



**Engineering,
Scientific,
Planning and
Management
Consultants**

2781 Lancaster Road
Suite 200
Ottawa Ontario
Canada K1B 1A7

Bus 613 738 0708
Fax 613 738 0721

www.jacqueswhitford.com

PRELIMINARY REPORT

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

Totten Sims Hubicki

PROJECT NO. ONO11685
GEOCRES NO. 31F-148

**Jacques
Whitford**

**An Environment
of Exceptional
Solutions**

Registered to
ISO 9001:2000 &
ISO 14001:1996

100% Post
Consumer
Content



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 – Clarke Creek Bridge
Replacement
County of Nipissing
District 43, Bancroft
Ministry of Transportation
Ontario
Geocres No. 31F-148**

June 2006

**Jacques Whitford
2781 Lancaster Road
Suite 200
Ottawa, Ontario
K1B 1A7**

**Phone: 613-738-0708
Fax: 613-738-0721
www.jacqueswhitford.com**



Table of Contents

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION AND GEOLOGY.....	1
3.0 PROCEDURE	2
3.1 Field Investigation	2
3.2 Survey	3
3.3 Laboratory Testing	3
4.0 SUBSURFACE CONDITIONS	3
4.1 Subsurface Profile	3
4.1.1 Fill: Sand, Trace Silt, Trace Gravel to Gravelly Sand, Trace Silt	4
4.1.2 Gravelly Sand to Sandy Gravel, Some Silt, Occasional Cobbles	4
4.1.3 Sand, Trace to Some Gravel, Trace Silt	4
4.1.4 Silty Sand / Sand with Silt.....	5
4.1.5 Silt and Sand / Silt with Sand / Silt, Some Sand	5
4.1.6 Bedrock	5
4.2 Groundwater	5
5.0 CLOSURE.....	6
6.0 DISCUSSION.....	7
6.1 Proposed Development.....	7
6.2 Soil Summary	8
6.3 Foundation Options	8
6.3.1 Replacement Structure.....	8
6.3.2 Detour Structure	9
7.0 PRELIMINARY RECOMMENDATIONS.....	10
7.1 Structure Foundations	10
7.1.1 Replacement Structure.....	10
7.1.2 Detour Structure	13
7.2 Earth Pressure Design	14
7.3 Seismic Design Considerations	15
7.3.1 Zonal Acceleration Ratio.....	15
7.3.2 Soil Profile Type	15
7.3.3 Liquefaction of Foundation Soils.....	15
7.3.4 Seismic Forces on Abutments and Retaining Walls.....	16
7.4 Embankment Design	17
7.4.1 Detour	17
7.4.2 Existing Alignment.....	18
7.5 Dewatering	18
7.6 Erosion Protection	18
7.7 Frost Protection	18
7.8 Other Construction Considerations.....	19
8.0 FUTURE INVESTIGATIONS.....	20

List of Tables

Table 6.1: Foundation Comparison for Replacement Structure	8
Table 6.2: Foundation Comparison for Detour Structure	9
Table 7.1: Recommended Pile Design Parameters for HP 310x110 Piles..	10
Table 7.2: Recommended Lateral Pile Design Parameters (Non-Cohesive Approach)	11
Table 7.3: Recommended Tensile Pile Design Parameters	13
Table 7.4: Recommended Spread Footing Design Parameters	13
Table 7.5: Recommended Lateral Earth Pressure Parameters	14
Table 7.6: Combined Coefficients of Static and Seismic Earth Pressure....	17
Table 7.7: Chemical Analysis Results	20

List of Appendices

APPENDIX A

Borehole Location Plans and Profile Plots

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Grain Size Distribution Test Results

PRELIMINARY FOUNDATION INVESTIGATION REPORT

for

W.P. 545-93-00

Highway 60 – Clarke Creek Bridge

County of Nipissing

District 43, Bancroft

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 Clarke 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.

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 provided in Appendix A. It is noted that for project orientation purposes, Highway 60 will be assumed to run north-south at the Clarke Creek Bridge, with chainage increasing from north to south.

Physiographically, the Clarke 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.

Clarke Creek flows from east to west and is approximately 13 m in width at the centreline of Highway 60. Water depths were estimated to be less than 1 m at the time of the investigation.

The existing roadway embankments are approximately 3.8 m and 5.0 m high at the north and south abutments, respectively. The water level in Clarke Creek was approximately 6 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, provided in Appendix A.

3.0 PROCEDURE

3.1 Field Investigation

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 9, 2005.

A total of eight (8) boreholes, designated as 05-9 through 05-16, were put down during the field investigation. Boreholes 05-9 through 05-12 were located beneath the proposed abutments and embankments along the proposed detour alignment. Boreholes 05-13 through 05-16 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 the augers. 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 BH 05-10, 05-12, 05-13 and 05-15. The standpipes consisted of slotted flexible poly-pipe 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 Clarke Creek bridge structure. The top of pavement at this location is identified on the existing Hwy 60 profile shown on Plate A-1, prepared by TSH, as having a geodetic elevation of 397.8 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 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.

4.0 SUBSURFACE CONDITIONS

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 detour alignment (Boreholes BH 05-9 to 05-12) consists of a fill or a thin topsoil layer, overlying sand on top of silty sand, over silt with some sand.

Within the existing roadway platform (Boreholes BH05-13 to 05-16), the subsurface profile consists of the pavement structure overlying the existing bridge approach fill, over native soils with significant particle size variations in the shallower zones, ranging from silty sand to sandy gravel, overlying silty sand and silt layers.

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. The composition of the fill ranged from sand, trace silt, trace gravel to gravelly sand, with silt. The thickness of the fill varied from 1.0 m in Borehole 05-13 to 3.6 m in Boreholes 05-14 and 05-16. The upper portion of the fill was frozen to a depth of approximately 1.2 m at the time of the investigation. The moisture content of the 5 samples of fill tested ranged from 3% to 7% and averaged 4%. The SPT 'N' values ranged from 6 to 41 (excluding the results within the upper frozen zone) with an average value of 20 indicating that the fill was generally compact. The asphalt surface overlying the fill was observed to be 90 mm to 100 mm thick at the borehole locations.

4.1.2 Gravelly Sand to Sandy Gravel, Some Silt, Occasional Cobbles

A deposit of gravelly sand, some silt, occasional cobbles to sandy gravel, some silt was observed directly beneath the fill in Boreholes 05-15 and 05-16. The thickness of this deposit ranged from 3.9 m in Borehole 05-16 to 6.0 m in Borehole 05-15. The base of the unit varied from elevation 388.7 m to 390.2 m (geodetic). SPT 'N' values ranged from 2 to 25 and averaged 13, indicating a very loose to compact state. This material ranges from an SP to GP soil using the MTO Soil Classification System.

4.1.3 Sand, Trace to Some Gravel, Trace Silt

A layer of sand with trace to some gravel, trace silt was observed in Boreholes 05-9, 05-11, 05-12 and 05-13. The thickness of this deposit, where present, ranged from 2.8 m to 5.8 m. The base of the unit varied from elevation 383.7 m to 390.2 m (geodetic). SPT 'N' values ranged from 9 to 95 and averaged 28, suggesting a generally compact state. The moisture content of the 6 samples tested ranged from 12% to 24% with an average of 22%. Grain size analysis of five samples indicated that this deposit contained 0% to 18% gravel, 79% to 92% sand and 2% to 9% 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 SP and SP-SM soil using the MTO Soil Classification System.

4.1.4 Silty Sand / Sand with Silt

A layer that ranged from silty sand to sand with silt was observed in all boreholes except Borehole 05-13. Where this deposit was fully penetrated, the thickness ranged from 4.6 m to 16.9 m with an average of 9.5 m. Borehole 05-15 was terminated within the sand with silt deposit at a depth of 33.9 m, indicating that this deposit was greater than 24.8 m at this location. The base of the unit varied from elevation 382.7 m to deeper than elevation 363.9 m (geodetic). SPT 'N' values ranged from 4 to 83 and averaged 29, suggesting a generally compact state. It is noted that loose zones several metres thick were identified in some boreholes. The moisture content of the 20 samples tested ranged from 11% to 28% with an average of 20%. Grain size analysis of the sample tested indicated that it contained 0% gravel, 77% sand and 23% 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.

4.1.5 Silt and Sand / Silt with Sand / 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 05-9, 05-10, 05-11, 05-12 and 05-14. All of these boreholes were terminated within this silty deposit at depths ranging from 15.9 m to 34.1 m. The base of this unit was not confirmed, however, it extended to at least elevation 359.7 m in Borehole 05-10. SPT 'N' values ranged from 10 to 116 and averaged 26 (excluding the three samples in which SPT refusal was achieved), suggesting a generally compact state. The moisture content of the 16 samples tested ranged from 16% to 24% with an average of 20%. Grain size analysis of three samples indicated that this deposit contained 0% gravel, 14% to 59% sand and 41% to 86% 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.

4.1.6 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 391.5 m to 392.2 m). The water level in Clarke Creek on January 20, 2005, was surveyed to be at elevation 392.2 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

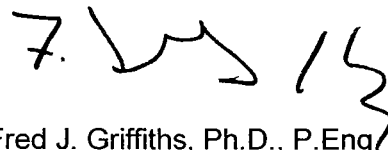
A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately 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 – Clarke Creek Bridge
County of Nipissing
District 43, Bancroft

6.0 DISCUSSION

6.1 Proposed Development

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

It is understood that the Ministry of Transportation of Ontario (MTO) plans to replace the existing Clarke Creek bridge (Site No. 43-149). Based on the Structural Planning Report, the existing structure was constructed in 1939 and consists of a 36.6 m long six span slab-on-girder structure. It has a concrete deck and steel girders supported on timber piles. 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 proposed replacement structure has been developed as a 26 m single span CPCI 1400 girder bridge with integral abutments. The proposed alignment will follow the existing vertical and horizontal highway alignment and the proposed abutments will be perpendicular to the centreline. The width of the bridge deck will remain essentially unchanged, providing 10.0 m between barriers. The base width of the proposed structure is 14 m with 2H:1V foreslopes and sideslopes. It is anticipated that the underside of the footing or pile cap would be approximately 392.9 m geodetic for both the replacement and detour structures.

Traffic management during construction of the replacement structure may require a 2-lane detour to the east side, thereby requiring a temporary detour bridge structure. The detour structure will also be a single span structure. The proposed profile for the detour indicates that finished grades will be between 395.8 m and 395.9 m at the south and north edges of the structure respectively. This represents up to 3.3 m of fill in the approaches.

6.2 Soil Summary

The native soil conditions at this site consist of a deep deposit of non-cohesive materials ranging from silt, some sand, to gravelly 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 kN/m³ and a minimum angle of internal friction of 30 degrees.

6.3 Foundation Options

6.3.1 Replacement Structure

The following table compares the available foundation options considered for this site:

Table 6.1: Foundation Comparison for Replacement Structure

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Spread Footings	<ul style="list-style-type: none">▪ moderate geotechnical resistance▪ allows for semi-integral abutment design	<ul style="list-style-type: none">▪ incompatible with integral abutment design▪ native soils easily disturbed when saturated	Low	<ul style="list-style-type: none">▪ erosion of foundation cover / loss of geotechnical resistance
Spread Footings on Structural Fill Pad	<ul style="list-style-type: none">▪ moderate geotechnical resistance but higher than spread footings on native soil▪ allows for semi-integral abutment design	<ul style="list-style-type: none">▪ requires excavation below water level▪ incompatible with integral abutment design	Low	<ul style="list-style-type: none">▪ excavation below waterline require / do work in the wet▪ erosion of foundation cover / loss of geotechnical resistance
Driven H-piles on Bedrock	<ul style="list-style-type: none">▪ readily incorporated into integral abutment design▪ high geotechnical resistance on bedrock	<ul style="list-style-type: none">▪ anticipated length of 35 m	Moderate	<ul style="list-style-type: none">▪ bedrock depth not yet confirmed by coring▪ piles reach refusal at greater depth / higher cost
Driven H-piles in Silt and Sand	<ul style="list-style-type: none">▪ readily incorporated into integral abutment design▪ moderate geotechnical resistance	<ul style="list-style-type: none">▪ anticipated length of 35 m	Moderate	
Caissons	<ul style="list-style-type: none">▪ high geotechnical resistive on bedrock▪ allows for semi-integral abutment design	<ul style="list-style-type: none">▪ require tremie concrete▪ require cased holes▪ incompatible with integral abutment design	High	<ul style="list-style-type: none">▪ base instability in saturated sands may require use of drilling mud / extra cost▪ bedrock depth not yet confirmed by coring. Caisson length may need to be extended.

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

6.3.2 Detour Structure

The following table compares the available foundation options considered for the detour structure:

Table 6.2: Foundation Comparison for Detour Structure

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Spread Footings	<ul style="list-style-type: none"> moderate geotechnical resistance 	<ul style="list-style-type: none"> native soils easily disturbed when saturated 	Low	<ul style="list-style-type: none"> erosion of foundation cover / loss of geotechnical resistance
Spread Footings on Structural Fill Pad	<ul style="list-style-type: none"> moderate geotechnical resistance but higher than spread footings on native soil 	<ul style="list-style-type: none"> may require excavation below water level 	Low	<ul style="list-style-type: none"> excavation below waterline require / do work in the wet erosion of foundation cover / loss of geotechnical resistance
Driven H-piles on Bedrock	<ul style="list-style-type: none"> high geotechnical resistance on bedrock 	<ul style="list-style-type: none"> anticipated length of 35 m 	Moderate	<ul style="list-style-type: none"> bedrock depth not yet confirmed by coring piles reach refusal at greater depth / higher cost
Driven H-piles in Silt and Sand	<ul style="list-style-type: none"> readily incorporated into integral abutment design moderate geotechnical resistance 	<ul style="list-style-type: none"> anticipated length of 35 m 	Moderate	
Caissons	<ul style="list-style-type: none"> high geotechnical resistance on bedrock 	<ul style="list-style-type: none"> require tremie concrete require cased holes 	High	<ul style="list-style-type: none"> base instability in saturated sands may require use of drilling mud / extra cost bedrock depth not yet confirmed by coring. Caisson length may need to be extended.

Given the temporary nature of the detour structure and relative cost advantage, it is recommended that the detour structure be supported on spread footings either on the native soil or on a structural fill pad, provided the available geotechnical resistance is sufficient for the design loads. If the design loads are too great to allow for an economical spread footing design, the detour structure should be founded on H-piles. It is noted that spread footings would be founded at approximate elevation 393.0 m. The creek level and groundwater levels were observed to be as high as 392.2 m at the time of the investigation. The construction of a Structural Fill Pad does risk working below the groundwater level in a highly permeable material. The consequence would be that the pad would need to be constructed in the wet using a clearstone material.

7.0 PRELIMINARY RECOMMENDATIONS

7.1 Structure Foundations

7.1.1 Replacement Structure

Axial Resistance

The replacement structure may be supported on steel H-piles. The estimated pile tip elevation is 359.0 m at both the north and south abutments.

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

Table 7.1: Recommended Pile Design Parameters for HP 310x110 Piles

Founding Material	Pile Tip Elevation (m)	Factored Axial Geotechnical Resistance at ULS (kN)	Unfactored Geotechnical Resistance at SLS (kN)
Silt and Sand	359.0	1450	1200
Bedrock	359.0	2000	1560

Note that the pile tip elevation in the above table has been estimated as approximately 5 m below the 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 at ULS 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 less than 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 SLS for piles on bedrock has been established as the load corresponding to 18 mm of elastic compression of the pile.

Downdrag forces are not anticipated at this site.

Lateral Resistance

For preliminary design purposes, 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 (Non-Cohesive Approach)

Parameter	OPSS Granular B Type I	Sand / Silty Sand	Silt / Silt and Sand
Bulk Unit Weight, kN/m ³	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:

$$k_s = n_h z/d$$

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 3,000 kN/m³ 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 11000 kN/m³ 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, Isenhower and Wang, 2006) have reported the following reduction between single piles and pile groups.

- Condition No. 1: Load is parallel to pile spacing

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

- Condition No. 2: Load is perpendicular to pile spacing

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

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

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

where

e_B = either e_T or e_L from above

α = angle between direction of loading and the skew

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

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

Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with Section 6.8.5.

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, γ	9.19 kN/m ³
Effective friction angle, ϕ	30°
Shaft Resistance Factor, β	0.5
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	359.0	1000

Pile Notes

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

7.1.2 Detour Structure

The detour structure may 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 393.0 m at both the north and south abutments.

The factored geotechnical resistance can be increased by constructing a 1000 mm thick pad of compacted OPSS Granular A or clearstone 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 beneath the waterline. 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 μ m 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 Design Parameters

Founding Layer	Footing Elev. (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Native Soil	393.0	3.0	260	150
	393.0	4.0	290	130
	393.0	5.0	315	110
	393.0	6.0	340	100
1000 mm thick Structural Fill Pad over Native Soil	393.0	3.0	350	175
	393.0	4.0	360	145
	393.0	5.0	380	125
	393.0	6.0	400	115

In accordance with Section 6.6.1 of the CHBDC, a resistance factor of 0.5 has been applied to calculate the factored geotechnical resistance at ULS. The above Factored Geotechnical Resistance at ULS incorporates the effects of a 2H:1V foreslope at the face of the footing.

The geotechnical resistance at SLS corresponds to a maximum settlement of 25 mm.

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 (P_A) and passive (P_P) 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 γ 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 III	OPSS Granular A and Granular B, Type II
Bulk Unit Weight, γ (kN/m ³)	21.2	22
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.

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 2 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 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

γ = total unit weight

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

- Zonal Acceleration Ratio, A 0.1
- Horizontal Acceleration Coefficient, k_h 0.05
- Vertical Acceleration Coefficient, k_v 0.033
- Horizontal Backslope to retaining wall
- Vertical back of wall
- For yielding abutments or walls

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

Table 7.6: Combined Coefficients of Static and Seismic Earth Pressure

Parameter	OPSS Granular B, Type I and III	OPSS Granular A and Granular B, Type II
Bulk Unit Weight, γ (kN/m ³)	21.2	22
Effective Friction Angle	32 degrees	35 degrees
Angle of Internal Friction between wall and backfill	0 degrees	0 degrees
Active Earth Pressure (K_{AE})	0.34	0.30
Height of Application of P_{AE} from base as a ratio of wall height (H)	0.349	0.350
Passive Earth Pressure (K_{PE})	3.16	3.59
Height of Application of P_{PE} from base as a 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.

7.4 Embankment Design

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

7.4.1 Detour

Embankment side slopes for the detour 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 3.3 m of fill will be required at some locations to achieve design grades at the approaches to the temporary detour structure. 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 3.3 m of fill will occur. This settlement will be complete at the completion of construction.

The construction of the roadway embankment along the proposed detour 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.

7.4.2 Existing Alignment

No significant changes to the plan or profile of the existing highway embankment are planned. Therefore no new settlement of the underlying soils is anticipated. 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.

7.5 Dewatering

It is anticipated that the excavation for the foundations of the abutments and retaining walls will extend to depths ranging from 392.0 m to 393.0 m. The water level in Clarke Creek at the time of the investigation was 392.2 m. The Draft Structural Planning Report identifies the water level as elevation 391.54 m and the high water level (100-year storm) as elevation 392.40 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.

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. Where embankment construction includes earth fill, 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 footings should be provided with the equivalent of 1.9 m of earth cover for frost protection or equivalent protection using polystyrene insulation.

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.

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 2 to 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 OPSS539 and should consider sloping backfill and traffic loading.

Cement Type and Corrosion Protection

Two 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
North Abutment Detour Alignment	05-10	SS 4	8.51	10,000 ohm cm	250 µg/g	15 µg/g
South Abutment Existing Alignment	05-15	SS 3	4.31	2,400 ohm.cm	40 µg/g	270 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 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.

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.

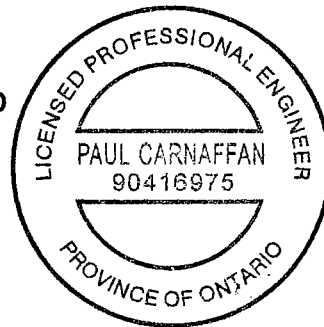
We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Yours very truly,

JACQUES WHITFORD LIMITED



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



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



P:\2005\10000\11685\Foundations\Clarke Creek\Prelim Report Clarke June 23, 2006.doc

APPENDIX A

Borehole Location Plans and Profile Plots



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

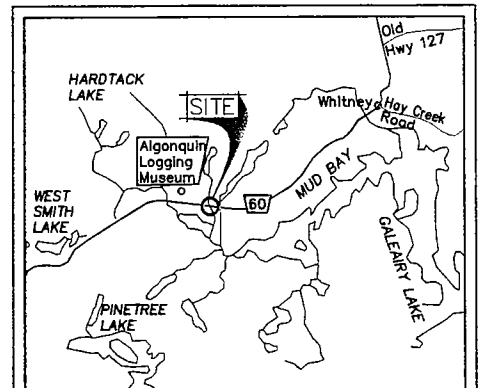
CONT No -
WP No 545-93-00

CLARKE CREEK BRIDGE SITE
STATION 9+940 - STATION 10+524
BORE HOLE LOCATIONS & SOIL STRATA

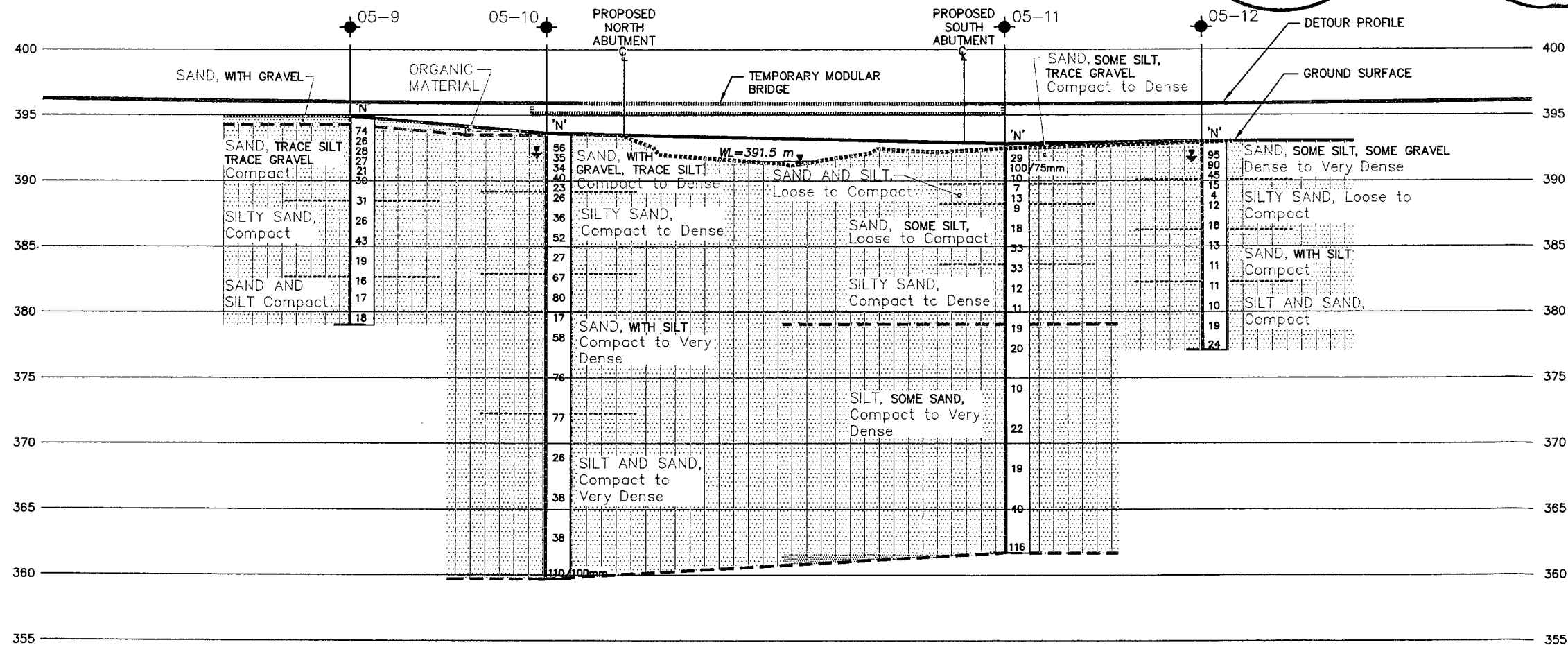
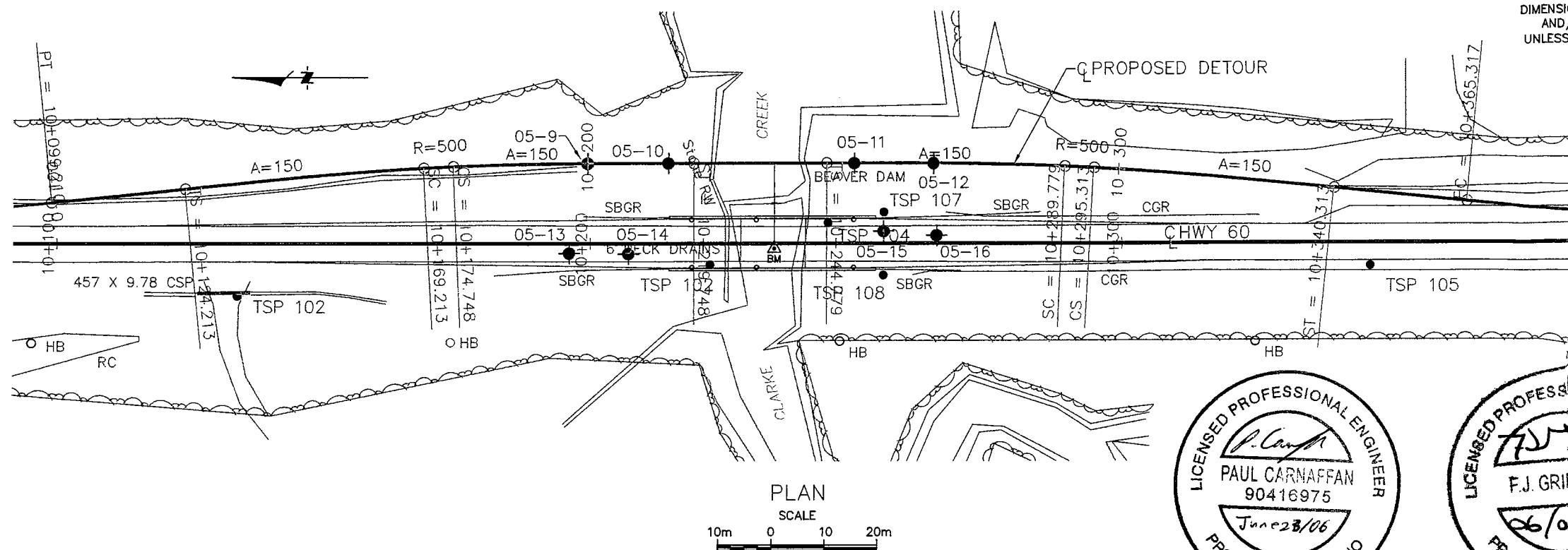


SHEET
-

Jacobs
Whitford



KEY PLAN
2 km 0 2 4 km

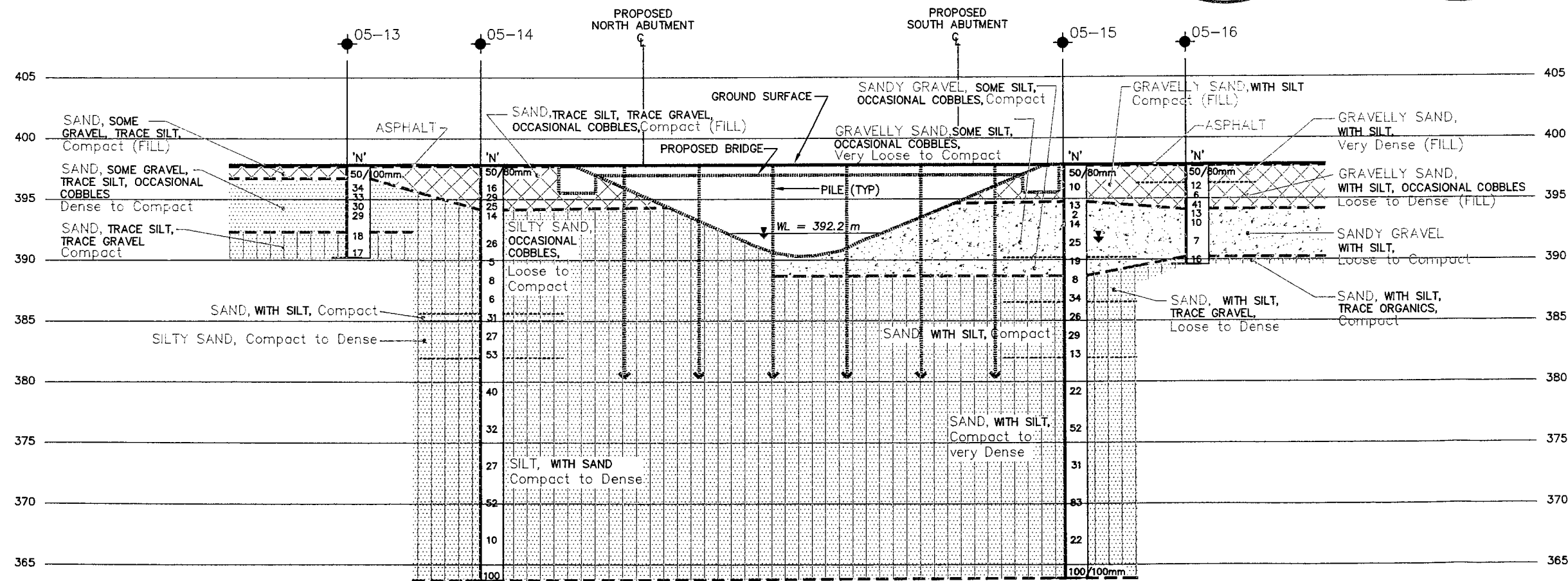
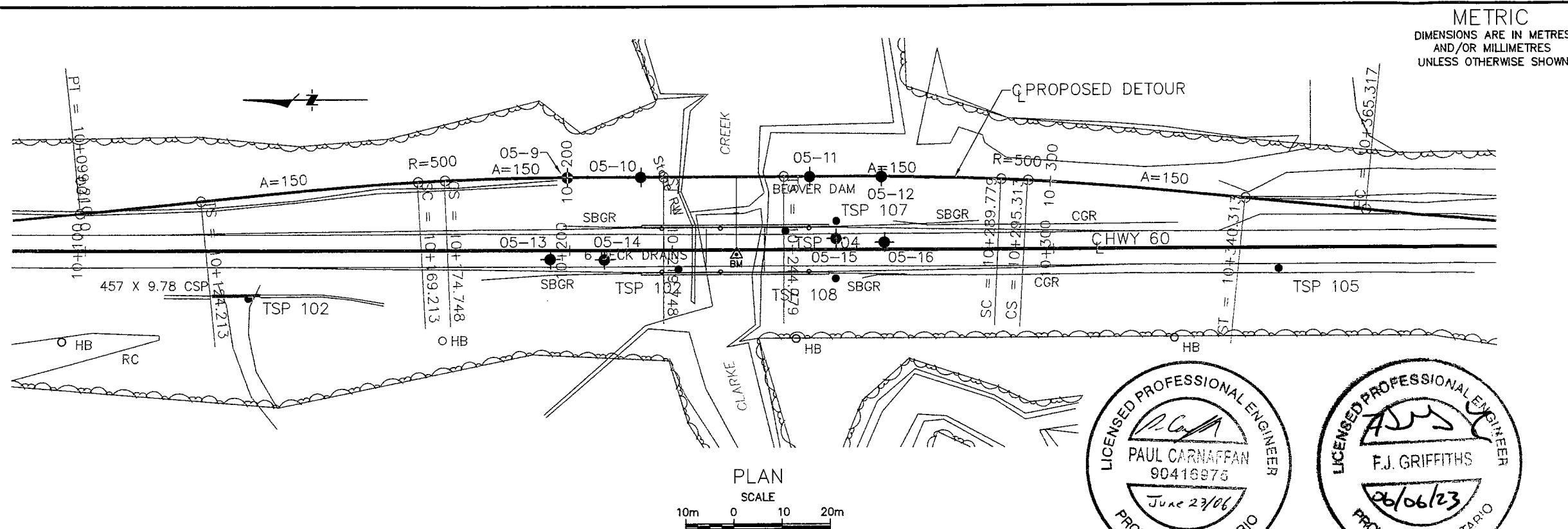


LEGEND			
●	Bore Hole		
⊕	Dynamic Cone Penetration Test (Cone)		
⊗	Bore Hole & Cone		
N	Blows/0.3m (Std Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60° Cone, 475 J/blow)		
↓	WL at time of investigation		
↓	WL in Piezometer		
△	Benchmark (Top of Pavement)		
BM	Elev = 397.8 m		
	Reference: TSH profile plate CC-1		
No	ELEVATION	STATION	OFFSET
05-9	394.9	10+200	C/L
05-10	393.6	10+215	C/L
05-11	392.8	10+250	C/L
05-12	393.0	10+265	C/L
05-13	397.8	10+196	1.8 Rt C/L
05-14	397.8	10+207	1.9 Rt C/L
05-15	397.8	10+255	2.3 Lt C/L
05-16	397.8	10+265	1.5 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			
2			
3			
4			
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			
16			
17			
18			
19			
20			
21			
22			
23			
24			
25			
26			
27			
28			
29			
30			
31			
32			
33			
34			
35			
36			
37			
38			
39			
40			
41			
42			
43			
44			
45			
46			
47			
48			
49			
50			
51			
52			
53			
54			
55			
56			
57			
58			
59			
60			
61			
62			
63			
64			
65			
66			
67			
68			
69			
70			
71			
72			
73			
74			
75			
76			
77			
78			
79			
80			
81			
82			
83			
84			
85			
86			
87			
88			
89			
90			
91			
92			
93			
94			
95			
96			
97			
98			
99			
100			

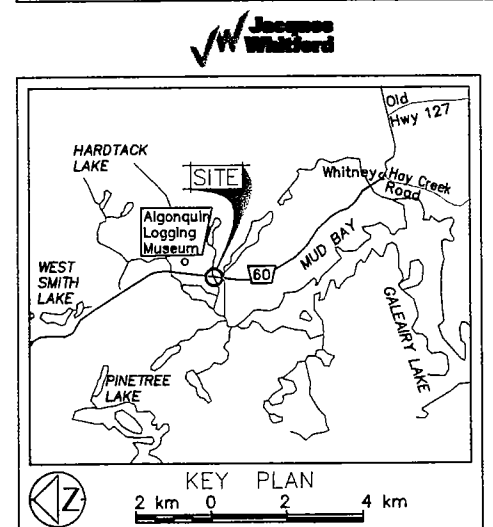


CONT No -
WP No 545-93-00

CLARKE CREEK BRIDGE SITE
STATION 9+940 - STATION 10+524
BORE HOLE LOCATIONS & SOIL STRATA



SHEET
-



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
- △ BM Benchmark (Top of Pavement)
Elev = 397.8 m
Reference: TSH profile plate CC-1

No	ELEVATION	STATION	OFFSET
05-9	394.9	10+200	C/L
05-10	393.6	10+215	C/L
05-11	392.8	10+250	C/L
05-12	393.0	10+265	C/L
05-13	397.8	10+196	1.8 Rt C/L
05-14	397.8	10+207	1.9 Rt C/L
05-15	397.8	10+255	2.3 Lt C/L
05-16	397.8	10+265	1.5 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-148

HWY No 60	CHECKED	DATE 2005-06-23	DIST 43
SUBWD 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:

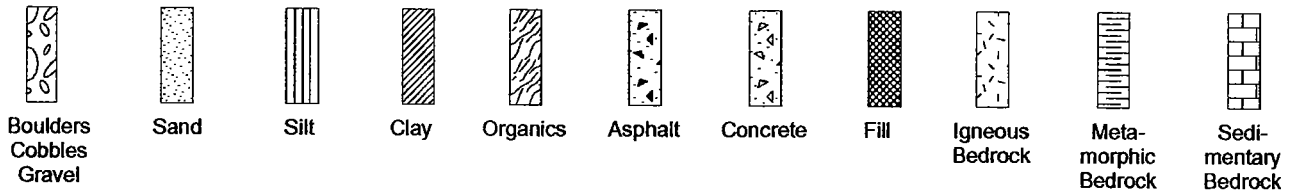
Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

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

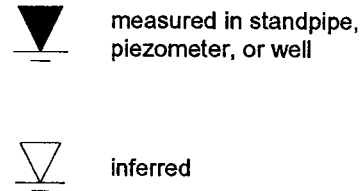
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



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
γ	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 05-9

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+200 C/L ORIGINATED BY AB
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF
 DATUM Geodetic DATE 04.02.05 - 04.02.05 CHECKED BY DC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20 40 60 80 100										
								○ UNCONFINED	×	FIELD VANE	×	LAB VANE						
394.9	Sand with gravel																	
0.0	SAND, with gravel, brown																	
394.3																		
0.6	SAND, trace silt, trace gravel, compact, brown		1	SS	74		394											
			2	SS	26		393											
			3	SS	28		392											
			4	SS	27		391											
			5	SS	21		390											
			6	SS	30		389											
388.5			7	SS	31		388											
6.4	SILTY SAND, compact to dense, brown to grey		8	SS	26		387											
			9	SS	43		386											
			10	SS	19		385											
382.7			11	SS	16		384											
12.2	SAND AND SILT, compact, grey		12	SS	17		383											
			13	SS	18		382											
379.1							381											
15.9	End of Borehole						380											

MTD 11885.GPJ ON MOT.GDT 23/06/06

RECORD OF BOREHOLE No 05-10

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+215 C/L ORIGINATED BY AB
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF
 DATUM Geodetic DATE 04.02.05 - 04.02.05 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
393.6	Grass						20	40	60	80	100				
393.6	Organic material SAND, with gravel, trace silt, occasional cobbles, compact to dense, brown		1	SS	56										
			2	SS	35										
			3	SS	34										
			4	SS	40										
389.2			5	SS	23										
4.4	SILTY SAND, brown, compact to dense		6	SS	26										
			7	SS	36										
			8	SS	52										
			9	SS	27										
382.9			10	SS	67										
10.7	SAND, with silt, compact to very dense, grey to brown		11	SS	80										
			12	SS	17										
			13	SS	58										
			14	SS	76										

Continued Next Page

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

2 OF 2

METRIC

DATUM Geodetic DATE 04.02.05 - 04.02.05 CHECKED BY pc

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

RECORD OF BOREHOLE No 05-11

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+250, C/L ORIGINATED BY AB
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF
 DATUM Geodetic DATE 08.02.05 - 08.02.05 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
392.8	Grass																
390.8	150 mm organic material																
390.2	SAND, trace silt, trace gravel, compact to dense, brown		1	SS	29												
			2	SS	100/ 75mm												
			3	SS	10												
389.8																	
3.1	SAND AND SILT, loose to compact, grey		4	SS	7												
			5	SS	13												
388.2																	
4.6	SAND, trace silt, loose to dense, brown		6	SS	9									0 91 9			
			7	SS	18												
			8	SS	33									0 92 9			
383.7																	
9.1	SILTY SAND, compact to dense, grey to brown		9	SS	33												
			10	SS	12												
			11	SS	11												
379.1																	
13.7	SILT, some sand, compact to very dense, grey		12	SS	19												
			13	SS	20												
			14	SS	10												

Continued Next Page

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

RECORD OF BOREHOLE No 05-11

2 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+250, C/L ORIGINATED BY AB
 DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF
 DATUM Geodetic DATE 08.02.05 - 08.02.05 CHECKED BY PL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)		
							○ UNCONFINED × FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100									10 20 30		
	SILT, some sand, compact to very dense, grey		15	SS	22		371									0 14 86			
							370												
							369												
				16	SS	19		368											
								367											
								366											
				17	SS	40		365											
								364											
							363												
361.7			18	SS	116		362												
31.1	End of Borehole																		

MT0 11685.GPJ ON MOT.GDT 23/06/06

RECORD OF BOREHOLE No 05-12

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+265 C/L ORIGINATED BY AB
DIST Bancroft HWY 60 BOREHOLE TYPE Hollow Stem Augers/Casing with Split Spoons COMPILED BY JF
DATUM Geodetic DATE 09.02.05 - 09.02.05 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	× FIELD VANE	× LAB VANE						
393.0	Grass						20	40	60	80	100					
392.9	Organic material															
	SAND, some silt, some gravel, dense to very dense, brown		1	SS	95											
			2	SS	90											
390.1			3	SS	45											
2.9	SILTY SAND, loose to compact, grey to brown		4	SS	15											
			5	SS	4											
			6	SS	12											
386.3			7	SS	18											
6.7	SAND, with silt, compact, grey															
			8	SS	13											
			9	SS	11											
382.3																
10.7	SILT AND SAND, compact, grey		10	SS	11											
			11	SS	10											
			12	SS	19											
377.2			13	SS	24											
15.9	End of Borehole Standpipe Installed (25 mm diameter flexible poly-tube)															

MT0 11685.GPJ ON MOT.GDT 23/06/06




RECORD OF BOREHOLE No 05-13

1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+196 1.8 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 23.01.05 - 23.01.05 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									

397.8	Asphalt															
398.9	90 mm Asphalt		1	GS												
	Sand, some gravel, trace silt, compact, brown (FILL)		2	SS	50/100mm											
396.7																
1.1	SAND, some gravel, trace silt, occasional cobbles, compact to dense, brown		3	SS	34											
			4	SS	33											
			5	SS	30											
			6	SS	29											
392.3																
5.5	SAND, trace silt, trace gravel, compact, brown		7	SS	18											
390.2			8	SS	17											
7.6	End of Borehole Standpipe Installed (25 mm diameter flexible poly-tube)															

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

RECORD OF BOREHOLE No 05-14

1 OF 2

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+207 1.9 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 23.01.05 - 23.01.05 CHECKED BY DC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	× FIELD VANE × LAB VANE						
397.8	Asphalt						20 40 60 80 100								
397.9	100 mm Asphalt		1	GS											
	Sand, trace silt, trace gravel, occasional cobbles, compact, brown (FILL)		2	SS	50/ 80mm										
			3	SS	16										
			4	SS	29										
			5	SS	25										
394.1			6	SS	14										
3.7	SILTY SAND, occasional cobbles, loose to compact, brown to grey		7	SS	26										
			8	SS	5										
			9	SS	8										
			10	SS	6										
385.6			11	SS	31										
12.2	SAND, with silt, compact, grey		12	SS	27										
385.0			13	SS	53										
12.8	SILTY SAND, compact to dense, brown to grey		14	SS	40										
382.0															
15.9	SILT, with sand, compact to dense, grey														

Continued Next Page

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

METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)				
								20							40	60	80	100
	SILT, with sand, compact to dense, grey		15	SS	32													
						376												
						375												
						374												
			16	SS	27													
						373												
						372												
						371												
			17	SS	52													
						370							0 26 74					
						369												
						368												
			18	SS	10													
						367												
						366												
						365												
363.7			19	SS	100													
34.1	End of Borehole					364												

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

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

1 OF 2

METRIC

[illegible]

Continued Next Page

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

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

2 OF 2

METRIC

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

RECORD OF BOREHOLE No 05-16

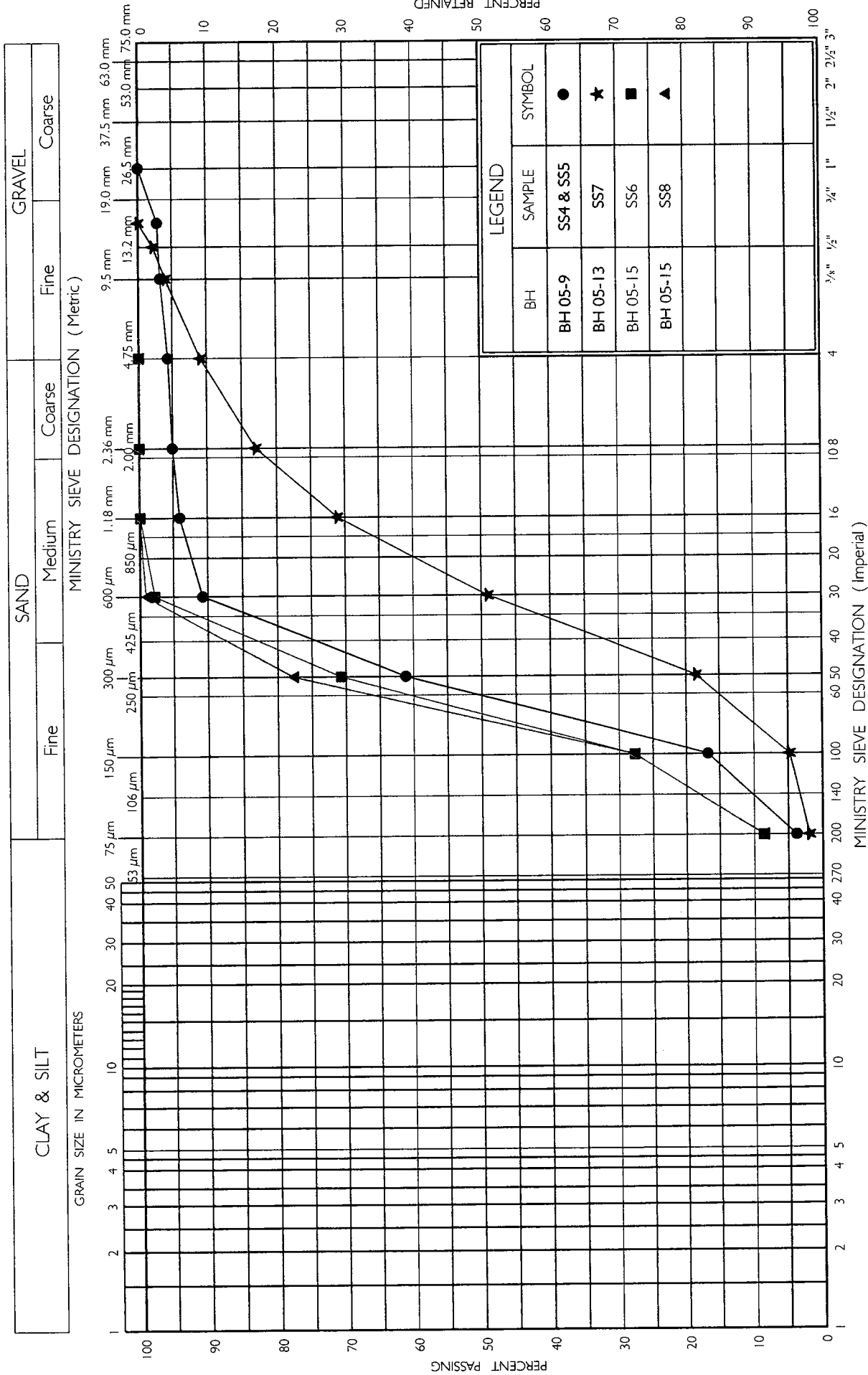
1 OF 1

METRIC

W.P. 545-93-00 LOCATION Highway 60, Clarke Bridge 10+265 1.5 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 - 21.01.05 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	× FIELD VANE	● QUICK TRIAXIAL	× LAB VANE									
397.8	Asphalt					20	40	60	80	100	10	20	30							
397.9	100 mm Asphalt		1	GS																
398.0	Gravelly sand, with silt, very dense, brown (FILL)		2	SS	50/80mm															
396.3	Gravelly sand, with silt, occasional cobbles, loose to dense, brown (FILL)		3	SS	12															
			4	SS	6															
394.1			5	SS	41															
394.1	SANDY GRAVEL, with silt, loose to compact, brown		6	SS	13															
			7	SS	10															
			8	SS	7															
390.2																				
389.6	SAND, with silt, trace organics, compact, grey		9	SS	16															
389.6	End of Borehole																			

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

SAND, TRACE SILT, TRACE GRAVEL

FIG No 1

WP 545-93-00

UNIFIED SOIL CLASSIFICATION SYSTEM

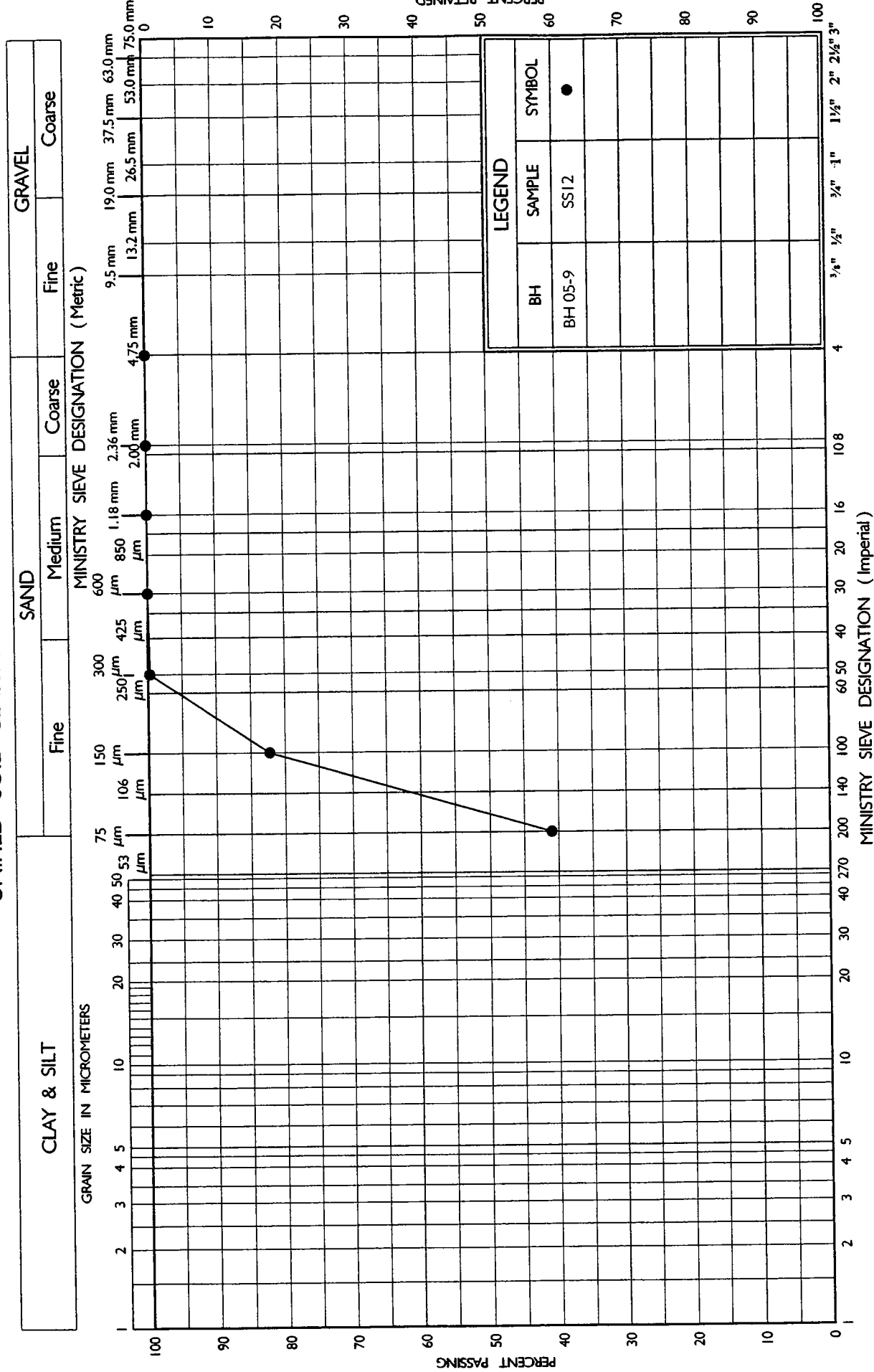


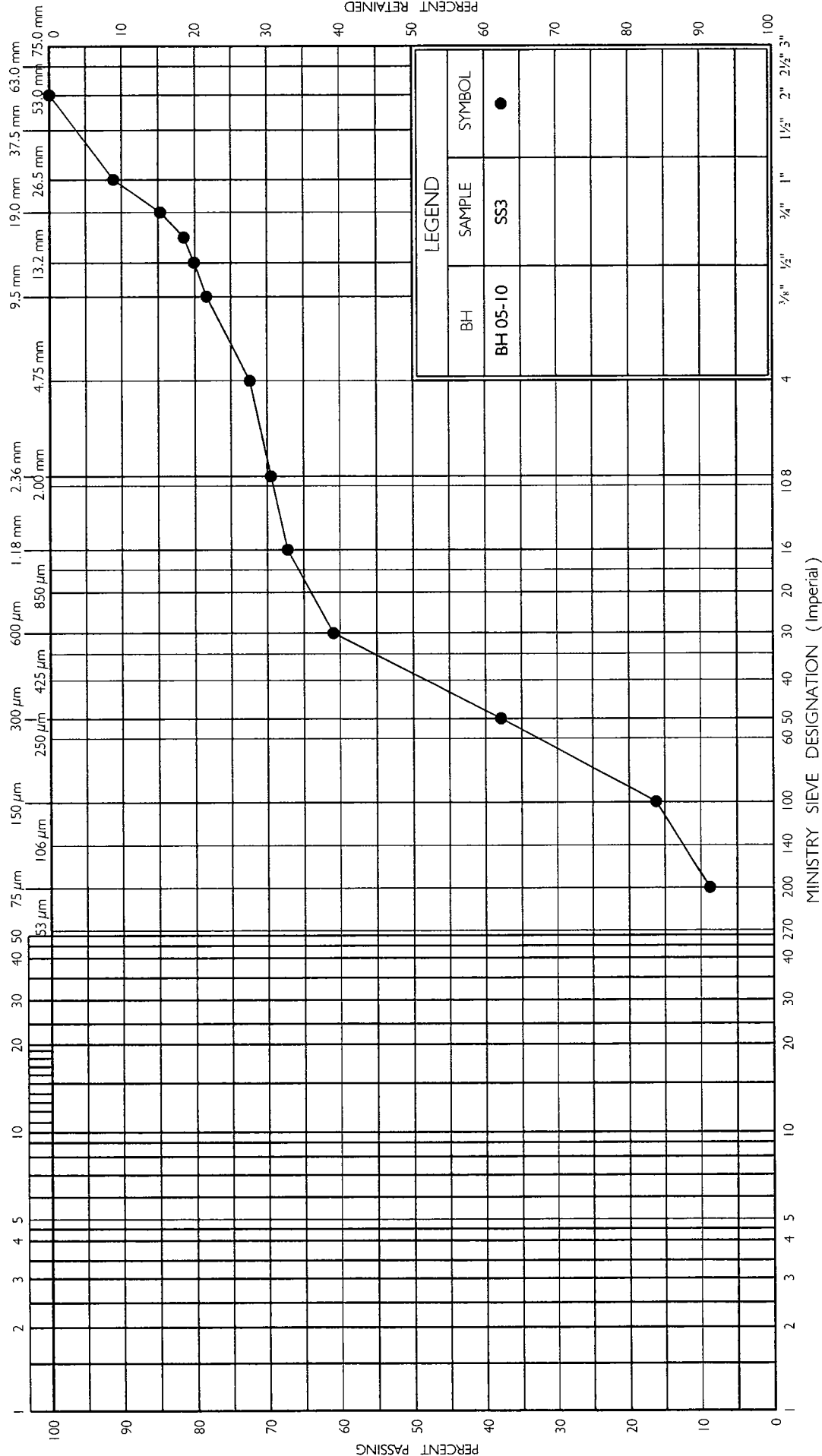
FIG No 2
GRAIN SIZE DISTRIBUTION
SAND AND SILT

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT				SAND			GRAVEL		
				Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

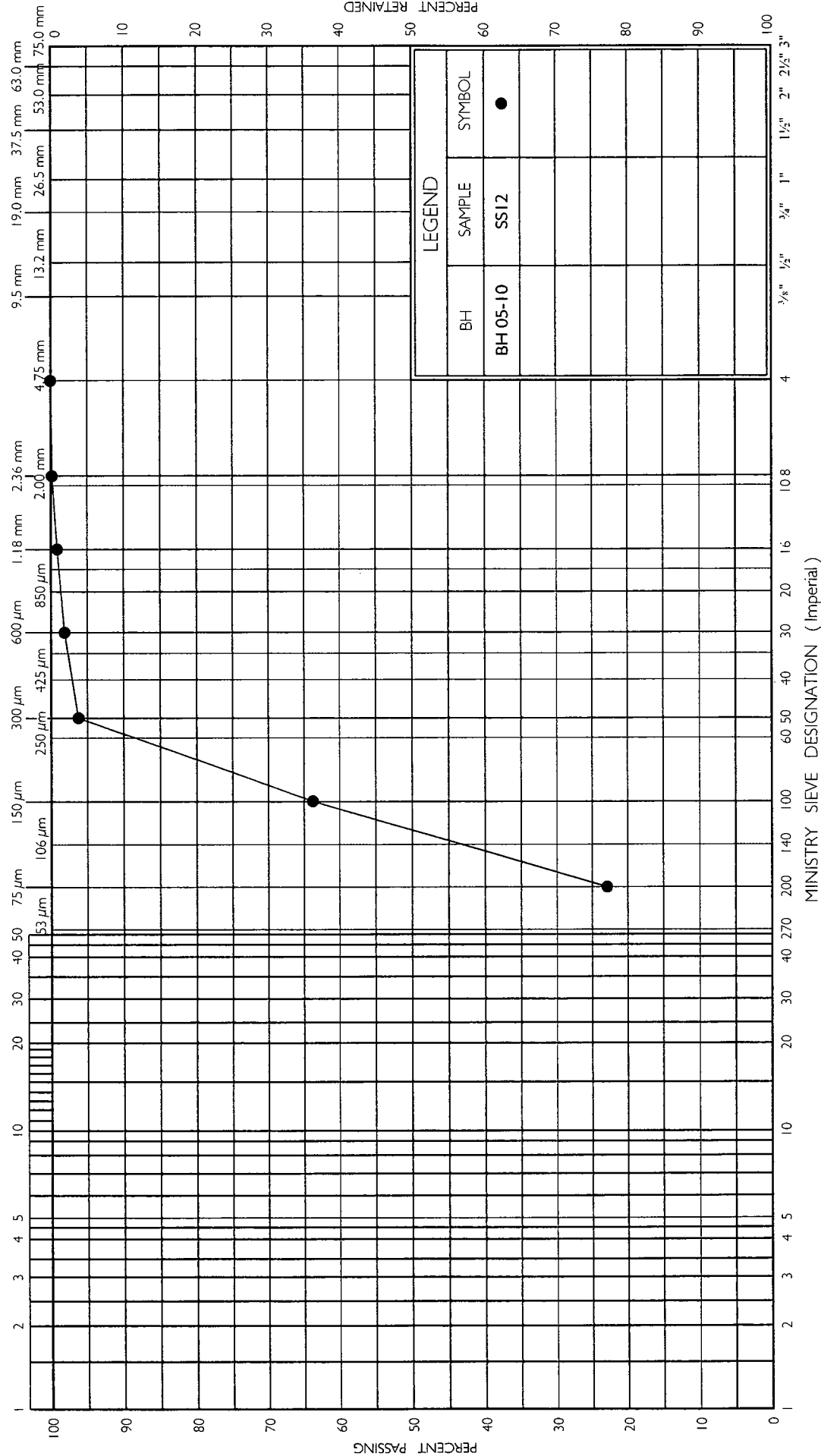
MINISTRY SIEVE DESIGNATION (Metric)



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	

MINISTRY SIEVE DESIGNATION (Metric)



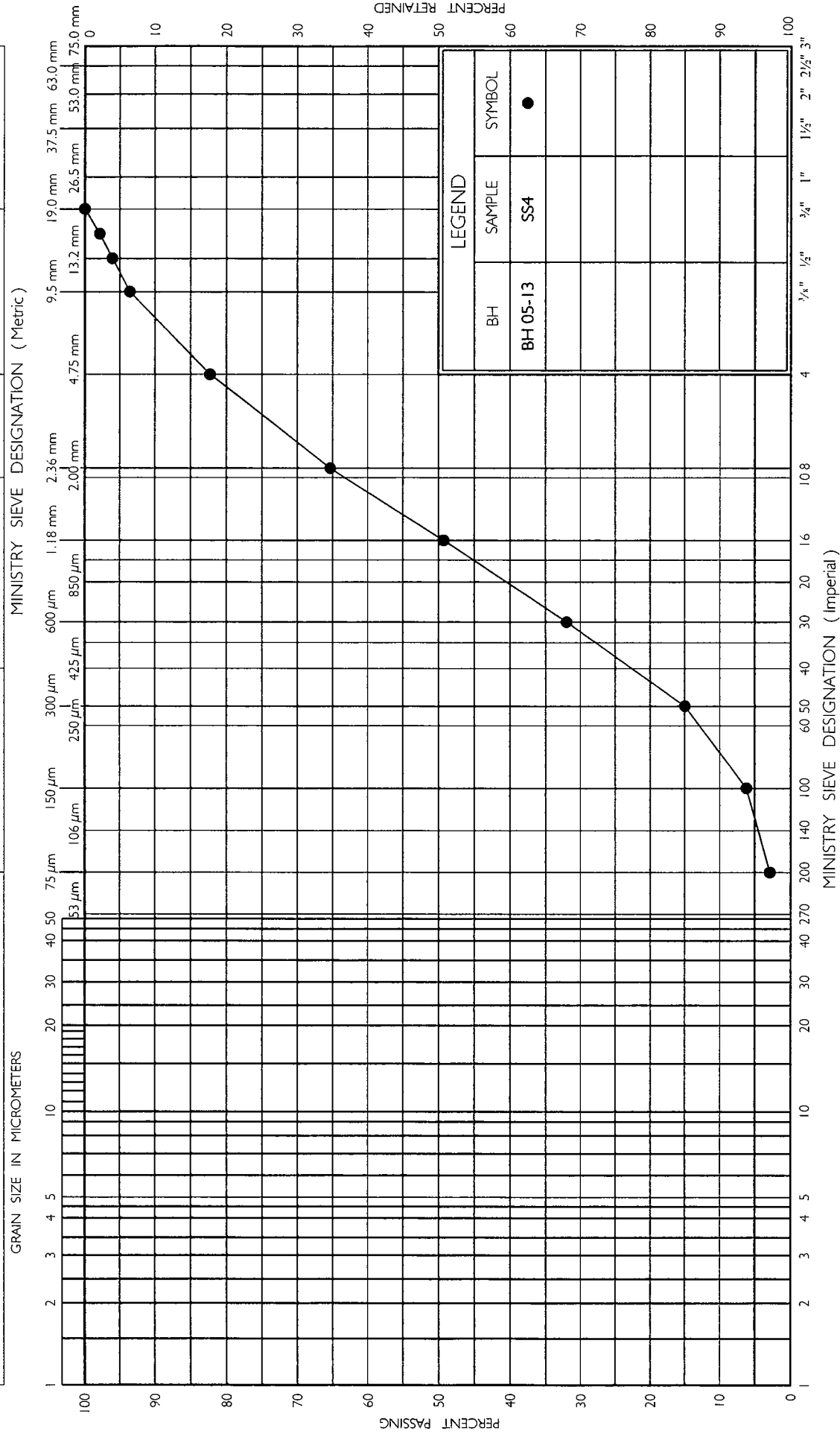
GRAIN SIZE DISTRIBUTION

SAND WITH SILT

FIG No 4

W P 545-93-00

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



Ontario
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SAND, SOME GRAVEL, TRACE SILT

FIG No 5

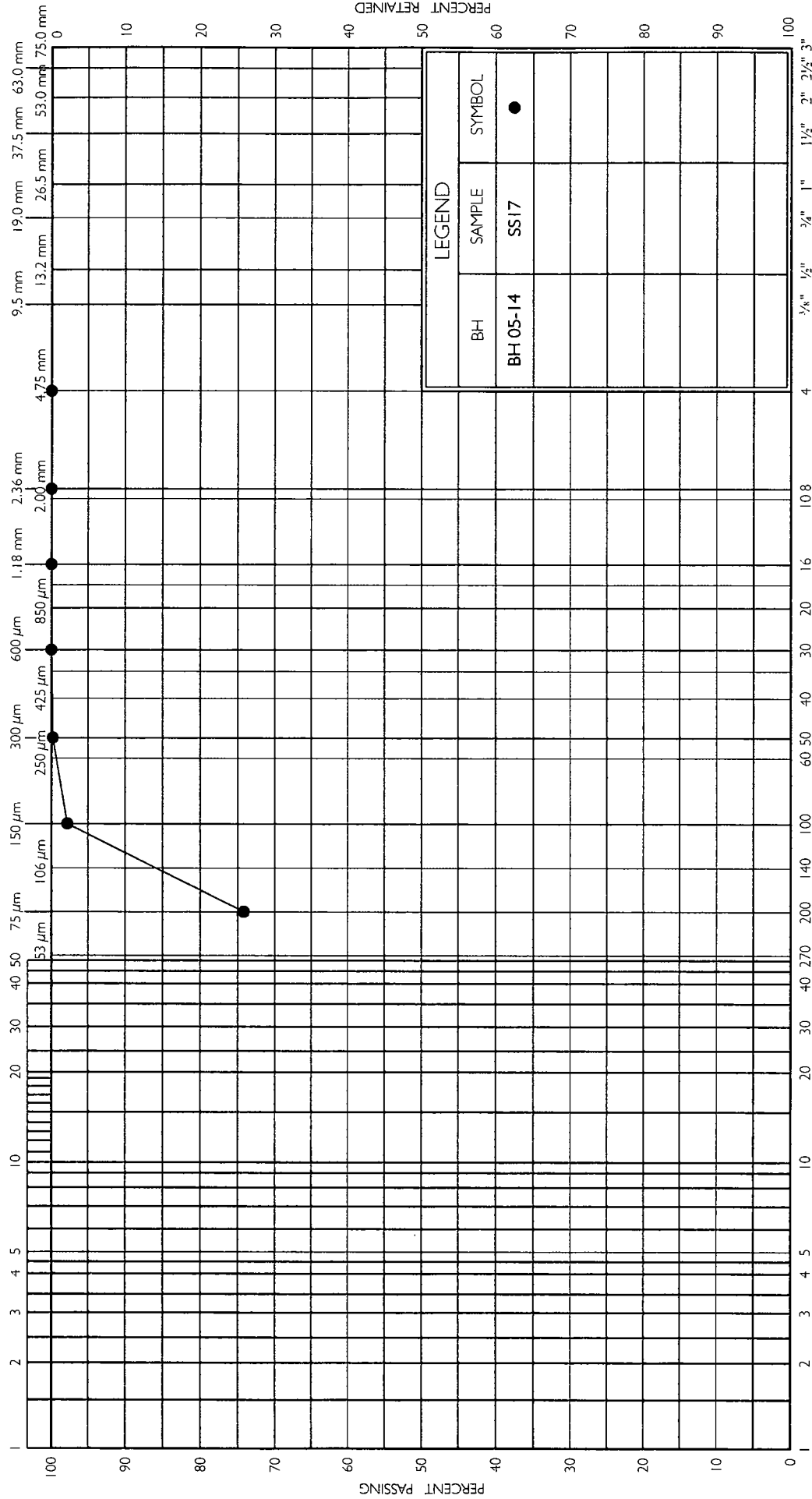
WP 545-93-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILT WITH SAND

FIG No 6

WP 545-93-00