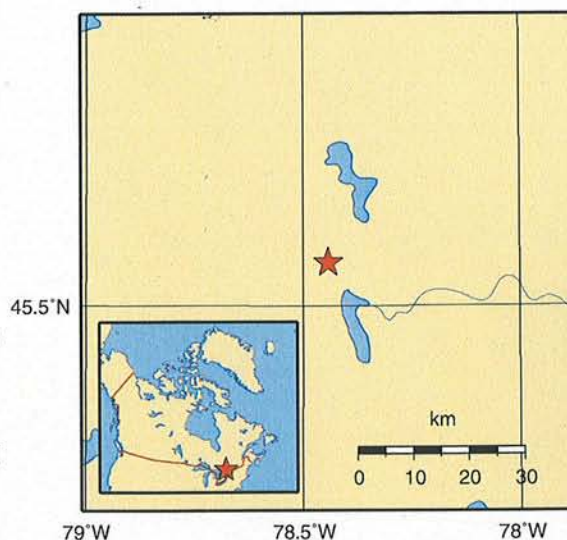
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File xxxx**

Fourth generation seismic hazard maps of Canada: Grid values to be used with the 2005 National Building Code of Canada (in preparation)

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information

Aussi disponible en français



Natural Resources  
Canada

Ressources naturelles  
Canada

Canada

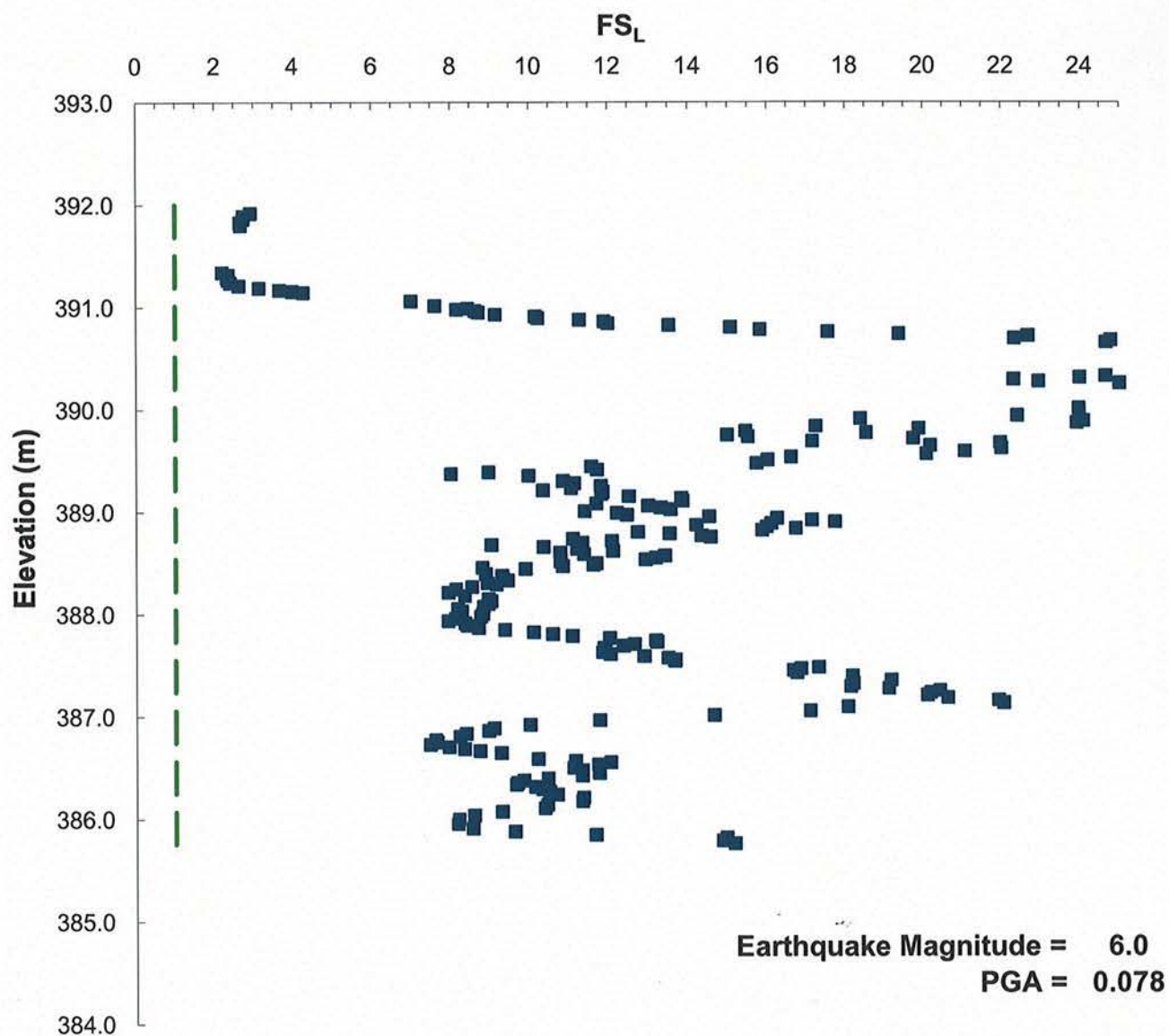




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## CHARACTERIZATION OF LIQUEFACTION RESISTANCE



$FS_L$  = Factor of Safety against Liquefaction

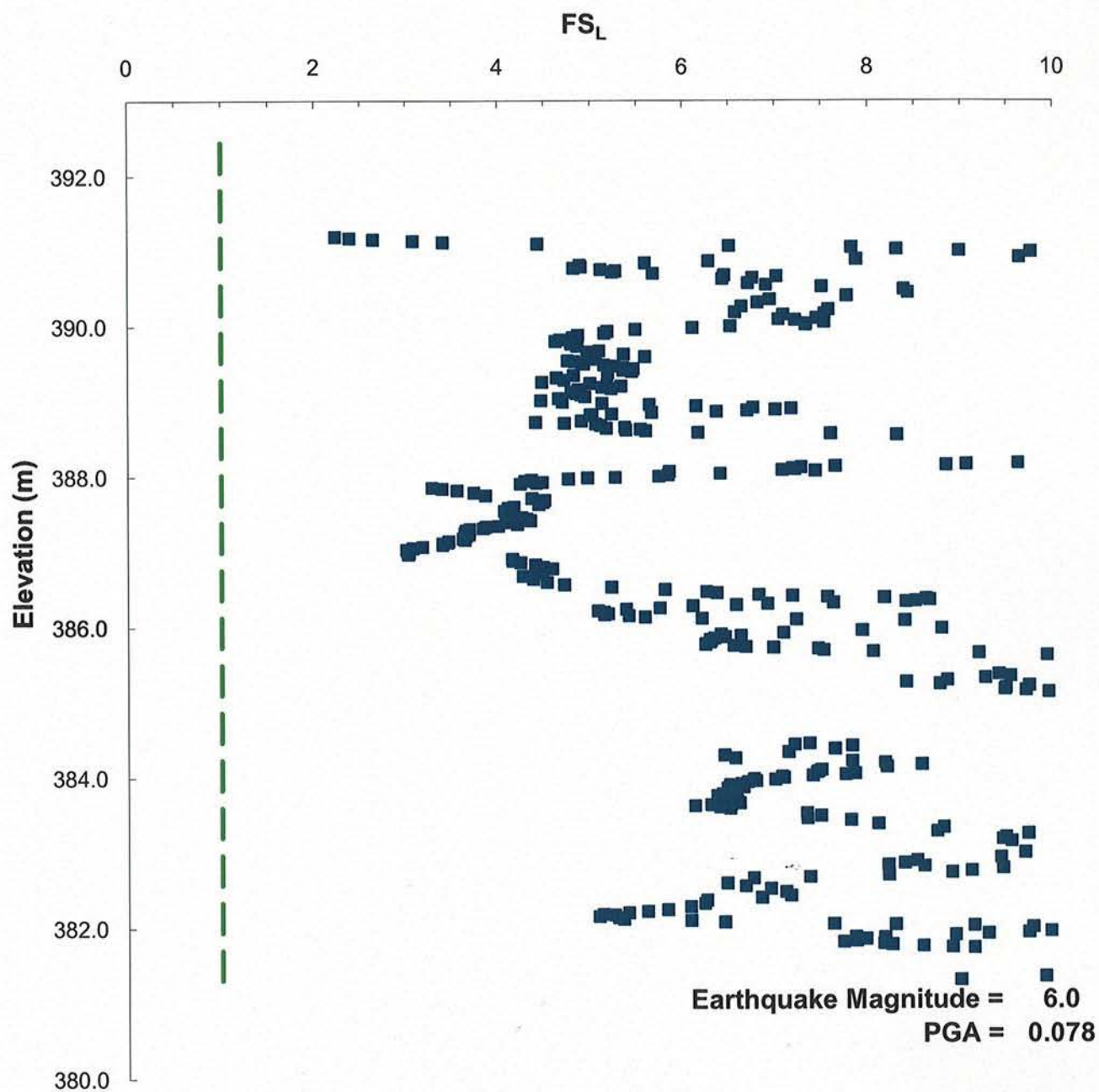
The Canadian Foundation Engineering Manual defines  $FS_L$  as the "soil deposit's cyclic resistance ratio (CRR)" divided by the "earthquake induced cyclic stress ratio (CSR)"



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The Canadian Foundation Engineering Manual defines  $FS_L$  as the "soil deposit's cyclic resistance ratio (CRR)" divided by the "earthquake induced cyclic stress ratio (CSR)"