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# **PRELIMINARY REPORT**

FOUNDATION INVESTIGATION  
AND DESIGN  
W.P. 545-93-00  
HIGHWAY 60 –  
KEARNEY CREEK  
BRIDGE REPLACEMENT

Totten Sims Hubicki

PROJECT NO. ONO11685  
GEOCRES NO. 31F-147

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# **PROJECT NO. ONO11685**

## **PRELIMINARY REPORT – FOUNDATION INVESTIGATION AND DESIGN**

**TO**

**Totten Sims Hubicki  
300 Water Street  
Whitby, Ontario  
L1N 9J2**

**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-147**

---

**June 2006**

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Borehole Records

Grain Size Distribution Test Results

# PRELIMINARY 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 in conjunction with the Preliminary Design Study and Environmental Assessment – Highway 60; Clarke Creek and Kearney Creek Bridge Replacements; W.P. 545-93-00.

This report presents the results of a preliminary foundation investigation carried out for the proposed replacement of the existing Kearney Creek Bridge on Highway 60, in Algonquin Park.

The preliminary foundation investigation was carried out in general accordance with our proposal number ONO 030438 dated June 17, 2003. Authorization to proceed was provided by the Ministry of Transportation of Ontario (MTO) under Agreement Number 4005-A-000302 with Totten Sims Hubicki Limited (TSH), the Preliminary 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.

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## 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. ONO11685-1 and ONO11684-2 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 surveyed water level at the time of the investigation was 392.4 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 2.9 m below the top of pavement on the existing bridge deck at the time of the investigation. The banks of the creek were covered with snow and ice at the time of the investigation but based on the topography, they appear to be steeply sloped for approximately 1 m above water level and then very gradually sloped upwards away from the creek. 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 cross sections are shown on Drawing No. ONO11685-1 and Drawing No. ONO11685-2, provided in Appendix A.

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## 3.0 PROCEDURE

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### 3.1 Field Investigation

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The site soil conditions were investigated with a borehole drilling investigation and laboratory testing program. The drilling was carried out using a combination of a truck-mounted CME-55 drill rig and a track-mounted CME-75 drill rig between January 21 and February 2, 2005.

A total of eight (8) boreholes, designated as 05-1 through 05-8, were put down during the field investigation. Boreholes 05-1 through 05-4 were located beneath the proposed abutments and embankments along the proposed new alignment. Boreholes 05-5 through 05-8 were drilled through the existing embankments and/or just behind the existing bridge abutments.

The boreholes were advanced through the overburden using hollow stem augers to a depth of approximately 10 m. Below this depth, the boreholes were further advanced using casing and drilling mud in order to balance the pressure within the borehole and minimize sand coming up inside the augers/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 (760 mm at shallow depths to 3 m at depths greater than 15 m). The boreholes at the foundation units were terminated at SPT refusal in accordance with the Terms of Reference for this project. SPT refusal is defined as 100 or more blows for 300 mm of penetration. 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.

Standpipes were installed in Boreholes 05-2 and 05-4. The standpipes consisted of slotted flexible poly-tube with a diameter of 25 mm. Groundwater levels were measured on February 10, 2005.

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

---

### 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 is identified on the existing Hwy 60 profile shown on Plate A-2, prepared by TSH, 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. Four representative 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.

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. In general, the subsurface profile beneath the proposed new alignment (Boreholes 05-1 to 05-4) consists of an upper sand layer with varying

amounts of silt, overlying silt and sand to silt deposits. Within the existing roadway platform (Boreholes 05-5 to 05-8), the subsurface profile consists of the pavement structure overlying the existing approach fill, over native soils consisting of an upper sand layer with varying amounts of silt, overlying silt and sand to silt deposits.

Borehole location plans and stratigraphic sections of the soils encountered within the boreholes are provided on Drawings ONO11685-1 and ONO11685-2 in Appendix A.

---

#### 4.1.1 Fill: Sand, Trace Silt, Trace Gravel to Gravelly Sand, Trace 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 investigation. The moisture content of the 8 samples of fill tested ranged from 5% to 34% and averaged 18%. 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.

---

#### 4.1.2 Sand, Trace to Some Gravel, Trace to Some Silt

A layer of sand with trace to some gravel, trace silt was observed in Boreholes 05-2, 05-3, 05-5, 05-6 and 05-7. The thickness of this deposit, where fully penetrated, ranged from 0.8 m to 6.3 m. The base of the unit varied from elevation 384.6 m to 390.2 m. SPT 'N' values ranged from 3 to 95 and averaged 29, suggesting a generally compact state. The moisture content of the 5 samples tested ranged from 19% to 22% with an average of 21%. Grain size analysis of two samples indicated that the samples contained 0% gravel, 87% to 93% sand and 7% to 13% silt and clay sized particles. The results of the grain size distribution testing are shown on the Figures in Appendix B. This material corresponds to an SP-SM soil using the MTO Soil Classification System.



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#### 4.1.3 Silty Sand / Sand with Silt

A layer that ranged from silty sand to sand with silt was observed in Boreholes 05-1, 05-2, 05-3 and 05-8. Where this deposit was fully penetrated, the thickness ranged from 3.8 m to 6.5 m with an average of 5.2 m. The base of the unit varied from elevation 380.6 m to 386.4 m. SPT 'N' values ranged from 3 to 42 and averaged 16, suggesting a generally compact state. The moisture content of the 10 samples tested ranged from 21% to 25% with an average of 23%. Grain size analysis of the five samples tested indicated that they contained 0% gravel, 66% to 80% sand and 20% to 34% silt and clay sized particles. The results of the grain size distribution testing are shown on the Figures in Appendix B. This material corresponds to an SM soil using the MTO Soil Classification System.

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#### 4.1.4 Silt and Sand / Sandy Silt / Silt, Some Sand

A deposit ranging from silt and sand to silt, some sand was encountered beneath the sand / silty sand / sand with silt deposits in Boreholes 05-2, 05-3, 05-4, 05-6 and 05-7. Boreholes 05-2, 05-3 and 05-6 were terminated upon SPT refusal within these deposits at depths ranging from 18.9 m to 42.8 m below existing grade (elevation 376.4 m to 349.8 m). SPT 'N' values ranged from 7 to 124 and averaged 26 (excluding the three samples in which SPT refusal was achieved), suggesting a generally compact state. The moisture content of the 19 samples tested ranged from 18% to 28% with an average of 21%. Grain size analysis of six samples indicated that this deposit contained 0% to 6% gravel, 20% to 59% sand and 41% to 80% silt and clay sized particles. The results of the grain size distribution testing are shown on the Figures in Appendix B. These materials correspond to SM and ML soils using the MTO Soil Classification System.

---

#### 4.1.5 Bedrock

Bedrock was not encountered within the depth of investigation in any of the boreholes.

---

### 4.2 Groundwater

Groundwater levels were measured in the standpipes on February 10, 2005. The water levels ranged from 1.2 m to 6.0 m below ground surface (elevation 392.1 m to 392.8 m). The water level in Kearney Creek on January 20, 2005, was surveyed to be at elevation 392.4 m. The measured groundwater levels 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.

---

## 5.0 CLOSURE

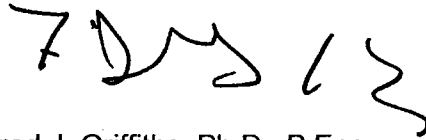
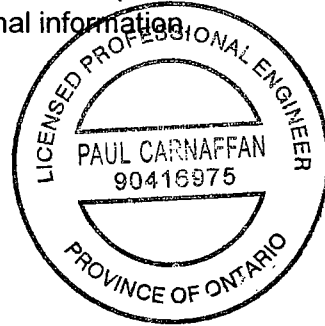
A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately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



# PRELIMINARY 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 structure 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.

Consideration is being given to reconstructing the Kearney Creek Structure on a new alignment approximately 16 m to the north. This option would improve highway geometry and facilitate single stage construction. This has been identified as the preferred alternative.

The other alignment alternative is to reconstruct the bridge along the existing alignment. This alternative would require a detour structure constructed to the north of the existing bridge.

In addition to the two alignment options, different replacement structure types are being considered, including a 16 m to 17 m single span CPCl 900 girder bridge with integral abutments; and a 6 m span by 2.5 m rise corrugated steel plate box culvert. It is understood that the bridge alternative has been identified as the preferred alternative.

It is noted that should the CPCI girder bridge be selected it may be necessary to raise the grade by 0.4 m in the area of the structure in order to achieve the required 1.0 m clearance above the 100-year storm water level and to improve an existing sub-standard sag curve. All options include 2H:1V foreslopes and sideslopes. The underside of the pile caps for the bridge would be at approximately elevation 390.5 m and the underside of footings or pile caps for the culvert option would be at approximately elevation 389.5 m.

The approach embankments will be up to 3.5 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 sand, trace silt, to silt, some sand. 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 preliminary design purposes, the soils will be considered to be compact to dense, with a design N-value of 15 blows/300 mm. For preliminary design purposes, the native non-cohesive soils at this site have been considered to have a unit weight of  $19.0 \text{ kN/m}^3$  and a minimum angle of internal friction of 30 degrees.

---

## 6.3 Foundation Options

For design purposes, the soil conditions along both the new and existing alignments are very similar with the exception of the embankment fill present at the existing alignment. The proposed founding elevations for the culvert options are the same for the two alignment options. The proposed founding elevations for the bridge options are the same for the two alignment options. The foundation options have been assessed for each of the two structure options, as follows:

---

### 6.3.1 Bridge Structure Alternative – New or Existing Alignment

The following table compares the available foundation options considered for the Bridge structure option regardless of which alignment is selected:

**Table 6.1: Foundation Comparison for Bridge Option**

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</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>• requires additional excavation below water level</li> </ul>	Low	<ul style="list-style-type: none"> <li>▪ excavation below waterline require / 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 m</li> </ul>	Moderate	<ul style="list-style-type: none"> <li>▪ bedrock depth not yet confirmed by coring</li> <li>▪ piles reach refusal at greater depth / higher cost</li> </ul>
Driven H-piles in silt and sand	<ul style="list-style-type: none"> <li>• readily incorporated into integral abutment design</li> <li>• moderate geotechnical resistance</li> </ul>	<ul style="list-style-type: none"> <li>• anticipated length of 40 m</li> </ul>	Moderate	
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> <li>▪ bedrock depth not yet confirmed by coring. Caisson length may need to be extended</li> </ul>

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

### 6.3.2 Culvert Alternative – New or Existing Alignment

The following table compares the available foundation options considered for the culvert option:

**Table 6.2: Foundation Comparison for Culvert Option**

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Spread Footings	<ul style="list-style-type: none"><li>• moderate geotechnical resistance</li></ul>	<ul style="list-style-type: none"><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></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></ul>	<ul style="list-style-type: none"><li>• requires additional excavation below water level</li></ul>	Low	<ul style="list-style-type: none"><li>• excavation below waterline require / 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>• high geotechnical resistance on bedrock</li></ul>	<ul style="list-style-type: none"><li>• anticipated length of 40 m</li></ul>	Moderate	<ul style="list-style-type: none"><li>• bedrock depth not yet confirmed by coring</li><li>• piles reach refusal at greater depth / higher cost</li></ul>
Driven H-piles in silt and sand	<ul style="list-style-type: none"><li>• moderate geotechnical resistance</li></ul>	<ul style="list-style-type: none"><li>• anticipated length of 40 m</li></ul>	Moderate	
Caissons	<ul style="list-style-type: none"><li>• high geotechnical resistance on bedrock</li></ul>	<ul style="list-style-type: none"><li>• require tremie concrete</li><li>• require cased holes</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>

All options will require excavation below the water level and within the creek bed. The spread footing option (with or without a structural fill pad) will be more economical than driven H-piles. Spread footings are therefore recommended provided they can provide the required geotechnical resistance. If the design loads are too great to allow for a spread footing design, the culvert structure should be founded on H-piles.

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## 7.0 PRELIMINARY RECOMMENDATIONS

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### 7.1 Structure Foundations

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#### 7.1.1 Pile Foundations

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##### **Axial Resistance**

The bridge structure may be supported on steel H-piles. The estimated pile tip elevation is 350 m for both the new and existing alignments and at both the east and west abutments.

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

**Table 7.1: Recommended Pile Design Parameters for HP 310 x 110 Piles**

Founding Material	Estimated Pile Tip Elevation (m)	Factored Axial Geotechnical Resistance at ULS (kN)	Unfactored Resistance at SLS (kN)
Silt and Sand	350.0	1,460	1,200
Bedrock	350.0	2,000	1,560

Note that the pile tip elevation in the above table has been estimated as approximately 5 m below the deepest elevation at which SPT refusal was encountered in the boreholes. See Section 8.0 for recommendations on further investigation.

A geotechnical resistance factor of 0.4 has been applied to generate the factored axial resistance for piles driven into the silt and sand.

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. Note, however, rock coring and rock testing will be required as part of the final design investigation to confirm this design assumption.

The top of the pile is expected to settle 20 mm at the SLS value for piles in the silt and sand layer.

The pile tip for piles set on bedrock is not anticipated to settle. Therefore the unfactored resistance at serviceability limit states (SLS) for piles on bedrock has been established as the load corresponding to 21 mm of elastic compression of the pile.

Downdrag forces are not anticipated at this site.

### Lateral Resistance

For preliminary design purposes, 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 (Non-Cohesive Approach)**

Parameter	OPSS Granular B Type I	Sand / Silty Sand	Silt / Silt and Sand
Bulk Unit Weight, kN/m <sup>3</sup>	21.2	19.0	19.0
Effective Friction Angle, degrees	35	30	29
Coefficient of Passive Earth Pressure	3.7	3.0	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:

$$k_s = nh \ z/d$$

where

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

$nh$  = 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  $nh$  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  $nh$  value of 11,000 kN/m<sup>3</sup> is recommended.

### 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, Isenhowe 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	30 degrees
$\beta$ Coefficient	0.5 (tension)

Resistance Factor	0.3
Design Critical Depth	6.2 m

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 110	350.0	1,000

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

#### **Pile Notes**

Pile tips should be reinforced as per OPSD-3000.100 Type I.

#### **7.1.2 Spread Footing Foundations**

The culvert alternative could be supported on spread footings founded either on the native soil or on a pad of structural fill. The underside of footing elevation is estimated to be 389.5 m at both the east and west abutments for either the new or existing alignment.

Note that a bridge alternative could also be founded on spread footings provided the geotechnical resistance values given in Table 7.4 below are adequate to support the structure.

The factored geotechnical resistance can be increased by constructing a 1000 mm thick pad of compacted OPSS Granular A beneath the foundation. The structural fill pad should extend a minimum of 1000 mm laterally beyond the edges of the footing.

Clearstone should be used as structural fill below the water line. The clearstone should meet the requirements of OPSS 1004 for 19.0 mm Type II. A non-woven Class II geotextile with a thickness greater than 1 mm and a typical FOS of 100 µm should be placed over the clearstone.

## Spread Footings – Preliminary Geotechnical Resistance

The following geotechnical resistances may be used in the preliminary design:

**Table 7.4: Recommended Spread Footing Preliminary Design Parameters**

Founding Layer	Footing Elev. (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Native Soil	389.5	1.0	200	*
	389.5	2.0	235	175
	389.5	3.0	270	130
1000 mm thick Structural Fill Pad over Native Soil	389.5	1.0	420	350
	389.5	2.0	450	200
	389.5	3.0	430	150

In accordance with Section 6.6 of the CHBDC, a resistance factor of 0.5 has been applied to calculate the factored geotechnical resistance at ULS.

The geotechnical resistance at SLS corresponds to a maximum settlement of 25 mm. Note that the ULS value will govern for those options indicated with a \*.

Note that a reduction factor to account for inclined loads will need to be applied in accordance with Section 6.7.4 of the CHBDC.

## Spread Footing – Horizontal Resistance

The unfactored horizontal resistance of spread footings may be calculated using an unfactored coefficient of friction of 0.7 between OPSS Granular A and cast-in-place concrete, and 0.4 between the native soil and cast-in-place concrete.

---

## 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 6.9.3 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 7.5: 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 100 mm diameter subdrain wrapped in geotextile. The subdrain should be installed as per OPSD 3501 and should provide positive drainage to a frost-free outlet. In addition, weep holes through the wall should be provided at regularly spaced intervals. Granular backfill should be designed as per OPSD 3501 using a depth of frost penetration,  $f$ , of 1.9 m.

## 7.3 Seismic Design Considerations

### 7.3.1 Zonal Acceleration Ratio

Table A3.1.7 of the CHBDC indicates that the Zonal Acceleration Ratio for Bancroft, which is 90 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.

### 7.3.2 Soil Profile Type

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

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### 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 results of this assessment revealed that generally this site would not be classified as liquefiable under a 0.1 g earthquake, however, at a few locations, individual low N-values ranging from 1 to 8, within soils with little fine content, were observed, suggesting that limited zones could be liquefiable.

Although the Standard Penetration Test N-value may be useful as a preliminary indicator on the potential for soil liquefaction, it should be used for this purpose with great caution. Some studies have indicated that soils with an average N-value of 12 could have standard deviations in the measured values of 6, where other test methods show no appreciable changes in density. As well, the drilling techniques used in Ontario easily disturb native soils, frequently producing falsely low N-values in clean native sands below the water table.

As a preliminary assessment, the soils at this site are not considered liquefiable under a 0.1 g earthquake, however, as part of the final foundation investigation it is recommended that a static Cone Penetration (CPT) investigation be carried out to further assess the liquefaction potential.

---

### 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 7.6: 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 7.6 deviate only slightly from the static coefficients presented in Table 7.5. This is due to the low zonal acceleration ratio at this site.

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## 7.4 Embankment Design

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

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### 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 carried out 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.

If the replacement structure is constructed along the existing alignment, a 0.4 m increase in the vertical profile is proposed. This increase in the embankment height is expected to result in minimal additional settlement (2 mm). As part of the construction, the existing backfill behind the abutments will be excavated and later replaced. Self settlement of the backfill of as much as 15 mm will occur. This settlement will be complete at the completion of construction.

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## 7.5 Dewatering

It is anticipated that the excavation for the foundations of the abutments and retaining walls will extend to depths ranging from 388.5 m to 390.5 m. 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.

Shoring or a coffer dam and piping system will be required for excavations that extend below the water level in the creek.

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.

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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.

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## 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.

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## 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 Excavation and Backfilling - Structures.



### **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. Shoring design should meet the requirements of Performance Level 2 as per OPSS 539 and should consider sloping backfill and traffic loading.

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.

### **Cement Type and Corrosion Protection**

Four representative 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 7.7: 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

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.

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## 8.0 FUTURE INVESTIGATIONS

Additional investigation is required at this site during the detailed design. The investigation should be carried out to satisfy MTO Foundation protocols and should include the following:

- Bedrock coring to confirm the depth, nature, and strength of the bedrock.
- At least two static cone penetration (CPT) tests using a seismic cone should be carried out to confirm that liquefaction is not a significant issue at this site.

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## 9.0 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.

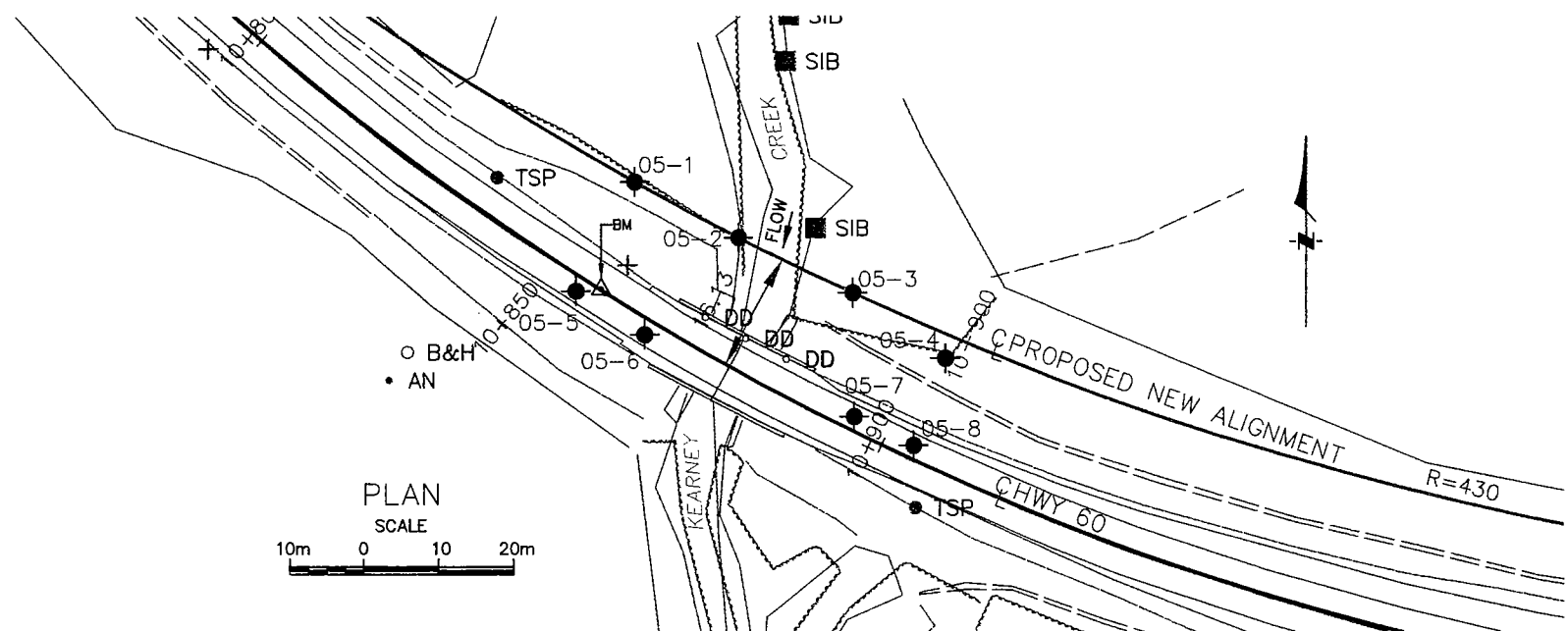
Designated Principal MTO Foundation Contact



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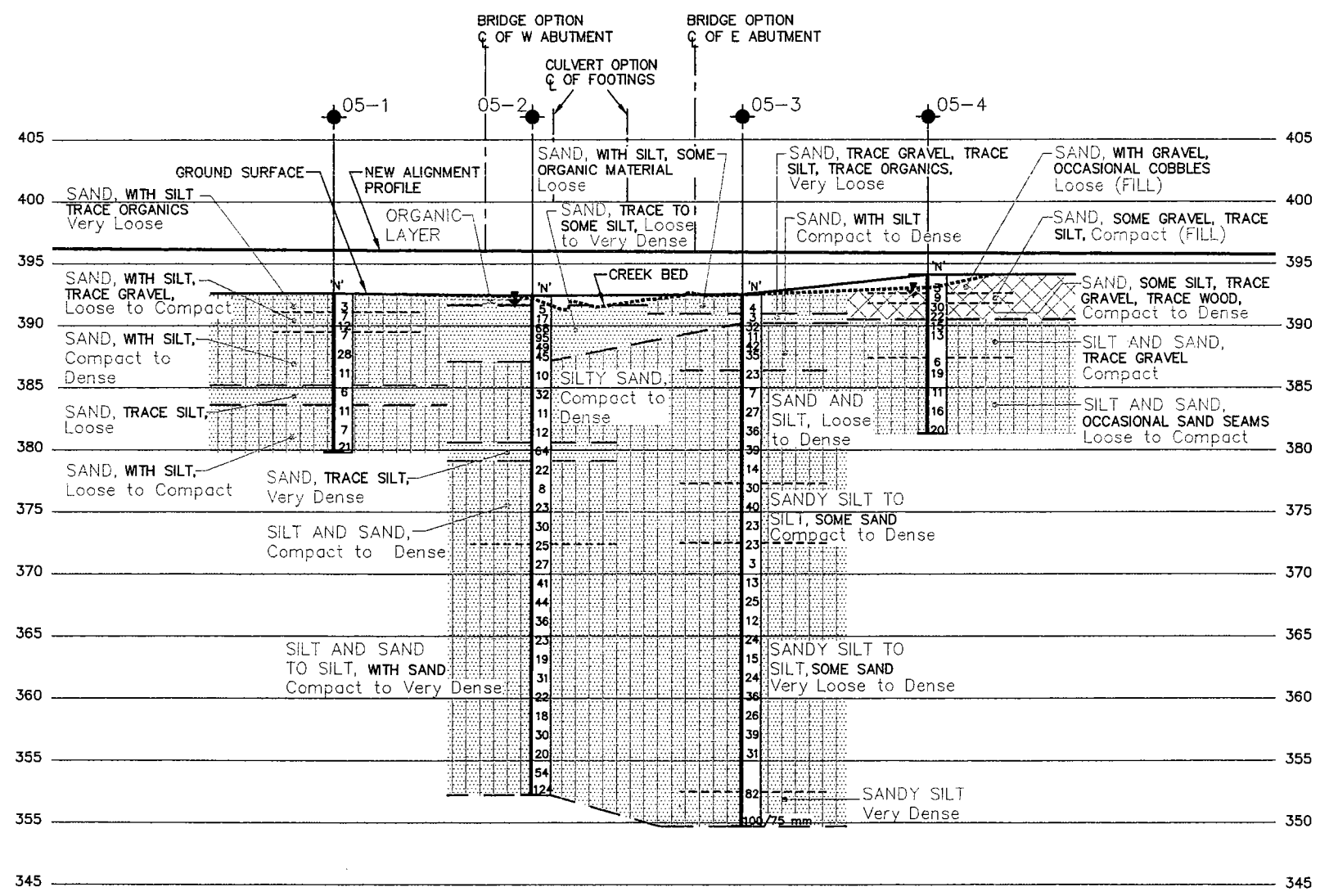
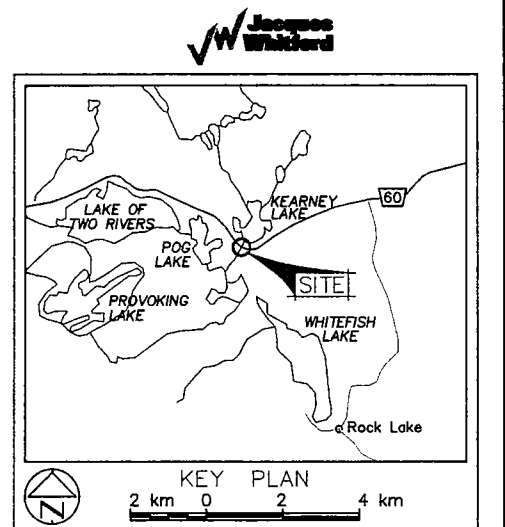
# **APPENDIX A**

## Borehole Location Plans and Profile Plots



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

CONT No - WP No 545-93-00	 SHEET -
KEARNEY CREEK BRIDGE SITE STATION 10+600 - STATION 11+366	
BORE HOLE LOCATIONS & SOIL STRATA	



LEGEND			
◆	Bore Hole		
⊕	Dynamic Cone Penetration Test (Cone)		
⊙	Bore Hole & 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	Elev = 395.5 m Geodetic		
	Reference: TSH profile plate A-2		
No	ELEVATION	STATION	OFFSET
05-1	395.4	10+850	C/L
05-2	395.2	10+866	C/L
05-3	395.3	10+883	C/L
05-4	396.9	10+898	3.0 Rt C/L
05-5	398.0	10+854	1.9 Rt C/L
05-6	398.2	10+865	2.0 Rt C/L
05-7	398.1	10+895	1.8 Lt C/L
05-8	398.3	10+904	1.8 Lt C/L



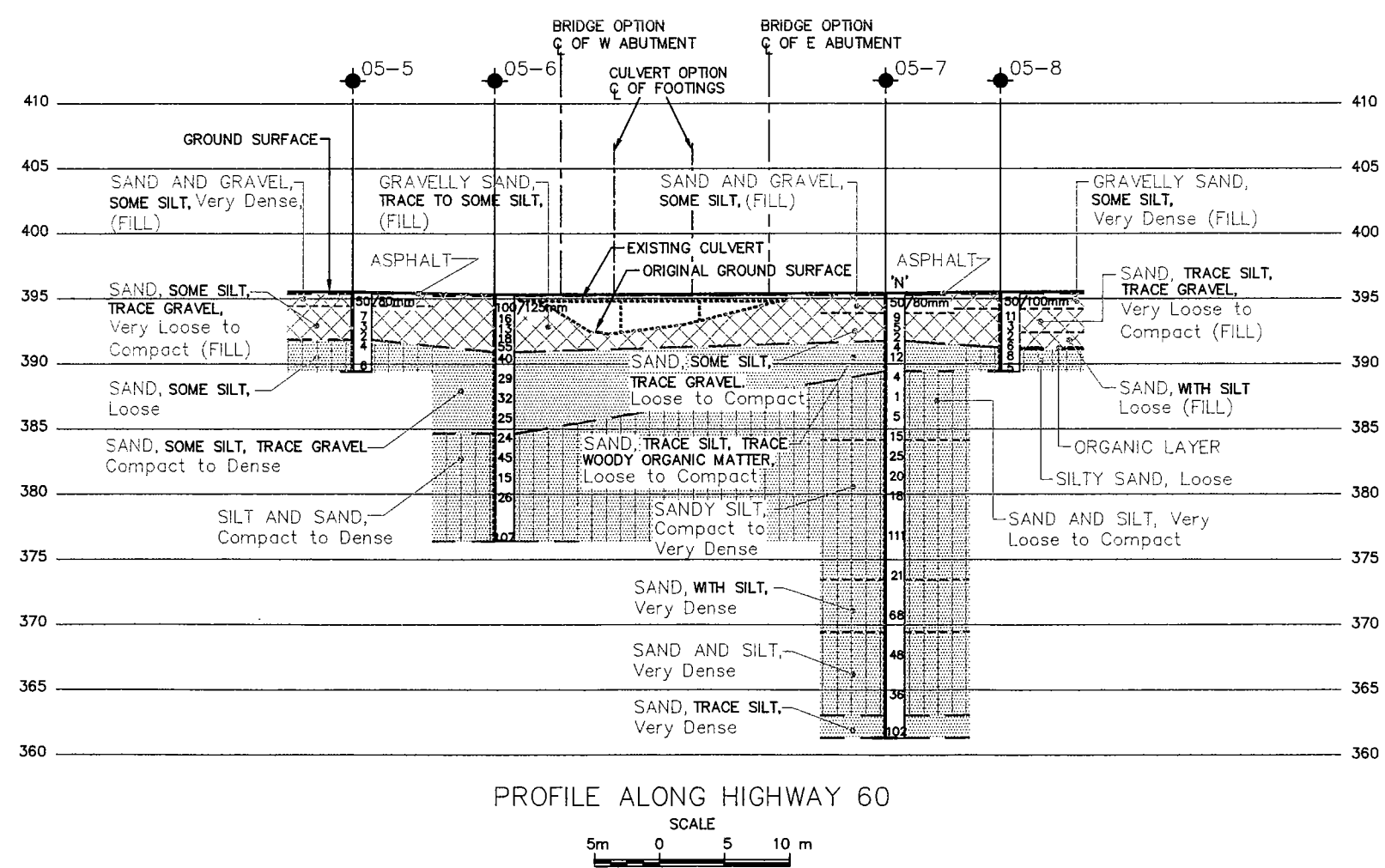
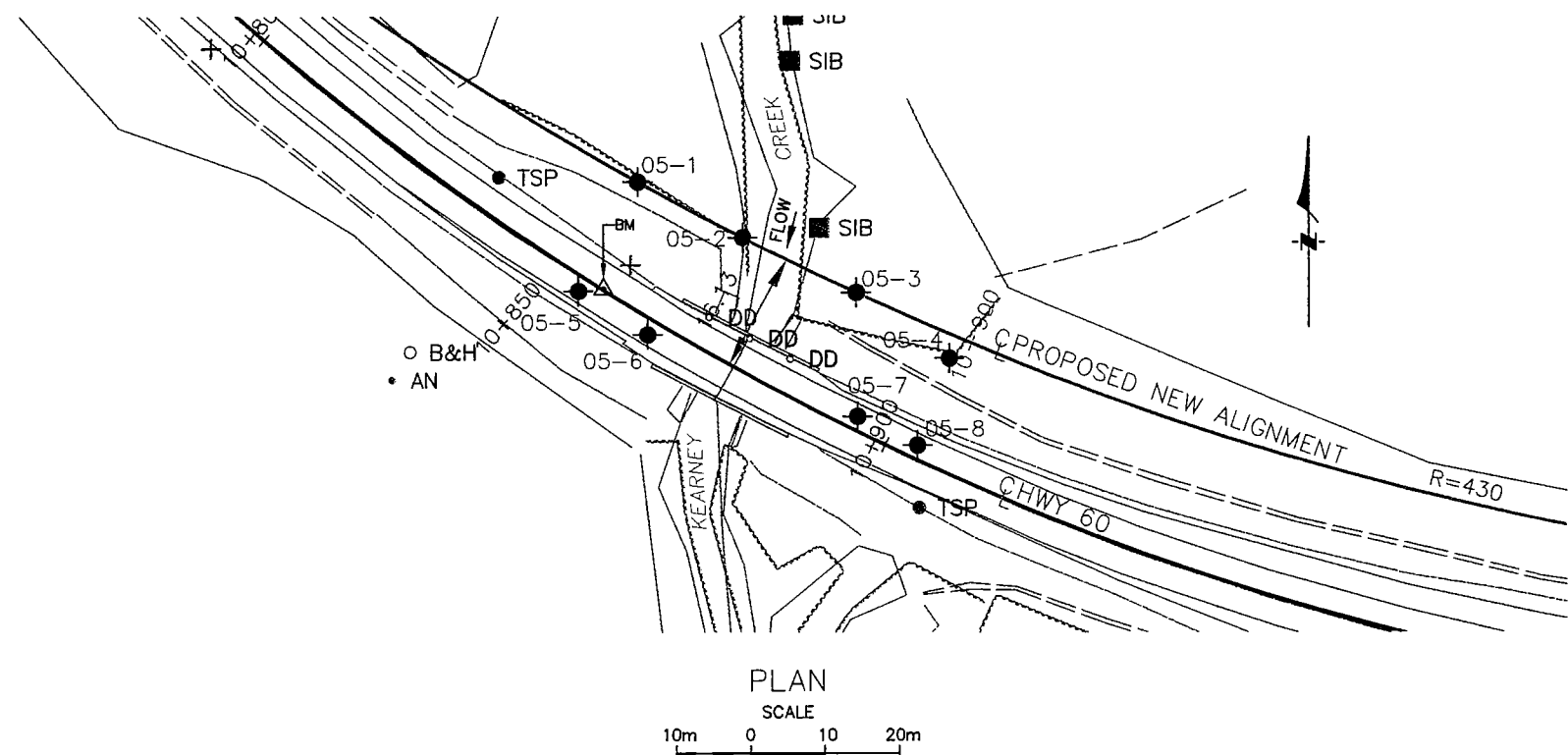
NOTE:  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

NOTE: 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 of Section 102-2 of Form 100.

REVISIONS	DATE	BY	DESCRIPTION

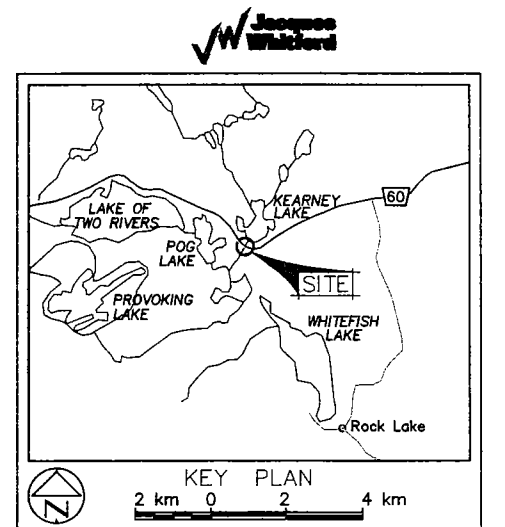
GEORES No 31F-147

HWY No 60	CHECKED	DATE 2005-06-23	DIST 43
SUBM'D PC	CHECKED	DATE 2005-06-23	SITE -
DRAWN GBB	CHECKED	APPROVED PC	DWG ONO11685-1



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

CONT No - WP No 545-93-00		 SHEET -
KEARNEY CREEK BRIDGE SITE STATION 10+800 - STATION 11+366		
BORE HOLE LOCATIONS & SOIL STRATA		



LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Test (Cone)		
	Bore Hole & 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	Elev = 395.5 m Geodetic		
	Reference: TSH profile plate A-2		
No	ELEVATION	STATION	OFFSET
05-1	395.4	10+850	C/L
05-2	395.2	10+866	C/L
05-3	395.3	10+883	C/L
05-4	396.9	10+898	3.0 Rt C/L
05-5	398.0	10+854	1.9 Rt C/L
05-6	398.2	10+865	2.0 Rt C/L
05-7	398.1	10+895	1.8 Lt C/L
05-8	398.3	10+904	1.8 Lt C/L



NOTE: The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

NOTE: 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 of Section 102-2 of Form 100.

REVISIONS	DATE	BY	DESCRIPTION
1			

GEORES No 31F-147

HWY No 60	CHECKED	DATE 2005-06-23	DIST 43
SUBMIT PC	CHECKED	APPROVED	SITE -
DRAWN GBB	CHECKED		DWG ON011685-2

## **APPENDIX B**

Symbols and Terms Used on Borehole Records

Borehole Records

Grain Size Distribution Test Results

## 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:

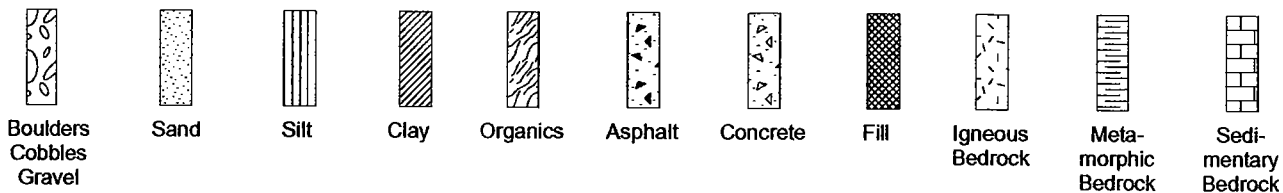
Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

### 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 10+850 C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 27.01.05 - 27.01.05 CHECKED BY PL

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			20	40	60	80	100		
392.6														
0.0	SAND, with silt, trace organics, very loose, brown		1	SS	3		392							
391.1			2	SS	7		391							
1.5	SAND, with silt, loose to compact, brown		3	SS	12		390							80 (20)
389.6			4	SS	7		389							
3.1	SAND, with silt, loose to compact, brown		5	SS	28		388							
	- becomes grey		6	SS	11		387							
			7	SS	6		386							
385.2			8	SS	11		385							
7.4	SAND, trace silt, loose, grey		9	SS	7		384							
383.6			10	SS	21		383							
9.0	SAND, with silt, loose to compact, grey						382							78 (22)
							381							
379.8							380							
12.8	End of Borehole													

MT0 11695.GPJ ON MOT.GDT 23/06/06

# RECORD OF BOREHOLE No 05-2

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+866 C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 24.01.05 - 25.01.05 CHECKED BY PC

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 $\gamma$ 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							WATER CONTENT (%)
								○ UNCONFINED	× FIELD VANE						
								● QUICK TRIAXIAL	× LAB VANE						
392.4	Grass							20 40 60 80 100	10 20 30						
0.0	Organic layer														
391.6															
0.8	SAND, trace to some silt, loose to very dense, grey to brown		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		7	SS	10										
			8	SS	32										
			9	SS	11										
			10	SS	12										
380.6															
11.8	SAND, trace silt, very dense, grey		11	SS	64										
379.1															
13.3	SILT and SAND, loose to dense, grey		12	SS	22										
			13	SS	8										
			14	SS	23										
			15	SS	30										
372.4															
20.0	SILT and SAND to SILT with sand, compact to very dense, grey		16	SS	25										

MTO 11685.GPJ ON MOT.GDT 23/06/05

Continued Next Page

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

**METRIC**

ELEV DEPTH	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 C
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
								SHEAR STRENGTH kPa ○ UNCONFINED      × FIELD VANE ● QUICK TRIAXIAL    × LAB VANE 20 40 60 80 100						

SILT and SAND to SILT with sand, compact to very dense, grey					
	17	SS	27	371	
				370	
	18	SS	41	369	o
				368	
	19	SS	44	367	
				366	o
	20	SS	36	365	
				364	
	21	SS	23	363	
				362	
	22	SS	19	361	o
				360	
	23	SS	31	359	
				358	
	24	SS	22	357	
				356	
	25	SS	18	355	G
				354	
	26	SS	30	353	
	27	SS	20		
	28	SS	54		
	29	SS	124		
352.2 40.2	End of Borehole Standpipe Installed (25 mm diameter flexible poly-tube)				

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

# RECORD OF BOREHOLE No 05-3

1 OF 3

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+883 C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 28.01.05 - 01.02.05 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/m³	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						
392.5							20	40	60	80	100					
0.0	SAND, with silt, some organic material, loose, dark brown		1	SS	4		392									
391.0							391									
1.5	SAND, trace gravel, trace silt, trace organics, very loose, brown		2	SS	3											
390.2							390									
2.3	SAND, with silt, compact to dense, brown		3	SS	32											
			4	SS	11		389								77 (23)	
			5	SS	42											
			6	SS	35		388									
							387									
386.4																
6.1	SAND and SILT, loose to dense, grey		7	SS	23		386								51 49	
							385									
			8	SS	7		384									
							383									
			9	SS	27		382									
							381									
			10	SS	36		380									
							379									
			12	SS	14		378								59 (41)	
							377									
377.3							376									
15.2	Sandy SILT to SILT some sand, compact to dense, grey		13	SS	30											
							375									
			14	SS	40		374									
							373									
			15	SS	23											
							372									
			16	SS	23										10 90	
	- sand rose up augers															

Continued Next Page

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

RECORD OF BOREHOLE No 05-3

2 OF 3

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+883 C/L ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 28.01.05 - 01.02.05 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									
	Sandy SILT to SILT, some sand, very loose to dense, grey		17	SS	3		371										
							370										
			18	SS	13		369										
							368										
			19	SS	25		367										
							366										
			20	SS	12		365										
							364										
			21	SS	24		363										
							362										
			22	SS	15		361										
							360										
			23	SS	24		359										
							358										
			24	SS	36		357										
							356										
			25	SS	26		355										
							354										
			26	SS	39		353										
							352										
			27	SS	31		351										
			28	SS	82												



MT0 11665.GPJ ON MOT.GDT 23/06/06

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

3 OF 3

METRIC

ELEV. DEPTH	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 (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100				
							SHEAR STRENGTH kPa ○ UNCONFINED      × FIELD VANE ● QUICK TRIAXIAL    × LAB VANE 20 40 60 80 100		WATER CONTENT (%) 10 20 30			

MTD 11685.GPJ ON MOT.GDT 23/06/06

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



# RECORD OF BOREHOLE No 05-4

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+898 3.0 Rt of C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 28.01.05 - 28.01.05 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							20 40 60 80 100				10 20 30					
0.0	Sand, with gravel, occasional cobbles, loose, brown (FILL)		1	SS	3		393									
392.6	Sand, some gravel, trace silt, compact, brown (FILL)		2	SS	9		392									
391.8	Sand, some silt, trace gravel, trace wood, compact to dense, brown (FILL)		3	SS	30		391									
390.5	SILT and SAND, trace gravel, compact, brown		4	SS	22		390									
3.6		5	SS	15	389										6 47 (47)	
		6	SS	13	388											
					387											
387.4	SILT and SAND, occasional sand seams, loose to compact, grey		7	SS	6		386									41 (59)
6.7			8	SS	19		385									
			9	SS	11		384									
							383									
			10	SS	16	382										
381.3	End of Borehole		11	SS	20											
12.8	End of Borehole  Standpipe Installed  (25 mm diameter flexible poly-tube)															

MTO 11685.GPJ ON\_MOT.GDT 23/06/05

# RECORD OF BOREHOLE No 05-5

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+854 1.9 Rt of C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 22.01.05 - 22.01.05 CHECKED BY DL

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						
395.5	Asphalt						20	40	60	80	100					
394.9	120 mm Asphalt		1	GS												
394.4	Sand and gravel, some silt, very dense, brown ( FILL)		2	SS	50/80mm											
394.4	Sand, some silt, trace gravel, very loose to compact, brown (FILL)		3	SS	7											
			4	SS	3											
			5	SS	2											
391.8	SAND, some silt, loose, brown		6	SS	4										1 83 (16)	
389.4			7	SS	6										87 (13)	
6.1	End of Borehole															



RECORD OF BOREHOLE No 05-7

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+895 1.8 Lt of C/L ORIGINATED BY AB  
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
DATUM Geodetic DATE 21.01.05 - 21.01.05 CHECKED BY DC

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						
395.4	Asphalt					20	40	60	80	100	10	20	30			
394.9	110 mm Asphalt		1	SS	50/ 80mm											
	Sand and gravel, some silt, brown (FILL)															
	- frozen to 1.4 m															
393.9	Sand, some silt, trace gravel, very loose to compact, brown ( FILL)		2	SS	9											
			3	SS	5											
			4	SS	2											
			5	SS	4											
391.7	SAND, trace silt, trace woody organic matter, loose to compact, brown		6	SS	12											
			7	SS	4											
			8	SS	1											
			9	SS	5											
389.4	SILT and SAND, very loose to compact, grey		10	SS	15											
			11	SS	25											
			12	SS	20											
			13	SS	18											
384.1	Sandy SILT, compact to very dense, grey		14	SS	111											
						</										

Continued Next Page

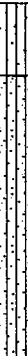
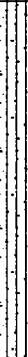
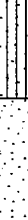
+ 3, x 3: Numbers refer to Sensitivity 3% STRAIN AT FAILURE

# RECORD OF BOREHOLE No 05-7

2 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+895 1.8 Lt of C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 21.01.05 - 21.01.05 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							
373.4	Sandy SILT, compact to very dense, grey		15	SS	21													
22.0	SAND, with silt, very dense, grey																	
				16	SS	68												
369.4																		
26.0	SILT and SAND, dense, grey																	
				17	SS	48												
			18	SS	36													
363.0																		
32.4	SAND, trace silt, very dense, grey																	
361.3			19	SS	102													
34.1	End of Borehole																	

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# RECORD OF BOREHOLE No 05-8

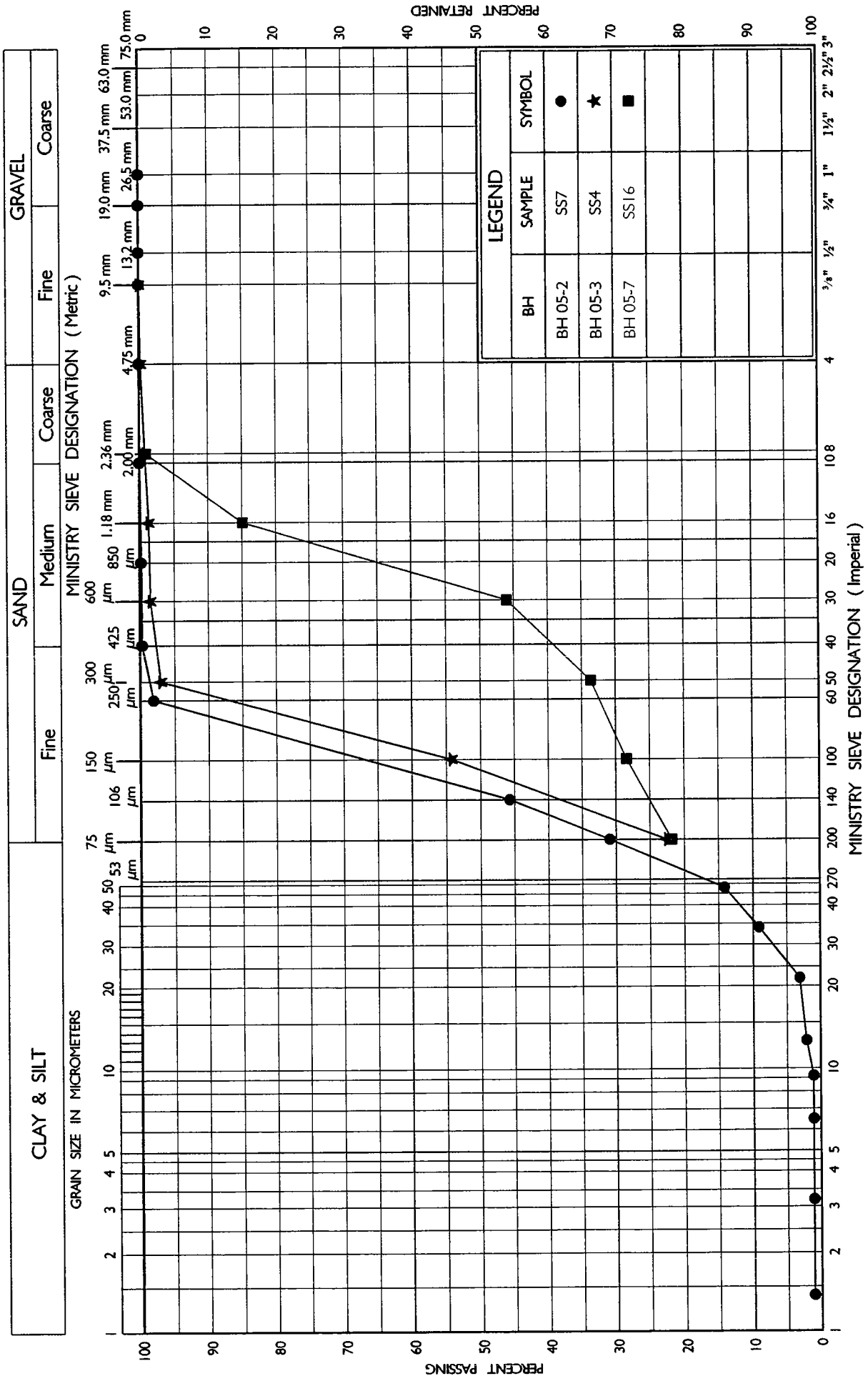
1 OF 1

METRIC

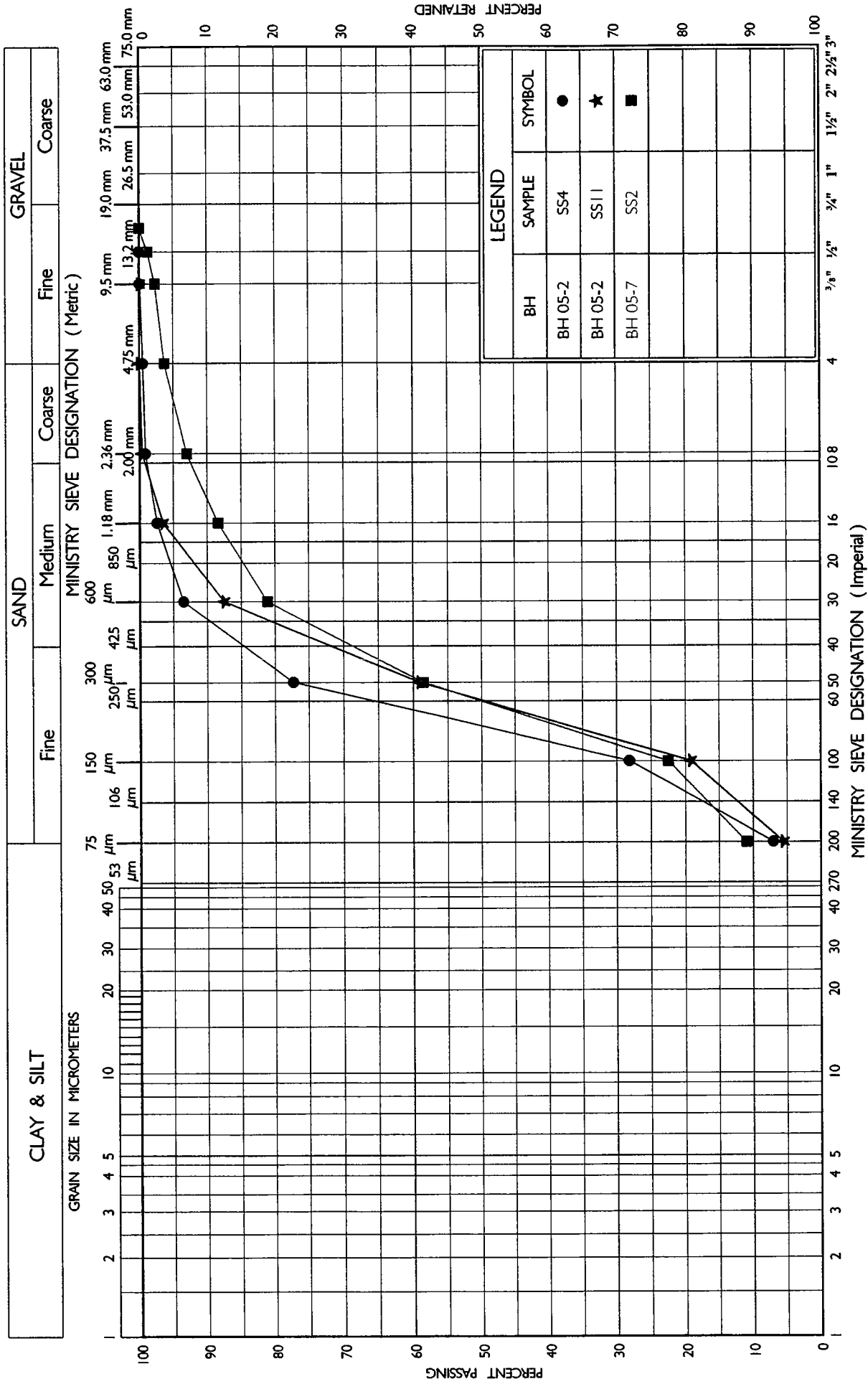
W.P. 545-93-00 LOCATION Highway 60, Kearney Bridge 10+904 1.8 Lt of C/L ORIGINATED BY AB  
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers with Split Spoons COMPILED BY JF  
 DATUM Geodetic DATE 22.01.05 - 22.01.05 CHECKED BY PC

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	× LAB VANE										
							20	40	60	80	100	10	20	30				
395.5	Asphalt														GR SA SI CL			
395.0	110 mm Asphalt		1	GS														
	Gravelly sand, some silt, very dense, brown (FILL)		2	SS	50/100mm													
394.2	- frozen to 1.3 m																	
1.3	Sand, trace silt, trace gravel, very loose to compact, brown (FILL)		3	SS	11										8 84 (8)			
			4	SS	3													
392.4																		
3.1	Sand, with silt, loose, brown to dark brown (FILL)		5	SS	2													
			6	SS	6													
391.2	Organic layer																	
391.3	Silty SAND, loose, brown		7	SS	8													
4.4																		
			8	SS	5										66 (34)			
389.4																		
6.1	End of Borehole																	

# UNIFIED SOIL CLASSIFICATION SYSTEM

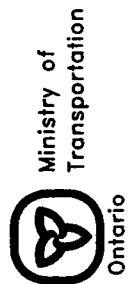


UNIFIED SOIL CLASSIFICATION SYSTEM



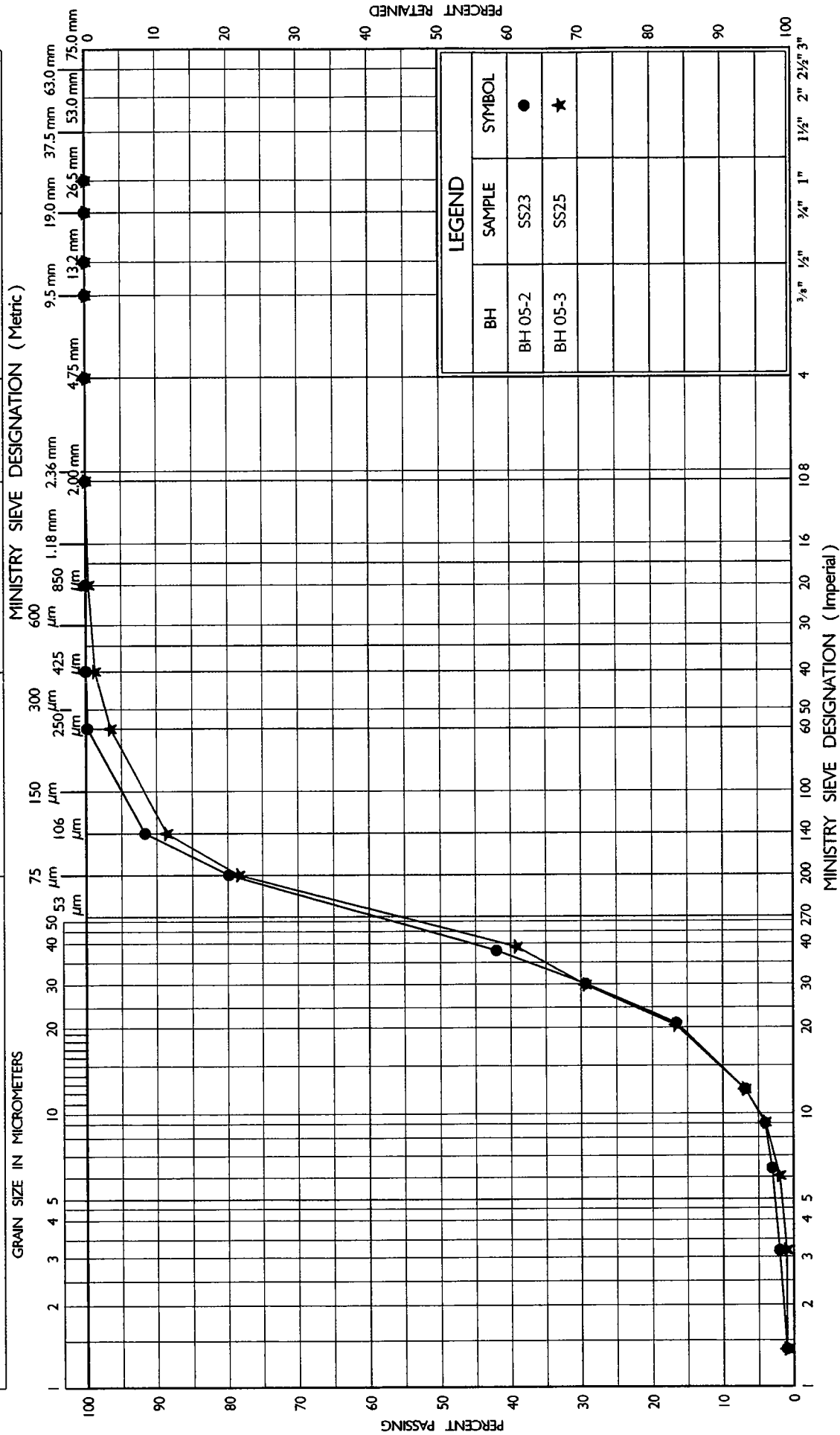
GRAIN SIZE DISTRIBUTION  
SAND, TRACE TO SOME SILT, TRACE GRAVEL

FIG No 2  
W P 543-93-00





CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



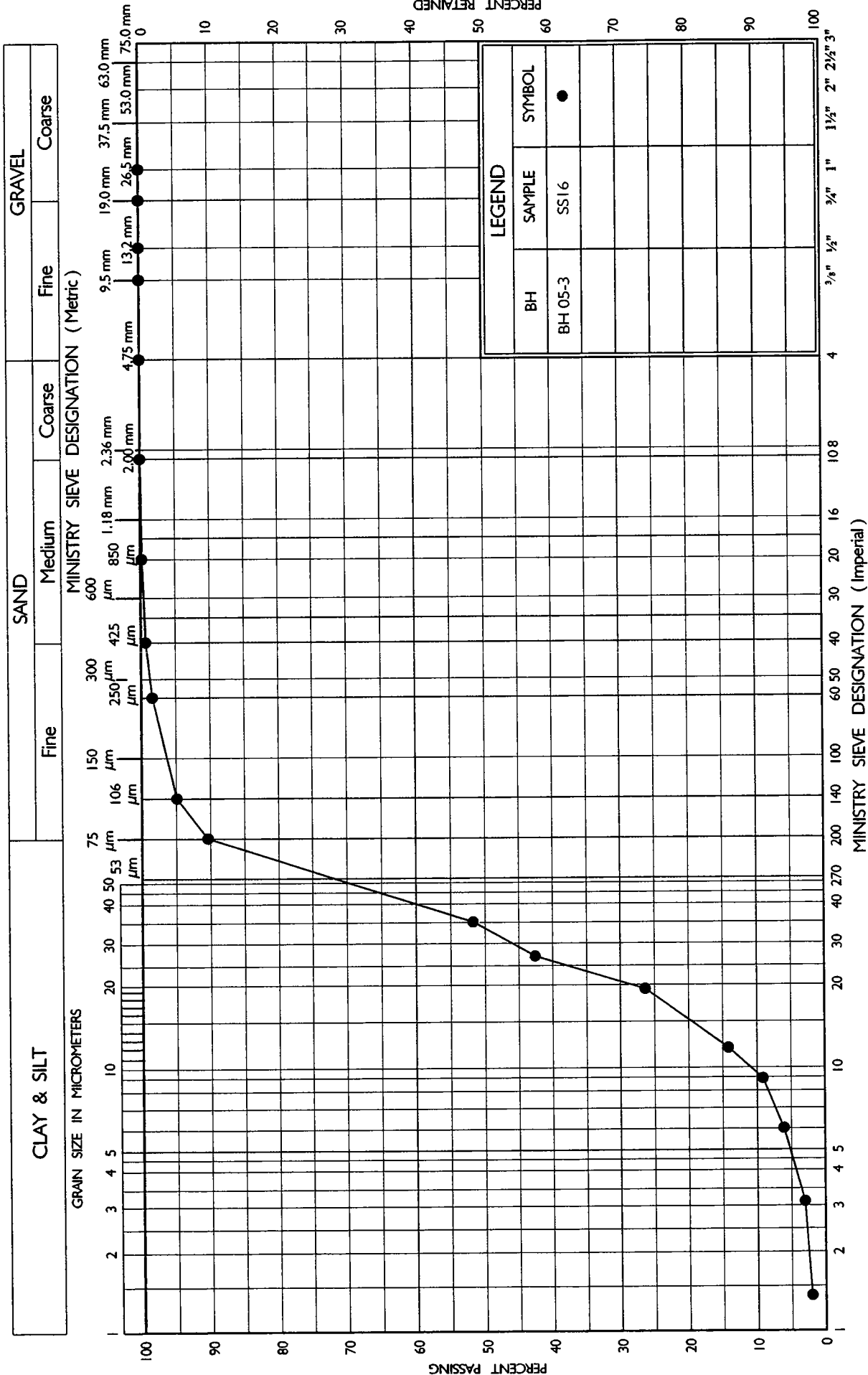
## GRAIN SIZE DISTRIBUTION

## SILT, WITH SAND

FIG No 3

WP 543-93-00

# UNIFIED SOIL CLASSIFICATION SYSTEM



PERCENT PASSING

PERCENT RETAINED

UNIFIED SOIL CLASSIFICATION SYSTEM

