

**Environmental
Engineering
Scientific
Management
Consultants**

Suite 200
2781 Lancaster Road
Ottawa ON
Canada K1B 1A7

Bus 613 738 0708
Fax 613 738 0721

www.jacqueswhitford.com



**Jacques
Whitford**

**An Environment
of Exceptional
Solutions**

Registered to
ISO 9001:2000
ISO 14001:2004



FINAL REPORT

Foundation Investigation and
Design
WBL Retaining Wall
Highway 417/Moodie Drive
Interchange – Ottawa Area
G.W.P. 302-89-00

Totten Sims Hubicki

**REPORT NO. NO11674
Geocres # 31G5-210**

REPORT NO. NO11674

TO

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

ON

**Foundation Investigation and Design
WBL Retaining Wall
Highway 417/Moodie Drive Interchange
G.W.P. 302-89-00

Ottawa Area
Geocres #31G5-210**

March 28, 2007

**Jacques Whitford
2781 Lancaster Road, Suite 200
Ottawa, ON
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 INVESTIGATION PROCEDURES.....	2
3.1 Field Program.....	2
3.2 Survey.....	3
3.3 Laboratory Testing	3
4.0 SUBSURFACE CONDITIONS	4
4.1 Topsoil/Rootmat	4
4.2 Silty Sand to Silty Clay (Fill).....	4
4.3 Silty Clay/Clay	4
4.4 Silty Sand (Till)	5
4.5 Bedrock.....	5
4.6 Groundwater	6
5.0 CLOSURE.....	7
6.0 DISCUSSION	8
6.1 Proposed Development.....	8
6.2 Foundation Assessment.....	9
6.3 Retaining Wall and Foundation Options.....	9
7.0 RECOMMENDATIONS	11
7.1 Structure Foundations	11
7.1.1 Shallow Foundations – Bearing Resistances.....	11
7.1.2 Subgrade Improvement	12
7.1.3 Frost Protection	12
7.2 Earth Pressure Design	12
7.3 Seismic Design Considerations	13
7.3.1 Zonal Acceleration Ratio.....	13
7.3.2 Soil Profile Type	13
7.3.3 Liquefaction of Foundation Soils.....	13
7.3.4 Seismic Forces on Abutments and Retaining Walls.....	14
7.4 Global Stability	15
8.0 CONSTRUCTION RECOMMENDATIONS.....	15
8.1 Open Cut Excavations.....	15
8.2 Site Preparation.....	16
8.3 Dewatering.....	16
8.4 Cement Type and Corrosion Potential.....	16
9.0 CLOSURE.....	17

List of Tables

Table 4.1: Bedrock Elevations.....	5
Table 4.2: Summary of Bedrock Unconfined Compressive Strength	6
Table 4.3: Summary of Groundwater Levels	6
Table 6.1: Comparison of Foundation Options	10
Table 8.1: Chemical Test Results.....	16

List of Appendices

APPENDIX A Symbols and Terms Used on Borehole and Test Pit Records Borehole Records Laboratory Test Results Rock Core Summary Table

APPENDIX B Borehole Locations and Strata
--

FOUNDATION INVESTIGATION REPORT
For
G.W.P. 302-89-00
WBL Retaining Wall
Highway 417 / Moodie Drive Interchange
Ottawa Area

1.0 INTRODUCTION

This report presents the results of a foundation investigation carried out for the proposed construction of a new retaining wall along the Highway 417 West Bound Lane (WBL) foreslope at the Moodie Drive Interchange in Ottawa, Ontario. The proposed construction is required to accommodate the Highway 417 widening to eight lanes from Highway 416 westerly to 0.5 km west of Eagleson Road.

The foundation investigation was carried out in general accordance with Jacques Whitford Scope Change Request # 6 dated March 24, 2006. Authorization to proceed was provided by the Ministry of Transportation of Ontario (MTO) under agreement Number 4005-A-000260 with Totten Sims Hubicki Limited (TSH), the Prime Consultant for this project.

This report has been prepared specifically and solely for the project described herein. It contains the factual information obtained from the field and laboratory investigations.

2.0 SITE DESCRIPTION AND GEOLOGY

The project site is located on Highway 417 WBL at the Moodie Drive Interchange within the City of Ottawa. The site location is shown on the Key Plan inset on Drawing No. NO11674-1 in Appendix B.

The project site lies within the physiographic region identified by Chapman and Putnam as the Ottawa Valley Clay Flats. Ontario Geological Survey Map P.2715 "Physiography of Southern Ontario" indicates that this area consists of clay plains interrupted by ridges of rock or sand.

Drainage is generally towards roadside ditches and the grassed centre median.

The Moodie Drive structure is a three-span concrete bridge structure. The bridge deck is approximately 5 m above the Highway 417 profile grade. The embankment slopes are graded at approximately 2H:1V. Concrete slope paving covers the face of the slope directly beneath the bridge structure.

The concrete slope paving exhibits signs of cracking and subsidence. The remainder of the approach embankment is vegetated with grass and small brush and does not exhibit any obvious signs of erosion or instability.

3.0 INVESTIGATION PROCEDURES

3.1 Field Program

The fieldwork for this investigation was carried out at night between November 22nd and 24th, 2006. The subsurface conditions were investigated through a borehole drilling program. A total of four (4) boreholes, numbered BH 06-101 through BH 06-104, were advanced at select locations. Boreholes BH 06-101 and BH04-4 were advanced to auger refusal. Boreholes BH 06-102 and BH 06-103 were advanced approximately 3 m below the bedrock surface by coring.

All boreholes were drilled using a truck-mounted CME 55 power auger drill equipped for soil and bedrock sampling. The drill was owned and operated by George Downing Estate Drilling Ltd. Hollow stem auger equipment was used to advance the boreholes in the overburden. Soil samples were generally retrieved at 0.75 m intervals by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). The SPT carried out with the drilling equipment was performed using a standard 64 kg hammer with a 760 mm drop. Boreholes BH 06-102 and BH 06-103 were advanced a minimum of 2.9 m into bedrock by coring with NQ-sized coring equipment. In-situ vane shear testing was carried out using an MTO size vane to determine the undrained shear strength of cohesive soils.

The automatic trip hammer broke during drilling of BH 06-104. In order to complete the borehole before re-opening the on-ramp to rush hour traffic, the remaining split spoon samples were collected by hydraulically pushing the sampler. Therefore, N-values were not obtained for the last four samples in this borehole. The hammer was fixed prior to continuing drilling of the other boreholes the following night.

Standpipes were installed in all four boreholes. The standpipes consisted of 32 mm diameter PVC well pipe screened over the bottom 3 m. Clean sand was used to backfill around the screened interval. Above the screened portion of the standpipe, all boreholes were backfilled with a bentonite-cement mixture. Groundwater levels were measured in the standpipes on February 22, 2007. The standpipes should be decommissioned prior to construction of the proposed retaining wall by removing them from within the depth of excavation and filling the remainder of the standpipe with a non-shrink grout having a minimum unconfined compressive strength of at least 20 MPa.

The subsurface conditions are described in detail on the Borehole Records presented in Appendix A. All soil samples recovered were identified in the field, stored in moisture proof containers and were returned to our laboratory for detailed classification and testing.

3.2 Survey

Borehole locations were established in the field by Jacques Whitford personnel relative to existing site features such as the highway shoulder and existing bridge abutment. The coordinates (UTM northing and easting) of the borehole locations are provided on the borehole records and on the Borehole Location Plan (Drawing No. NO11674-1 in Appendix B).

The ground surface elevations at the borehole locations were surveyed relative to an existing benchmark located on the west side of the north abutment of the Moodie Drive structure. The benchmark was identified on the contract survey plans as having a geodetic elevation of 74.392 m. The location of the benchmark and the borehole locations are shown on Drawing No. NO11674-1 located in Appendix B.

3.3 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual classification by a geotechnical engineer. Selected samples were tested for moisture content, Atterberg limits and grain size distribution. Selected samples of the recovered rock core were tested for unconfined compressive strength. All soil and bedrock samples will be stored for a period of twelve months after issuance of the final report. Unless otherwise directed, the stored samples will be disposed of after this period.

4.0 SUBSURFACE CONDITIONS

The borehole location plan is provided on Drawing No. NO11674-1 in Appendix B.

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix A. An explanation of the symbols and terms used to describe the Borehole Records is also provided. In general, the observed stratigraphy consisted of a thin layer of topsoil, overlying a fill layer, over native silty clay, over till, over bedrock. A detailed description of the subsurface conditions encountered is given below.

4.1 Topsoil/Rootmat

A layer of topsoil/rootmat approximately 75 mm thick was observed at all of the borehole locations.

4.2 Silty Sand to Silty Clay (Fill)

A layer of fill material was encountered in all of the boreholes. The composition of the fill varied from silty clay with sand and gravel, trace organic matter, to silty sand with clay and some organic matter. The thickness of the fill deposit was observed to range from 0.7 m to 1.3 m. The base of the fill layer varied from elevation 67.3 m to 67.8 m. Standard Penetration tests in the fill yielded N values of 5 and 6 blow/0.3 m.

The natural moisture content of one sample of the fill was 25%. A grain-size distribution analysis carried out on one sample of the fill indicated that it contained 2% gravel, 27% sand, and 71% silt and clay sized particles. The results of the grain size distribution analyses are provided in Figure 1 in Appendix A.

4.3 Silty Clay/Clay

A deposit of silty clay/clay was encountered below the fill layer in all boreholes.

The silty clay/clay within the upper 3 m was generally brown to greyish-brown, indicating an oxidized crust. The crust was too stiff to conduct in-situ shear vane tests but was described as very stiff to stiff based on visual-manual classification methods. Standard penetration test N-values within the crust ranged from 4 to 8. The moisture content of 5 samples tested ranged from 35% to 47% with an average of 40%. Atterberg Limit testing on two samples indicated Liquid Limits of 38% and 63% and Plastic Limit of 15% and 23%, corresponding to clays of intermediate to high plasticity (CI to CH). The base of the crust was located between elevations 65.5 m and 66.0 m.

Beneath the crust, the silty clay became grey in color. In-situ shear vane tests indicated that the undrained shear strength of the material ranged from 25 to 74 kPa, with an average of 59 kPa indicating that the clay layer ranged from firm to stiff, but is generally classified as stiff. The sensitivity of the clay ranged from 3 to 12. The moisture content of 12 samples tested ranged from 24% to 52% with an average of 38%. Atterberg Limit testing on six samples indicated Liquid Limits between 23% and 54% and Plastic Limits between 12% and 20%, corresponding to clays ranging from low to high plasticity (CL, CI and CH). The results of the Atterberg limit tests are provided on the plasticity charts in Figures 4 through 6 in Appendix A. The results of grain size analyses on three samples of the silty clay are presented in Figure 2 in Appendix A. The base of the silty clay deposit was located between elevations 57.2 m and 60.4 m.

4.4 Silty Sand (Till)

A glacial till deposit was encountered beneath the silty clay deposit in all of the boreholes. The glacial till consisted of silty sand (SM). The material also included gravel and clay sized particles. The thickness of the till deposit ranged from 2.9 m to 4.5 m. The base of the till deposit varied from elevation 56.7 m to 57.3 m.

The moisture contents of the two samples tested were 10% and 11%. Grain-size distribution analyses carried out on two samples of the till indicated that it contained 10% to 12% gravel, 42% to 43% sand, and 46% to 47% silt and clay sized particles. The results of the grain size distribution analyses are provided in Figure 3 in Appendix A.

Standard Penetration tests in the till yielded N values ranging from 3 blow/300 mm to 72 blows/250 mm indicating that the till ranges in density from very loose to very dense. Glacial Till within the area is also noted to contain cobbles and boulders.

4.5 Bedrock

Bedrock was proven by coring in Borehole BH 06-102 and in BH 06-103 at depths of 12.0 m and 11.4 m, respectively. The bedrock surface elevations are presented in the table below.

Table 4.1: Bedrock Elevations

Borehole	Bedrock Surface Elevation	Comments
BH 06-101	56.7 m	Auger Refusal
BH 06-102	57.1 m	Bedrock Cored
BH 06-103	57.2 m	Bedrock Cored
BH 06-104	57.3 m	Auger Refusal

The bedrock consists of light grey to buff dolomitic sandstone. Total core recoveries (TCR) were typically between 83% and 98% and rock quality designations (RQD) generally ranged from 70% to 80% indicating very poor to good rock mass quality. Detailed rock core descriptions are provided in the Rock Core Summary Table in Appendix A. In general, the rock was fresh to slightly weathered with minimal thinly laminated shale bedding and very close to closely spaced fractured. Unconfined compressive strength testing was carried out on four samples of the recovered rock core. The results ranged from 103 MPa to 113 MPa, corresponding to a very strong rock strength classification.

The results of the Unconfined Compressive Strength Testing are presented in Table 4.2 below:

Table 4.2: Summary of Bedrock Unconfined Compressive Strength

Location	Elevation (m)	Unconfined Compressive Strength (MPa)
BH 06-102	56.9	113
BH 06-102	57.1	112
BH 06-103	57.2	103
BH 06-103	57.4	111

4.6 Groundwater

Groundwater levels were measured within the standpipes on February 22, 2007. The measured water levels are summarized in the table below.

Table 4.3: Summary of Groundwater Levels

Location	Ground Surface Elevation (m)	Depth to Groundwater (m)	Groundwater Elevation (m)
06-101	68.7	1.9	66.8
06-102	69.1	2.3	66.8
06-103	68.6	blocked	--
06-104	68.6	1.6	67.0

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

5.0 CLOSURE

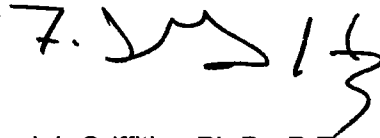
A subsurface investigation is a limited sampling of a site. The subsurface conditions provided herein are based on information gathered at specific borehole locations and can only be extrapolated to an undefined limited area around these locations. The extent of the limited area depends on the soil and groundwater conditions as well as the history of the site reflecting natural, construction and other activities. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Yours very truly,

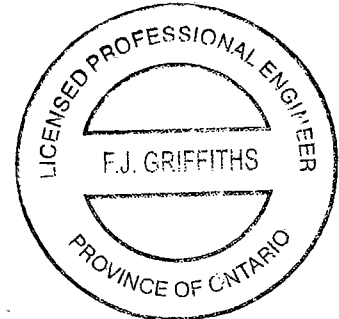
JACQUES WHITFORD LIMITED



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



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



FOUNDATION DESIGN REPORT
For
G.W.P. 302-89-00
WBL Retaining Wall
Highway 417/Moodie Drive Interchange
Ottawa Area

6.0 DISCUSSION

6.1 Proposed Development

It is understood that the Ministry of Transportation of Ontario (MTO) plans to widen Highway 417 to eight lanes from Highway 416 westerly to 0.5 km west of Eagleson Road. The widening requires the construction of a new retaining wall at the West Bound Lane (WBL) foreslope beneath the Moodie Drive overpass. This work is a component of G.W.P. 302-89-00.

Plans provided by MTO indicate that the existing Moodie Drive bridge is supported on Steel H-piles driven to refusal in bedrock. Each abutment is supported by two rows of piles. The front row of piles is battered toward the driving lanes of Highway 417, however, the proposed retaining wall alignment should result in a minimum lateral separation of 3 m between the battered piles and the edge of a footing founded at frost depth (1.8 m).

The limited clear height beneath the bridge structure will be a construction constraint consideration.

It is noted that, for project orientation purposes, Highway 417 will be assumed to run east-west at the Moodie Drive Interchange, with chainage increasing from west to east.

Design Objectives:

Based on preliminary plans, we understand that the proposed retaining wall is a standard concrete toe wall (OPSD 3120.100 Type III). The wall will be 52.2 m long and will vary in height above the Highway 417 WBL pavement shoulder from approximately 0.85 m to 1.1 m. The top of the foreslope fill will be somewhat lower than the top of wall elevation and will be sloped at 2H:1V. The alignment of the proposed retaining wall varies from 5219 mm to 5670 mm from the centerline of the north abutment bearing pads.

The site is within an area with a Mean Freezing Index of 1000 Degree Days (°C) (Canadian Foundation Engineering Manual). Using Figure 3.4 of the MTO Pavement Design Rehabilitation Manual, the Frost Penetration Depth for this area is 1.8 m.

6.2 Foundation Assessment

Soil conditions are relatively uniform along the proposed retaining wall alignment. The critical features of the soils profile include:

- A fill deposit extending to between 0.8 m and 1.4 m below the current ground surface.
- A very stiff to stiff silty clay crust extending to 3 m below the current ground surface and underlain by a weaker (stiff to firm) grey clay deposit.
- A glacial till deposit beneath the clay layer that may contain obstructions such as cobbles and boulders.
- Sandstone bedrock at a depth of 11 to 12 m below ground surface.
- Groundwater level approximately 1.6 m below ground surface.

Some of the critical design considerations for the proposed work include the following:

- Protection of adjacent structures including the Highway 417 WBL pavement structure and the Moodie Drive Bridge abutment and approach fills during excavation for the proposed retaining wall
- Height limitations for work carried out beneath the existing bridge structure.
- Potential interference with piles supporting the existing abutment.
- Horizontal and vertical loads on the wall are relatively low due to the short wall height.

6.3 Retaining Wall and Foundation Options

A standard OPSD Concrete Toe Wall (OPSD 3120.100 Type III) has been identified as the preferred wall design provided adequate foundation support is available. Table 6.1 compares the options considered for this site. Due to the relatively short height of the wall, geometry of the backslope and relatively competent soils beneath the proposed wall alignment, no problems are anticipated in achieving a design that provides the required resistance to lateral loads, sliding or global stability with any of the foundation options presented in Table 6.1.

Table 6.1: Comparison of Foundation Options

Option	Advantages	Disadvantages	Relative Cost	Risk/ Consequences
Concrete Toe Wall (OPSD 3120.100)	<ul style="list-style-type: none"> Standard design Economical from a structural perspective 	<ul style="list-style-type: none"> Existing soils will not provide required ULS bearing resistance 	N/A	Not feasible
Concrete Toe Wall (OPSD 3120.100) on granular pad	<ul style="list-style-type: none"> Standard design Economical from a structural perspective 	<ul style="list-style-type: none"> Requires some excavation and unwatering 	Low	
Retained Soil System	<ul style="list-style-type: none"> Flexible system can tolerate some movement 	<ul style="list-style-type: none"> Requires wider excavation for installation of reinforcement Not a standard design 	Medium	
Concrete Wall supported on Deep Foundations	<ul style="list-style-type: none"> High capacity 	<ul style="list-style-type: none"> Low head room restriction Potential interference with batter piles of abutment 	High	- potential interference with existing piles beneath abutment / extract piles and adjust design

A standard OPSD Concrete Toe Wall (OPSD 3120.100 Type III) supported on a granular pad is the recommended design/foundation option for this site. This option meets foundation design requirements and presents few special risks or disadvantages in terms of the potential construction constraints and considerations noted in Section 6.2. The impact of the selection of this foundation options on the potential construction constraints and considerations noted in Section 6.2 is discussed below:

- The face of the proposed wall is offset a lateral distance of between 5.2 m and 5.7 m from the face of the abutment. Based on the width of a standard toe wall design, this lateral offset should ensure that there is no interference with the existing piles supporting the abutment.

- Assuming 1H:1V excavation side slopes, excavation for construction of the standard toe wall on a 1.0 m thick granular pad extending to the frost depth of 1.8 m should not result in the existing piles being exposed.
- The limited height beneath the Moodie Drive structure would be a significant impediment if a deep foundation option was selected. For construction of the proposed toe wall on a granular pad, the space constraints (vertical clearance and proximity to Hwy 417 driving lanes and adjacent bridge abutment) will limit the size of equipment that can be used and will require care during operation.

7.0 RECOMMENDATIONS

7.1 Structure Foundations

7.1.1 Shallow Foundations – Bearing Resistances

A conventional gravity style retaining wall may be founded on undisturbed very stiff to stiff native clay/silty clay. Based on the geometry of an OPSD 3120.100 Type II or Type III toe wall and assuming that the wall is founded at the design frost penetration depth of 1.8 m (approximately elevation 67.3 m), it is anticipated that the base would vary in width from approximately 1.55 m to 1.65 m.

Based on the above design assumptions, the wall structure could be designed based on an ultimate limits states (ULS) bearing resistance of 235 kPa and a serviceability limit states (SLS) bearing resistance of 160 kPa. The ULS bearing resistance includes a resistance factor of 0.5. The SLS bearing resistance corresponds to 25 mm of total settlement. The effects of eccentrically loaded foundations should be considered in the structural design as per Section 6.7 of the CHBDC.

It is noted that the ULS bearing resistance provided above does not meet the requirements of an OPSD 3120.100 Type II or Type III Toe Wall. The ULS bearing resistance can be increased by constructing the wall on a granular pad as described in Section 7.1.2.

The sliding resistance at the base of the foundation can be calculated using an unfactored coefficient of friction of 0.35 for concrete on the very stiff to stiff clay.

7.1.2 Subgrade Improvement

A standard OPSD Concrete Toe Wall (OPSD 3120.100 Type III) may be used at this site provided it is founded on a 1.0 m thick structural fill pad. The base of the granular pad should extend to a minimum depth of 1.8 m below finished grade. The granular pad should be placed on clean undisturbed native silty clay crust.

Structural Fill should consist of OPSS Granular A placed and compacted to at least 100% Standard Proctor Maximum Dry Density (SPMDD). The Structural Fill should be placed throughout the influence zone of the footings. 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 edges of the base of the wall.

The subgrade prepared as described above will provide the required ULS bearing resistance of 300 kPa required for a Type III OPSD Concrete Toe Wall (OPSD 3120.100 Type III). The SLS bearing resistance, corresponding to 25 mm of total settlement, is 150 kPa.

The sliding resistance at the base of the foundation can be calculated using an unfactored coefficient of friction of 0.55 for concrete on the compacted OPSS Granular A.

7.1.3 Frost Protection

The frost penetration depth for design at this site is 1.8 m. The granular pad beneath the base of the wall should be drained with a longitudinal subdrain in order prevent accumulation of water within the depth of frost penetration.

7.2 Earth Pressure Design

The retaining wall should be backfilled with free-draining material such as OPSS Granular B Type II or OPSS Granular A to prevent hydrostatic pressure build-up. The extent of granular backfill should include the full wedge extending up and back from the bottom of the wall at an angle of 1H:1V in order for the properties of the granular backfill to govern for earth pressure design. The backfill wedge described above coincides with the safe excavation side slope limits identified in Section 8.1.

Computation of earth pressures should be in accordance with Section 6.9 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. The unfactored soil parameters in Table 7.1 may be used for the design of retaining structures with the following characteristics:

- 1H:3V wall backslope
- 2H:1V ground slope above top of wall

Table 7.1: Recommended Lateral Earth Pressure Parameters

Parameter	OPSS Granular A and Granular B Type II
Total Unit Weight, γ (kN/m ³)	22.0
Effective Friction Angle	37 degrees
Coefficient of Active Earth Pressure (K_a)	0.58
Coefficient of Passive Earth Pressure (K_p)	6.40

It is noted that the resultant lateral earth pressure is not horizontal but acts perpendicular to the back of the wall.

Compaction of the granular backfill near the walls should be carried out using hand-operated equipment to prevent over-stressing the abutment walls. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

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 3102.100 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 3101.150 using a depth of frost penetration, f , of 1.8 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 Ottawa is 0.20. Reference is made to Section C4.6.4 of the CHBDC for the calculation of seismic forces on 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 procedures outlined in CHBDC, Section C4.6.2. Liquefaction of Foundation Soils. Specifically, the Chinese criteria was used to assess the silty clay deposit.

The results of this assessment revealed that the silty clay deposit is not considered as liquefiable.

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 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
- 1H:3V back of wall
- 2H:1V ground slope above 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 7.2: Combined Coefficients of Static and Seismic Earth Pressure

Parameter	OPSS Granular A & Granular B Type II
Total Unit Weight, γ (kN/m ³)	22.0
Effective Friction Angle	37 degrees
Active Earth Pressure (K_{AE})	1.0
Height of application of P_{AE} from base as ratio of wall height (H)	0.429
Passive Earth Pressure (K_{PE})	6.4
Height of application of P_{PE} from base as ratio of wall height (H)	0.302

7.4 Global Stability

The global stability of the proposed OPSS 3120.100 Type III concrete toe wall founded on a 1.0 m thick granular pad extending to the design frost depth has been checked using the Slope/W slope stability modeling software.

The global stability was found to be greater than 2.0 under static loading conditions and greater than 1.6 under seismic loading conditions.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Open Cut Excavations

Earth excavation should be carried out in accordance with SP206S03. Side slopes for open cut excavations should conform to the requirements of the edition of the Occupational Health and Safety Act and Regulations for Construction Projects (OHSA) current at the time of construction. In accordance with the present act, the existing fill and silty clay would be considered a Type 3 soil and temporary excavations deeper than 1.2 m should be made with side slopes no steeper than one horizontal to one vertical from the base of the excavation.

Excavation for the granular pad and toe wall are not expected to expose the existing piles supporting the Moodie Drive structure abutments or to extend beneath the abutment, therefore, no restrictions need be applied regarding limitations to the section length of excavation open at any given time.

Excavation side slopes should, however, be protected from erosion and should be inspected regularly for signs of instability. Excavation side slopes should be protected from erosion and should be inspected regularly for signs of instability. Slopes should be flattened or supported as required to

maintain safe working conditions. The excavation side slopes in accordance with the OHSA are intended for short term temporary conditions and will not remain stable in the long term. Therefore, completion of the wall in stages would be appropriate if it is expected that the excavation will remain open for longer than 3 weeks.

8.2 Site Preparation

Site preparation should be carried out in accordance with the requirements of SP 902S01 Excavation and Backfilling - Structures.

8.3 Dewatering

Dewatering of temporary excavations will be required. Dewatering of structure excavations should be carried out in accordance with OPSS 902.07.06.

No special dewatering problems are anticipated. Dewatering using conventional sump pumping techniques is expected to be adequate for the depth of excavation proposed for the recommended foundation option.

8.4 Cement Type and Corrosion Potential

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

Table 8.1: Chemical Test Results

Borehole	Sample	pH (pH units)	Resistivity (ohm.m)	Soluble Sulphate (:g/g)	Chloride (:g/g)
BH 06-102	SS3	7.46	7.6	120	820
BH 06-104	SS3	7.63	4.3	140	1,800

The soluble sulphate results indicate that Type GU (formerly Type 10) Portland cement is 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 systems.

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.

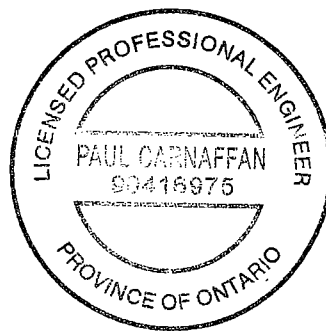
We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Yours very truly,

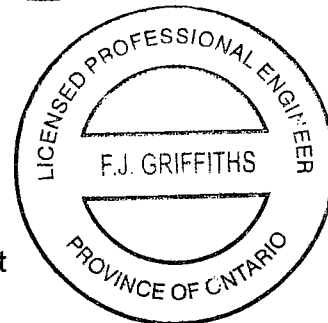
JACQUES WHITFORD LIMITED



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



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



P:\2007\10000\11674-hwy 417\Moodie WBL Wall\FINAL Fnd Invest & Design Report March 2007.doc

APPENDIX A

Symbols and Terms Used on Borehole and
Borehole Records
Laboratory Test Results
Rock Core Summary Table

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

Terminology describing soil types:

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

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

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

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

Terminology describing compactness of cohesionless soils:

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

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

Terminology describing consistency of cohesive soils:

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

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



ROCK DESCRIPTION

Terminology describing rock quality:

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

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

Terminology describing rock mass:

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

Terminology describing rock strength:

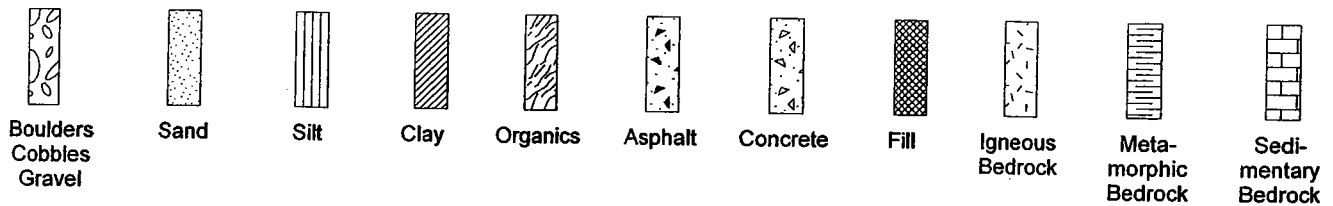
Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

Terminology describing rock weathering:

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

STRATA PLOT

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



SAMPLE TYPE

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

WATER LEVEL MEASUREMENT



measured in standpipe, piezometer, or well



inferred

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

METRIC[illegible]

OLD MTO NO11674.GPJ OLD MTO.GDT 07/03/26

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

RECORD OF BOREHOLE No BH 06-102

1 OF 1

METRIC

W.P. GWP 302-89-00 LOCATION Highway 417 WBL at Moodie Drive, 356403.1N, 5022293.1E ORIGINATED BY AB
 DIST 42 HWY 417 BOREHOLE TYPE HS Augers, Split Spoons, Casing, NQ Core COMPILED BY JF
 DATUM Geodetic DATE 06.11.23 - 06.11.24 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	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		
69.1	Grass																	
68.9	75 mm ORGANICS/TOPSOIL		1	BS	-										GR SA SI CL			
68.3	Silty sand with clay, some gravel, trace organic matter, brown (FILL)																	
0.8	Silty clay with sand, brown (FILL)		2	SS	6													
67.7																		
1.4	CLAY of high plasticity, brown to greyish brown, very stiff to stiff (CH)		3	SS	7													
			4	SS	4													
66.1																		
3.0	SILTY CLAY, grey, stiff (Cl)																	
			5	SS	3													
			6	SS	3													
			7	SS	1/300 mm													
			8	SS	1/450 mm													
59.9																		
9.1	Silty sand, loose to very dense (TILL) (SM)		9	SS	4													
			10	SS	19													
57.1			11	SS	72													
12.0	BEDROCK																	
	Dolomitic Sandstone		12	NQ														
	Light grey to buff, fresh, minimal thinly laminated shale bedding, very close to closely spaced fractures (dip of 0 degrees), very strong		13	NQ														
54.1																		
14.9	End of Borehole																	
	Standpipe installed to 6 m																	

RECORD OF BOREHOLE No BH 06-103

1 OF 1

METRIC

W.P. GWP 302-89-00 LOCATION Highway 417 WBL at Moodie Drive, 356433.7N, 5022308.9E ORIGINATED BY AB
 DIST 42 HWY 417 BOREHOLE TYPE HS Augers, Split Spoons, Casing, NQ Core COMPILED BY JF
 DATUM Geodetic DATE 06.11.22 - 06.11.23 CHECKED BY PC

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

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

RECORD OF BOREHOLE No BH 06-104

1 OF 1

METRIC

W.P. GWP 302-89-00 LOCATION Highway 417 WBL at Moodie Drive, 356444.9N, 5022318.8E ORIGINATED BY AB
 DIST 42 HWY 417 BOREHOLE TYPE HS Augers, Split Spoons COMPILED BY JF
 DATUM Geodetic DATE 06.11.23 - 06.11.23 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED × FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
68.6	Grass													
68.4	75 mm ORGANICS/TOPSOIL		1	BS	-		68							
67.8	Silty clay with sand, brown (FILL)													
0.8	SILTY CLAY, brown to greyish brown, very stiff to stiff (CI)		2	SS	6		67					41.5		
			3	SS	7									
			4	SS	4		66							
65.6														
3.0	SILTY CLAY, grey, stiff (CI to CL)		5	SS	1/600 mm		65					42		0 20 (80) PL=17 LL=42
								x ⁴						
			6	SS	hyd		64							
			7	SS	hyd		63							
							62							
								x ¹²						
60.4			8	SS	hyd		61							
8.2	Silty sand, loose to very dense (TILL) (SM)		9	SS	hyd		60							PL=12 LL=23
							59							
							58							
57.4	End of Borehole													
11.2	Auger Refusal on Inferred Bedrock Standpipe installed to 6 m hyd = split spoon advanced by hydraulic push due to hammer malfunction													

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

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

Borehole #	Sample #	Recovery (%)	R.Q.D. (%)	Description
06-102	12	98	77	Dolimitic SANDSTONE, light grey to buff, fresh, minimal thinly laminated shale bedding, very close to closely spaced fractures: (dip of 0 degrees), very strong
	13	93	80	
	13	83	0	
06-103	14	98	70	Dolimitic SANDSTONE, light grey to buff, fresh to slightly weathered, minimal thinly laminated shale bedding, very minor rust staining at fractures, closely spaced fractures: (dip of 0 degrees), very strong
	15	97	75	

P:\2007\10000\11674-hwy 417\Moodie WBL Wall\Rock Core Summary Table.xls

UNIFIED SOIL CLASSIFICATION SYSTEM

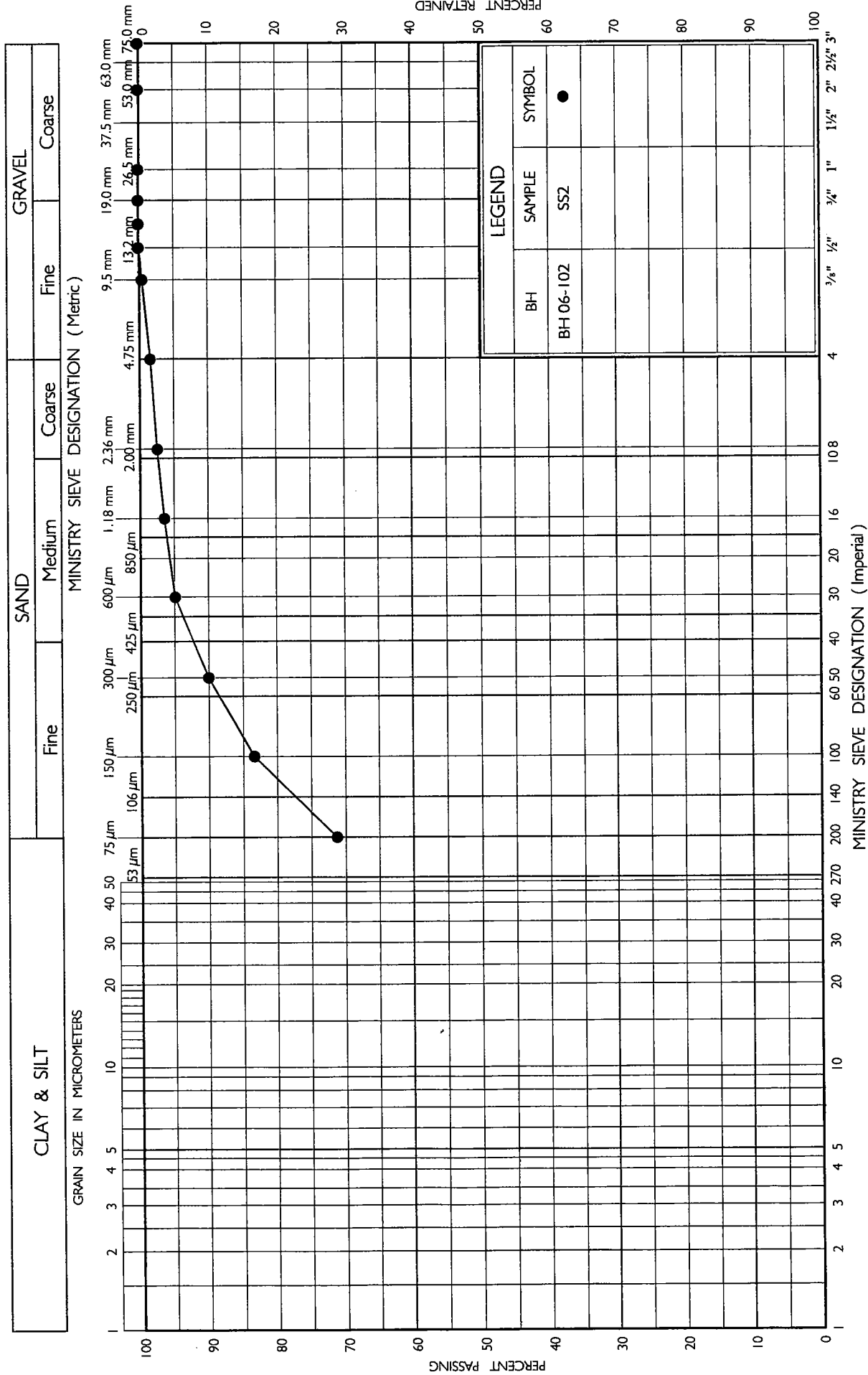


FIG No 1
GRAIN SIZE DISTRIBUTION

FILL - SILTY CLAY WITH SAND

UNIFIED SOIL CLASSIFICATION SYSTEM

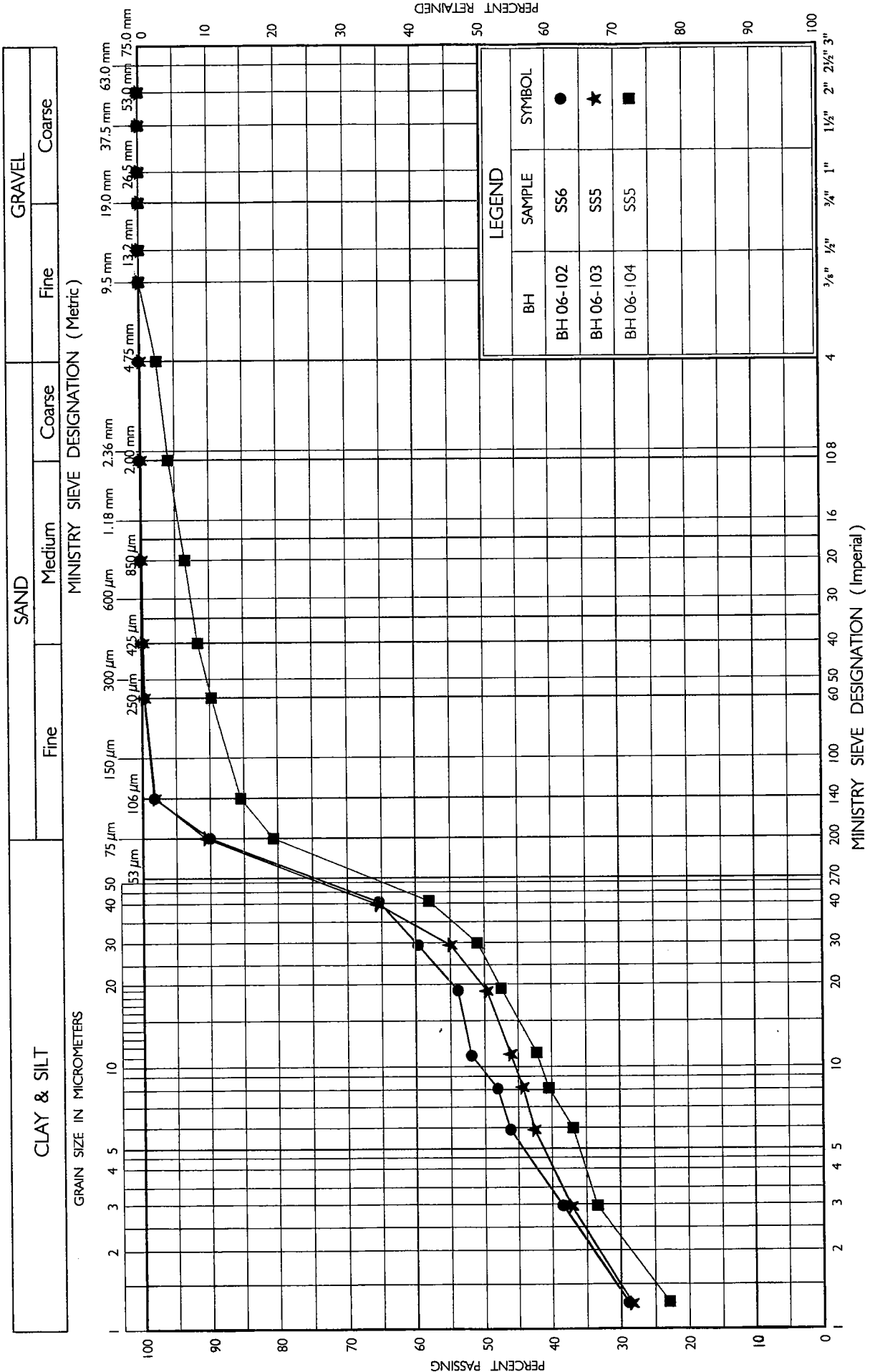
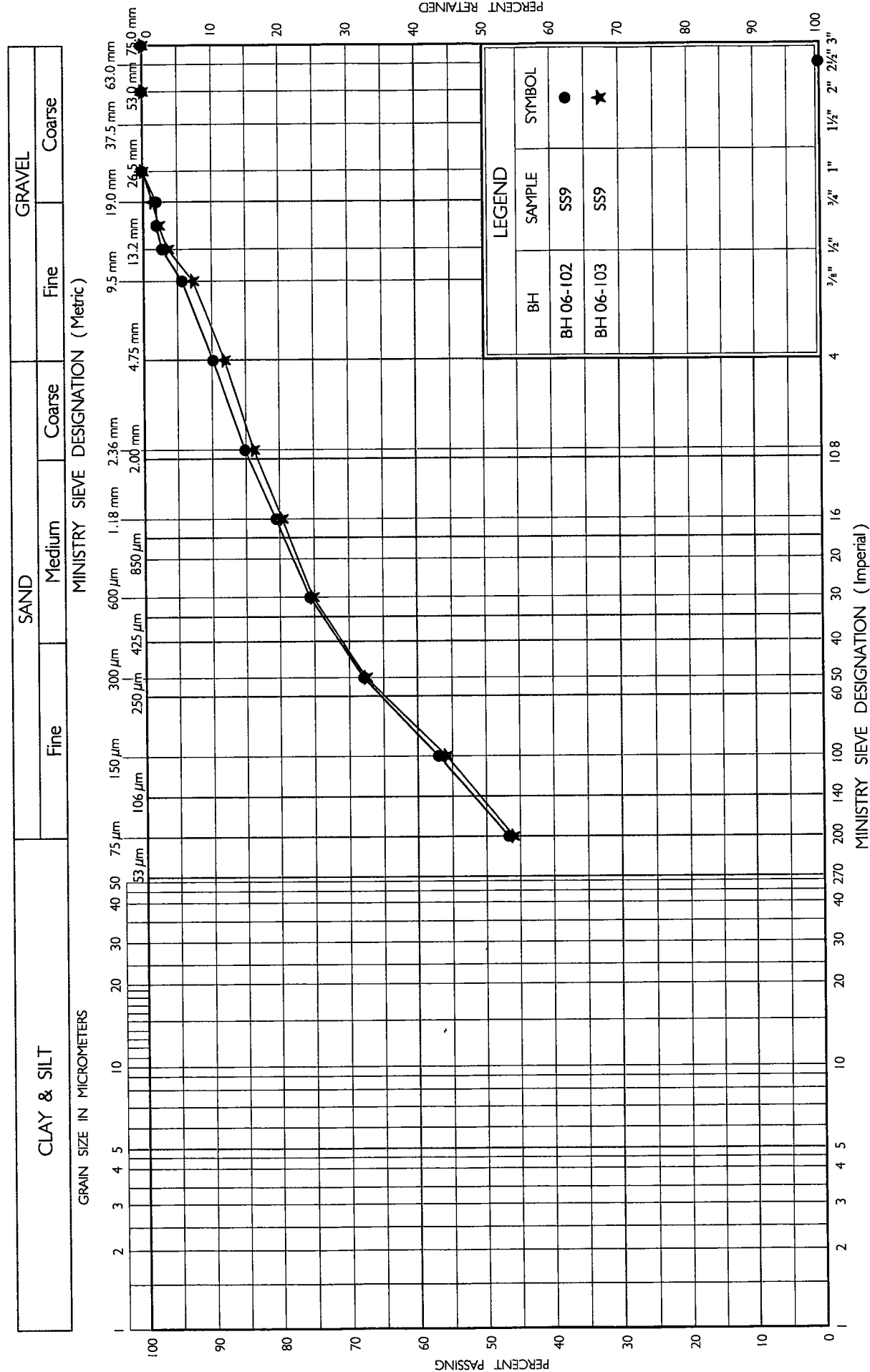


FIG No 2
GRAIN SIZE DISTRIBUTION
SILTY CLAY

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION

TILL - SILTY SAND

FIG No 3

GWP 302-89-00

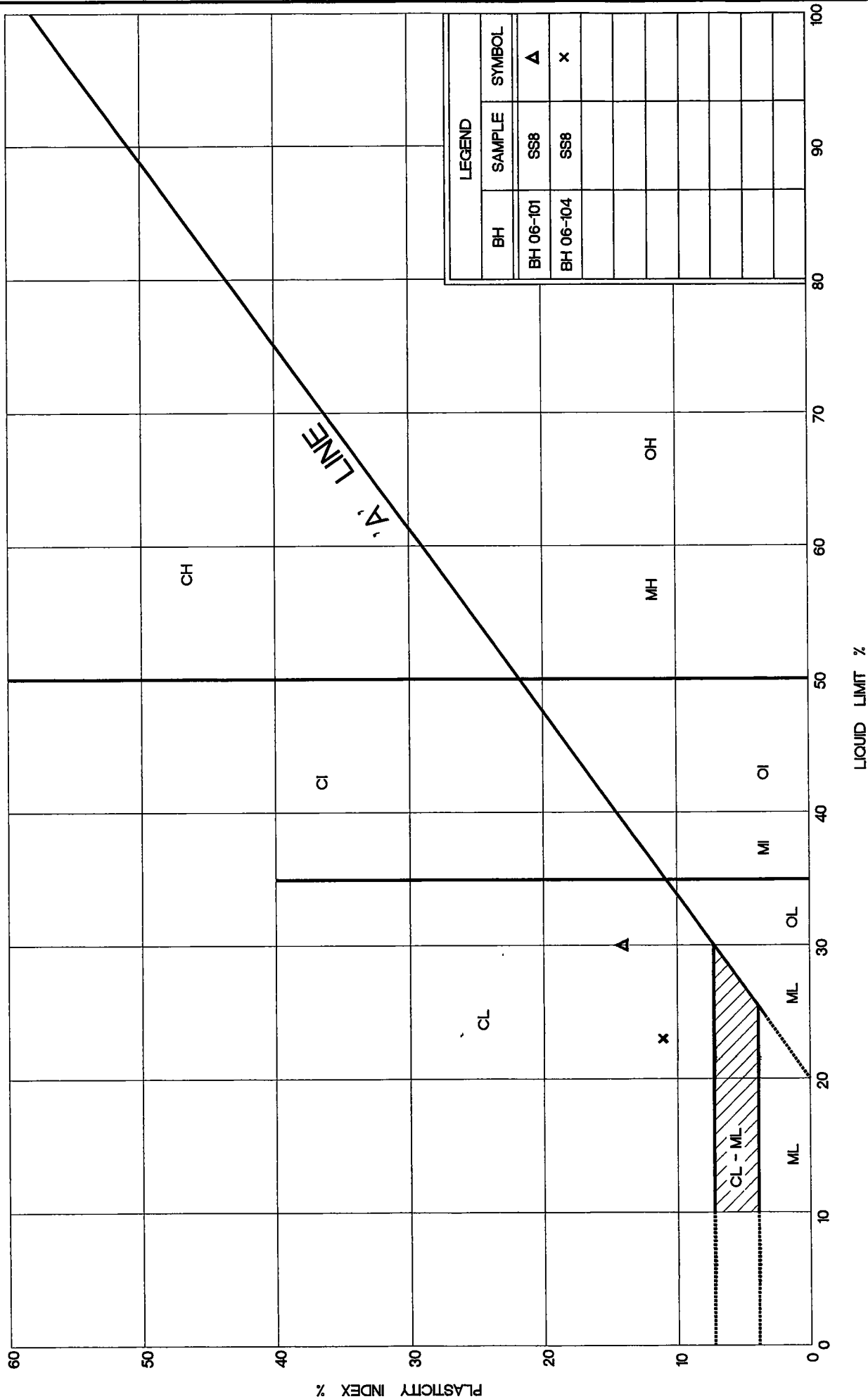
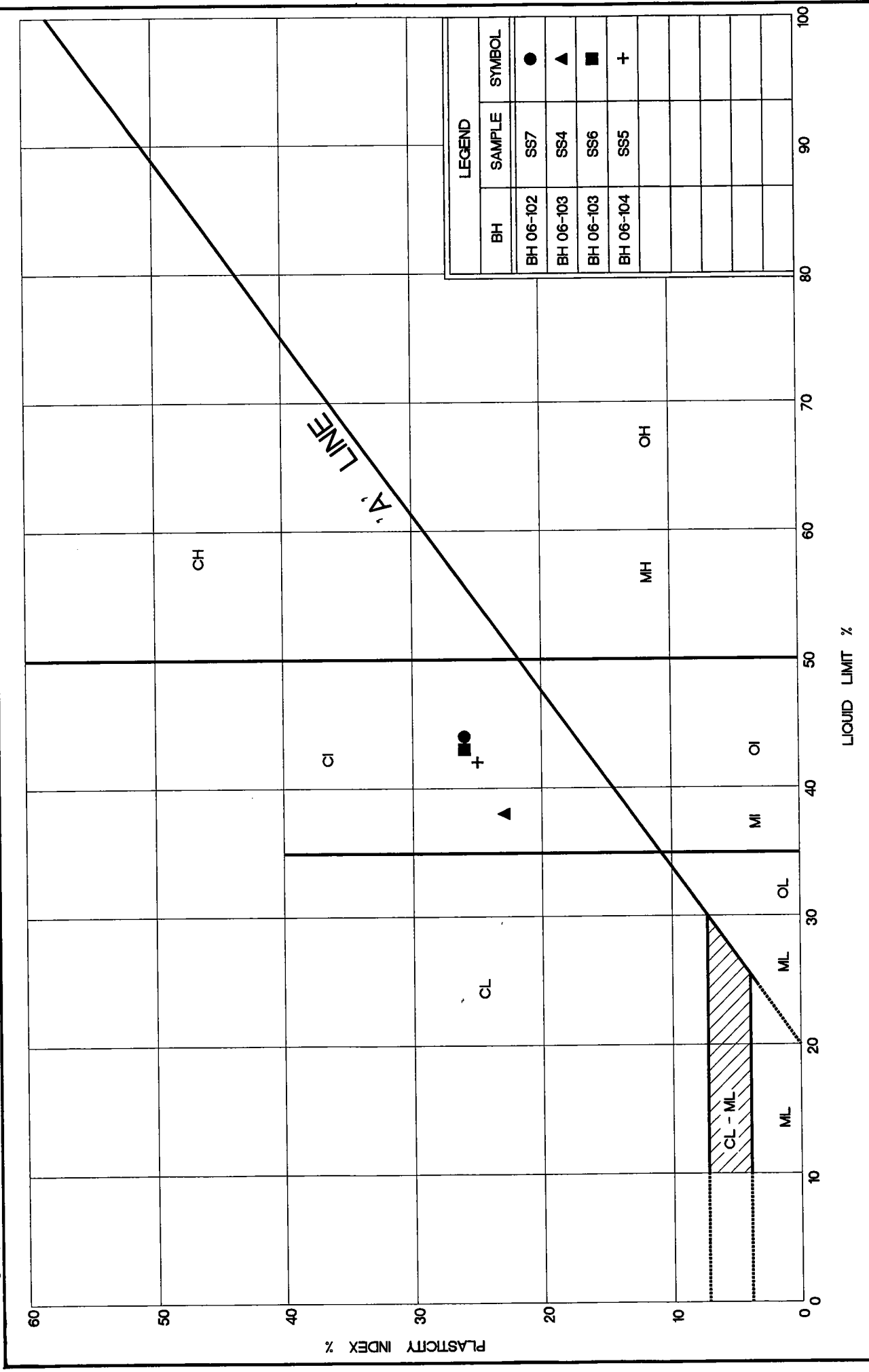


FIG No 4

PLASTICITY CHART

SILTY CLAY (CLAY OF LOW PLASTICITY)





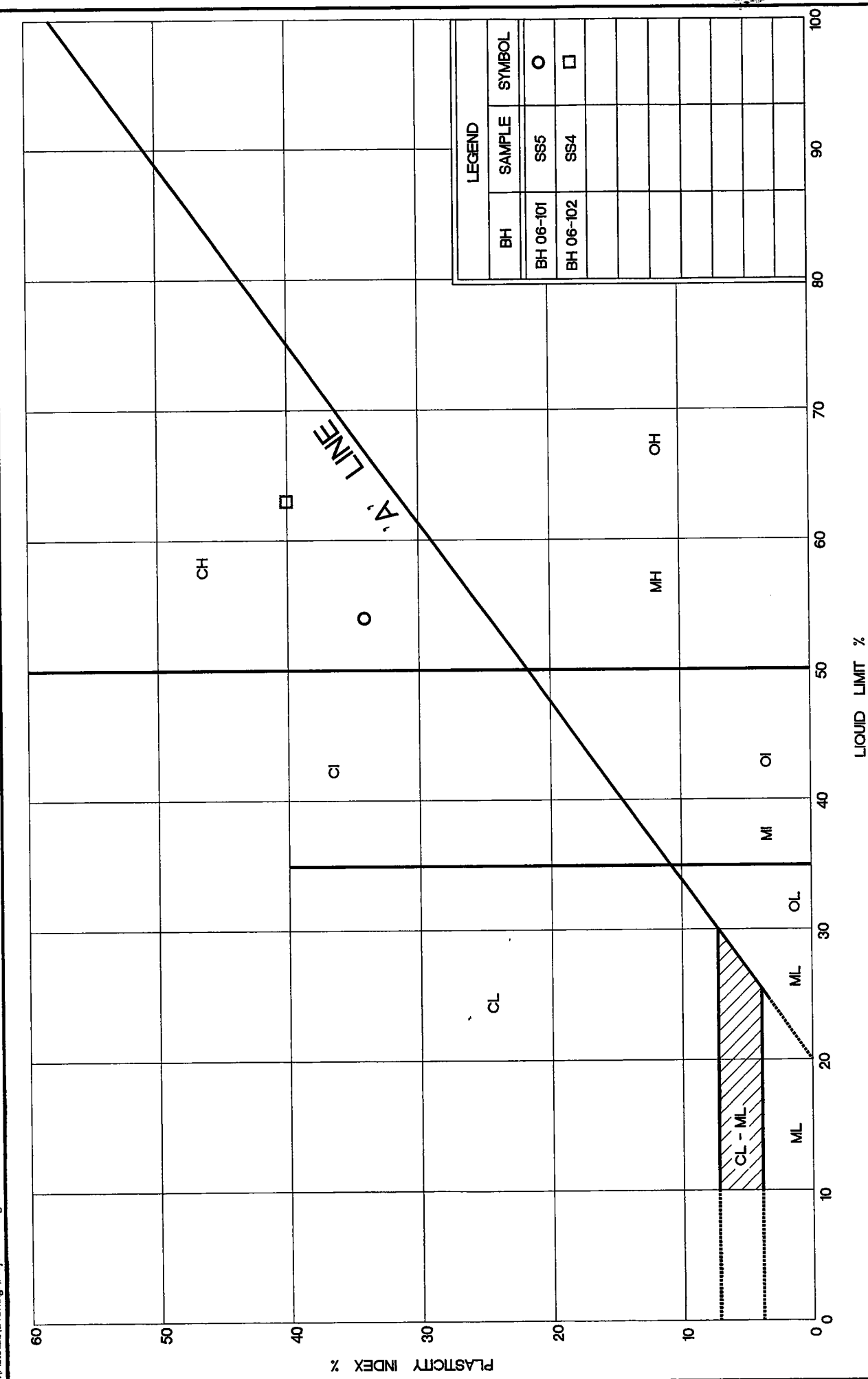
Ministry of
Transportation
Ontario

FIG No 5

PLASTICITY CHART

SILTY CLAY (CLAY OF INTERMEDIATE PLASTICITY)

GWP **302-89-00**



PLASTICITY CHART CLAY OF HIGH PLASTICITY

FIG No 6

GWP	302-89-00
-----	-----------



PARACEL

Laboratories Ltd.

Environmental & Indoor Air Quality

300-2319 St. Laurent Blvd.
Ottawa ON K1G 4J8
Phone: (613) 731-9577
Fax: (613) 731-9064
Toll Free: 800-7491947
email: paracel@paracellabs.com

Order #: L8781

Certificate of Analysis

Jacques Whitford Limited

2781 Lancaster Road, Suite 200
Ottawa, Ontario K1B 1A7
Attn: Mr. Paul Carnaffan

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

Client PO: **N011674, phase Z9100**

Project: **Highway 417**
Custody #: **39140**

Report Date: 06-Dec-2006
Order Date: 30-Nov-2006

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
L8781.1	BH06-102 SS3
L8781.2	BH06-104 SS3

Approved By: _____ Dale Robertson, B.Sc.
Laboratory Director

Any use of these test results implies your agreement that our total liability in connection with this work, however arising, shall be limited to the amount paid by you for this work, and that our employees or agents shall not under any circumstance be liable to you in connection with this work.

*Certificate of Analysis***Client: Jacques Whitford Limited**

Client PO: N011674, phase Z9100

Project: **Highway 417**

Report Date: 06-Dec-2006

Order Date: 30-Nov-2006

Analysis Summary Table

Analysis	Method Reference/Description
Anions	EPA 300.1 - ion chromatography
pH	EPA 150.1 - pH probe
Resistivity	EPA 120.1 - electrode

n/a: not applicable

MDL: Method Detection Limit

Soil results calculated on a dry weight basis.

Certificate of Analysis

Client: Jacques Whitford Limited

Client PO: N011674, phase Z9100

Project: Highway 417

Report Date: 06-Dec-2006

Order Date: 30-Nov-2006

Matrix: Soil

Parameter	Sample ID:	BH06-102 SS3	BH06-104 SS3
	Sample Date:	24/11/2006	24/11/2006
	MDL/Units	L8781.1	L8781.2
Chloride	5 ug/g	820	1,800
Sulphate	5 ug/g	120	140
pH	0.05 pH units	7.46	7.63
Resistivity	0.1 ohm.m	7.6	4.3

*Certificate of Analysis***Client: Jacques Whitford Limited**

Client PO: N011674, phase Z9100

Project: **Highway 417**

Report Date: 06-Dec-2006

Order Date: 30-Nov-2006

QA/QC Results

	Blank	Spike (QC Limits)	Duplicate	
Chloride	< 5 ug/g	87% (75 - 125%)	820	820
Sulphate	< 5 ug/g	95% (75 - 125%)	120	120
pH	n/a	n/a	7.45	7.45
Resistivity	< 0.1 ohm.m	n/a	7.6	7.6

Rock Core Summary Table

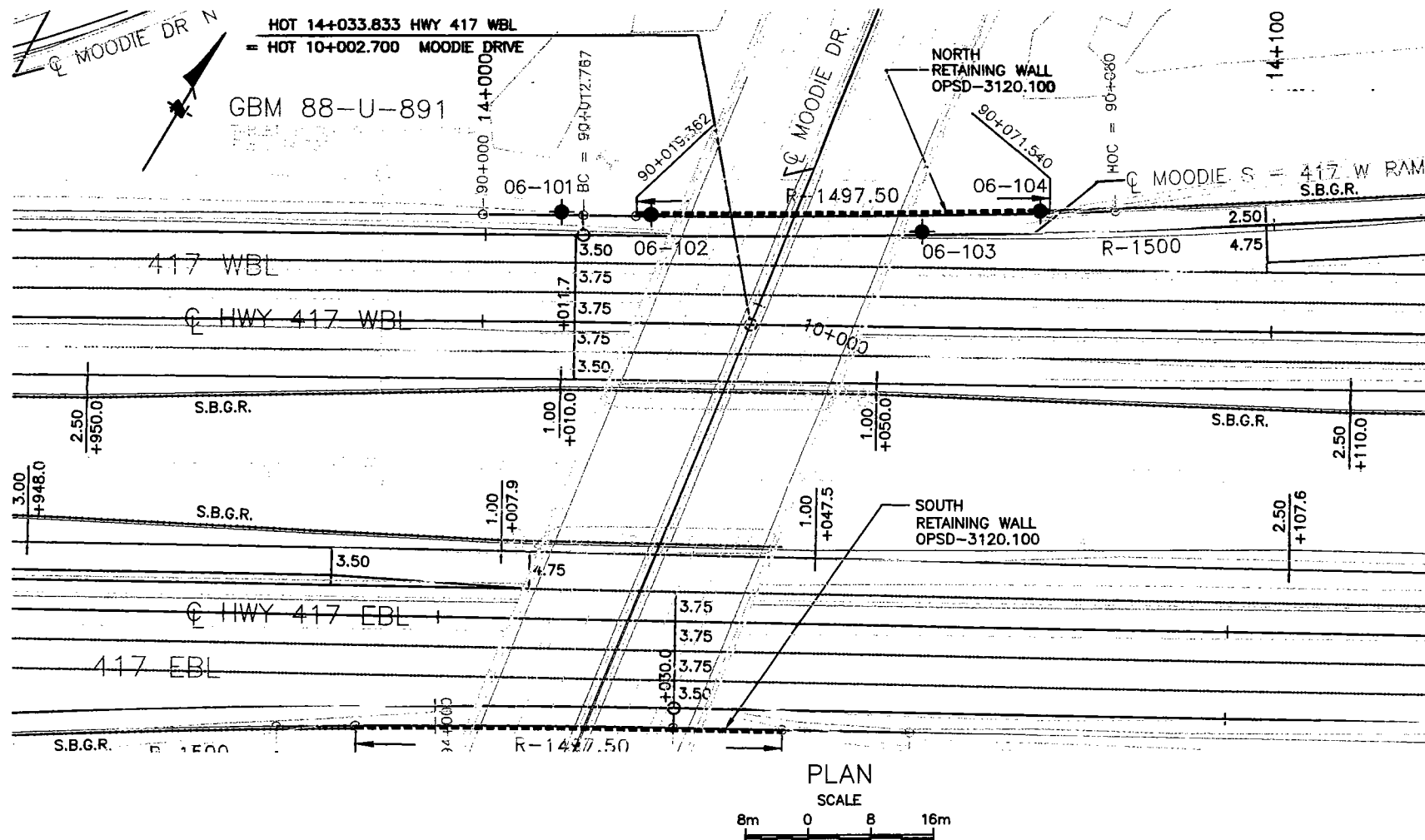
W.P. 302-89-00

Borehole #	Sample #	Recovery (%)	R.Q.D. (%)	Description
06-102	12	98	77	Dolimitic SANDSTONE, light grey to buff, fresh, minimal thinly laminated shale bedding, very close to closely spaced fractures: (dip of 0 degrees)
	13	93	80	
	13	83	0	
06-103	14	98	70	Dolimitic SANDSTONE, light grey to buff, fresh to slightly weathered, minimal thinly laminated shale bedding, very minor rust staining at fractures, closely spaced fractures: (dip of 0 degrees)
	15	97	75	

P:\2007\10000\11674-hwy 417\Moodie WBL Wall\Rock Core Summary Table.xls

APPENDIX B

Borehole Locations and Strata (Drawing No. NO11674-1)

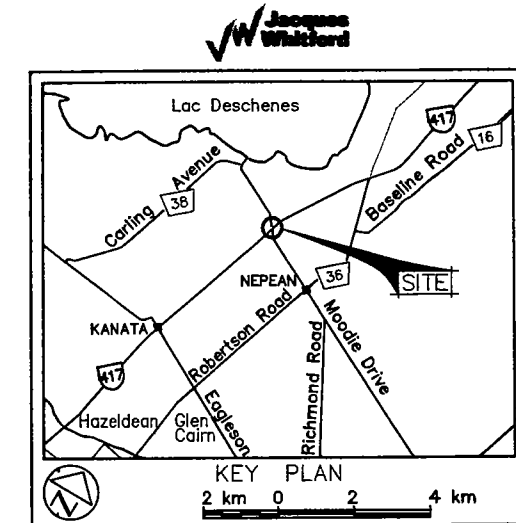
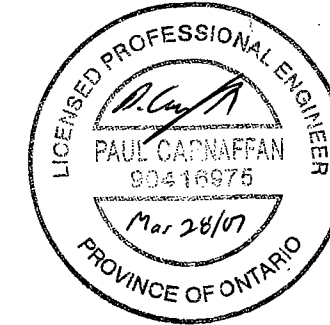


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN







CONT No 2006-4080
WP No 302-89-00

MOODIE DRIVE / HWY 417
NORTH RETAINING WALL
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 Nov 2006
	WL in Piezometer
	Piezometer

No	ELEVATION	COORDINATES	
		NORTH	EAST
06-101	68.7	356 393.3	5 022 287.6
06-102	69.1	356 403.1	5 022 293.1
06-103	68.6	356 433.7	5 022 308.9
06-107	68.6	356 444.9	5 022 318.8

==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, Downview. 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
GEORGES No 31G5-210			

HWY No 417		DIST 9	
SUBMIT	PC	DATE 2007-03-28	SITE 3-255
DRAWN	GH	APPROV <i>PC</i>	DWG NO 11674-1