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FOUNDATION INVESTIGATION AND DESIGN REPORT

EBL Retaining Wall
Highway 417/Moodie Drive
Interchange – Ottawa Area
G.W.P. 302-89-00

Totten Sims Hubicki

**REPORT NO. NO11674
Geocres # 31G5-203**

REPORT NO. NO11674

TO

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

ON

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

Ottawa Area
Geocres # 31G5-203**

November 2007

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FOUNDATION INVESTIGATION REPORT

For

G.W.P. 302-89-00

EBL Retaining Wall

Highway 417/Moodie Drive Interchange

Ottawa, Ontario

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 East Bound Lane (EBL) 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 our proposal number ONO030445 dated November 11, 2003. 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 EBL 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 two-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, particularly at the east end, where it appears that erosion due to water flowing down from the Moodie Drive pavement level has undermined the slope paving (see photos in Appendix C). The remainder of the approach embankment is vegetated with grass and small brush and does not exhibit any obvious signs of erosion or instability.

An existing 914 mm diameter storm sewer is located beneath the existing slope in front of the abutment as shown on Figure 4 in Appendix C.

3.0 INVESTIGATION PROCEDURES

3.1 Field Program

The fieldwork for this investigation was carried out between January 12 and February 10, 2004. The subsurface conditions were investigated through a borehole drilling program. A total of four (4) boreholes, numbered BH04-1 through BH04-4, were advanced at select locations. Boreholes BH04-1 and BH04-4 were advanced to auger refusal. Boreholes BH04-2 and BH04-3 were advanced approximately 3 m below the bedrock surface by coring.

All boreholes were drilled using a truck-mounted CME 55 power auger drill suitably equipped for soil and bedrock sampling. 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 BH04-2 and BH04-3 were advanced a minimum of 2.8 m into bedrock by coring with NQ-sized coring equipment. In situ vane shear testing was carried out to determine the undrained shear strength of cohesive soils. Shelby tube samples were recovered from Borehole BH04-2 between depths of 3.1 m and 9.1 m below ground surface.

Standpipes were installed in two of the boreholes and a monitoring well was installed in one borehole. Groundwater levels were measured in the standpipes and the monitoring well on February 10, 2004. All boreholes were backfilled with a bentonite-cement mixture.

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 ground surface elevations at the borehole locations were surveyed relative to an existing benchmark located on the west side of the north abutment. 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 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.

The borehole location plan is provided on Drawing No. NO11674-1 in Appendix B. A detailed description of the subsurface conditions encountered is given below.

4.1 Topsoil/Rootmat

A layer of topsoil/rootmat varying from 40 mm to 140 mm in thickness was observed at all of the borehole locations.

4.2 Fill

A layer of granular fill material was encountered in all of the boreholes. It consisted of sand, some gravel, trace silt, trace clay to silty sand, trace clay, trace gravel. A fill layer of brown grey silty clay was observed in borehole BH04-1 overlying the granular fill layer. The thickness of the fill deposit was observed to range from 2.4 m to 3.9 m. The base of the fill layer varied from elevation 65.0 m to 66.4 m.



Standard Penetration tests in the fill yielded N values ranging from 5 blow/0.3 m to 93 blows/0.3 m indicating that the fill ranges in density from very loose to very dense. Split spoon refusal (50+ blows/0.15 m) was encountered at several locations within the fill. The occurrences of split spoon refusal have been attributed to the presence of gravel within the fill.

The natural moisture content of seven samples tested ranged from 9% to 24% with an average of 18%. Three grain-size distribution analyses carried out on representative samples of the granular fill indicated that it contained 3% to 12% gravel, 56% to 68% sand, and 20% to 41% silt and clay sized particles. The results of the grain size distribution analyses are provided in Appendix A.

4.3 Silty Clay

A deposit of silty clay was encountered below the fill layer in all boreholes. This silty clay was grey in colour and extended to a depth of as much as 9.5 m below ground surface in Borehole BH04-4. The base of the silty clay deposit varied from elevation 59.4 m to 60.5 m. The thickness of the silty clay deposit ranged from 5.4 m to 7.1 m at the borehole locations.

The moisture content of 18 samples tested ranged from 32% to 52% with an average of 42.5%. Atterberg Limits testing on two samples from Borehole BH04-3 indicated a Liquid Limit of 39% and 51% and Plastic Limit of 18% and 19% at depths of 5 m and 7.8 m, respectively.

In situ shear vane tests indicated that the undrained shear strength of the material was relatively uniform with depth and ranged from 32 to 50 kPa indicating that the clay layer has a firm consistency. The sensitivity of the clay ranged from approximately 2 to 13 with an average of 7. The deposit is generally classified as sensitive clay. SPT N-values ranged from 1 to 8 blows for 300 mm of penetration.

4.4 Till

A glacial till deposit was encountered beneath the silty clay deposit in all of the boreholes. The glacial till consisted of silty sand, trace clay, trace gravel. The thickness of the till deposit was inferred to range from 1 m to 3 m. The inferred base of the till deposit varied from elevation 57.3 m to 58.4 m.

The moisture content of ten samples tested ranged from 8% to 12% with an average of 11%.

Standard Penetration tests in the till yielded N values ranging from 2 blow/0.3 m to 63 blows/0.3 m indicating that the till ranges in density from very loose to very dense. Split spoon refusal (50+ blows/0.15 m) was encountered at several locations within the till. The occurrences of split spoon refusal have been attributed to the presence of gravel within the till material.



Glacial Till within the area is also noted to contain cobbles and boulders.

4.5 Bedrock

Bedrock was proven by coring in Borehole BH04-2 and in BH04-3 at depths of 11.6 m and 10.7 m, respectively. The bedrock surface elevations are presented in the table below.

Table 4.1: Bedrock Elevations

| Borehole | Bedrock Surface Elevation | Comments |
|----------|---------------------------|---------------|
| BH04-1 | 57.6 m | Auger Refusal |
| BH04-2 | 57.3 m | Bedrock Cored |
| BH04-3 | 58.2 m | Bedrock Cored |
| BH04-4 | 58.4 m | Auger Refusal |

The bedrock consists of white to grey sandstone with close to moderately spaced fractures. The aperture of the fractures was typically less than 0.5 mm. Core recoveries were typically between 88 % and 100 % and rock quality designations (RQD) generally ranged from 88 % to 94 % indicating good to excellent rock mass quality. Detailed rock core descriptions are provided in the Rock Core Summary Table in Appendix A. Unconfined compressive strength testing was carried out on four samples of the recovered rock core. The results ranged from 89 MPa to 104 MPa.

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

Table 4.2: Summary of Bedrock Unconfined Compressive Strength

| Location | Elevation | Unconfined Compressive Strength (MPa) |
|----------|-----------|---------------------------------------|
| BH04-2 | 56.6 m | 91 |
| BH04-2 | 55.3 m | 81 |
| BH04-3 | 57.5 m | 104 |
| BH04-3 | 56.0 m | 89 |

4.6 Groundwater

Groundwater levels were measured within the monitoring well and standpipes on February 10, 2004. 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) |
|----------|------------------------------|--------------------------|---------------------------|
| BH04-1 | 68.97 | 2.41 | 66.56 |
| BH04-2 | 68.95 | 2.33 | 66.62 |
| BH04-3 | 68.89 | 2.04 | 66.85 |

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
EBL Retaining Wall
Highway 417/Moodie Drive Interchange
Ottawa, Ontario
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, for 6.5 km. The widening requires the construction of a new retaining wall at the EBL foreslope beneath the Moodie Drive overpass. This work is a component of G.W.P. 302-89-00.

The proposed retaining wall construction will need to consider temporary support for the soil beneath the existing abutment and within the existing approach fills. Plans provided by MTO indicate that the existing 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 and therefore the potential for interference between the existing piles and new deep foundations supporting the proposed retaining wall will need to be considered. In addition, the original contract drawings indicate the presence of a 36" (914 mm) R.C.P. sewer beneath the slope paving near the proposed wall alignment. The centerline of the pipe is located 1.5 m north of the front face of the proposed wall. The invert of the pipe is approximately 5.4 m below the top of pavement. The pipe location is shown on the cross-section in Figure 5 in Appendix C. It is understood that this pipe is now abandoned. The limited clear height beneath the bridge structure will also be a constraint during the construction of the retaining wall.

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 will be 52.8 m long and will vary in height above the Highway 417 EBL pavement shoulder from 0.8 m to 1.8 m. The wall height includes a 0.3 m cap, thus the maximum height of retained soil is 1.5 m. The alignment of the proposed retaining wall is approximately 3.0 m from the centerline of the south abutment.

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 ranging from 2.4 m to 3.9 m thick.
- A intermediate to high plasticity clay layer between 5 m and 7 m thick with an undrained shear strength of between 32 and 50 kPa.
- A glacial till deposit beneath the clay layer that may contain obstructions such as cobbles and boulders.
- Sound sandstone bedrock at a depth of 10 to 12 m below ground surface.
- Groundwater level approximately 2 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 EBL pavement structure, 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. The option of installing deep foundations through the existing bridge deck is not practical for this assignment. No significant works are planned for the existing bridge deck, thus the costs to repair the temporary foundation installation holes, as well as the traffic staging considerations preclude this option.
- Potential interference with piles supporting the existing abutment.
- Potential interference with the sewer pipe located beneath the slope paving.
- Horizontal and vertical loads on the wall are relatively low due to the short wall height.



- The base of the wall will be constructed at a depth of between 1000 mm and 1450 mm below finished grade (top of shoulder pavement) in order to minimize the depth of excavation adjacent to the driving lanes and existing bridge abutments.

6.3 Foundation Options

Deep foundations are recommended for the support of the proposed retaining wall. From a geometric design perspective, a standard OPSD toe wall (OPSD 3120.100 Type III) is suitable for this site, however, shallow foundations are not considered suitable for the following reasons:

- Spread footings may impose excessive bending stresses on the existing battered piles that support the abutment.
- The undrained shear strength of the native silty clay is between 32 kPa and 50 kPa and therefore provides limited bearing resistance.
- The existing fill material is erratic in density and unsuitable to support conventional spread footings. There would be constructability issues relating to removal of up to 3.9 m of fill present beneath the proposed alignment. This approach would require extensive shoring and dewatering.

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



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 Will impose bending stresses on existing battered piles supporting adjacent abutment | Not feasible | 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"> Shoring required to retain soil beneath existing bridge abutment during construction of granular pad Will impose bending stresses on existing battered piles supporting adjacent abutment | Medium | |
| Concrete Toe Wall (OPSD 3120.100) on unshrinkable fill (staged excavation) | <ul style="list-style-type: none"> Standard design Economical from a structural perspective | <ul style="list-style-type: none"> Excavation may extend below water table resulting in dewatering issues. Will impose bending stresses on existing battered piles supporting adjacent abutment | Low | Excavation walls and/or existing embankment fill sloughs in during excavation prior to placing unshrinkable backfill. / Minor occurrences reinstated with unshrinkable fill; major occurrences require revision to work methodology resulting in construction delays and cost extras. |
| Retained Soil System | <ul style="list-style-type: none"> Economical Flexible system can tolerate some movement | <ul style="list-style-type: none"> Insufficient space for lateral reinforcement | Not feasible | Not feasible |
| Soil Nailing with cast-in-place or pre-cast wall facing | <ul style="list-style-type: none"> Not hindered by low head room. | <ul style="list-style-type: none"> Limited local experience Physical interference with existing piles and/or bridge abutment Generally not feasible in loose or saturated soil – this site has localized loose zones. Observed deformation to existing slope pavement suggests loose conditions exist on slope as well. Will impose bending stresses on existing battered piles supporting adjacent abutment | Not Feasible | Not Feasible |
| H-piles End-bearing on rock | <ul style="list-style-type: none"> readily installed on a batter rugged member can be used for both shoring and permanent wall | <ul style="list-style-type: none"> low head room restriction | Medium | |
| Pipe-piles End-bearing on rock | <ul style="list-style-type: none"> can be installed on a batter | <ul style="list-style-type: none"> low head room restriction not easily incorporated into shoring system | Medium | Damage to piles during driving resulting in need for additional piles. |
| Caissons End-bearing on rock | <ul style="list-style-type: none"> large capacity | <ul style="list-style-type: none"> difficult to batter require tremie concrete require cased holes low head room restriction | High | |

Table 6.1: Comparison of Foundation Options

| Option | Advantages | Disadvantages | Relative Cost | Risk/Consequences |
|---------------------------------|---|---|---------------|-------------------|
| H-piles as Friction Piles | <ul style="list-style-type: none">▪ readily installed on a batter▪ rugged member▪ can be used for both shoring and permanent wall | <ul style="list-style-type: none">▪ low head room restriction▪ reduced resistance compared to piles driven to rock▪ minimal resistance immediately after installation | Low | |
| Pipe-piles as Friction Piles | <ul style="list-style-type: none">▪ can be installed on a batter | <ul style="list-style-type: none">▪ low head room restriction▪ reduced resistance compared to piles driven to rock▪ minimal resistance immediately after installation▪ not easily incorporated into shoring system | Low | |
| Caissons as Friction Piles | <ul style="list-style-type: none">▪ greater resistance than driven piles due to capacity (but less than caisson on rock) | <ul style="list-style-type: none">▪ difficult to batter▪ require tremie concrete▪ require cased holes▪ low head room restriction▪ minimal resistance immediately after installation | High | |

The foundation options were discussed with the structural design team who indicated that the proposed wall design does not require high axial or lateral load resistance from the deep foundations that will support it. Therefore, based on a review of these options, it is recommended that the proposed retaining wall be founded on H-piles installed as friction piles.

7.0 RECOMMENDATIONS

7.1 Structure Foundations

7.1.1 Axial Resistance

The retaining walls may be supported on steel H-piles driven to provide a minimum embedment length of 5 m. It is understood that the design elevation at the underside of the pile cap is 67.555 m. Therefore a pile tip elevation of no higher than 62.555 m is recommended. The pile length must be selected based on the anticipated maximum loads (likely during construction) in comparison to the available geotechnical resistance.

The following values are recommended for the design of pile foundations:

Table 7.1: Axial Resistance of Piles

| Pile Type | Embedment Length (m) | Pile Tip Elevation (m) | Factored Axial Resistance at ULS (kN) | Axial Resistance at SLS (kN) |
|--------------|----------------------|------------------------|---------------------------------------|------------------------------|
| HP 310 x 110 | 5 | 62.555 | 45 | See Note 1 |
| HP 310 x 110 | 6 | 61.555 | 60 | See Note 1 |
| HP 310 x 110 | 7 | 60.555 | 80 | See Note 1 |

Note 1: The maximum load transfer from skin friction, pile to soil, is normally considered to occur when the relative movement between the pile and soil is in the range of 0.5 to 1.0 mm for sands and 1.5 to 2.0 mm for clays. Assuming an SLS value equal to the ULS value, pile movements of less than 5 mm would be anticipated. Furthermore, elastic compression of these piles is likely to be minimal given the limited pile length and low anticipated loads. Therefore SLS movements need not be considered.

Note 2: The piles will have very little resistance at ULS immediately after driving. It should be assumed that 70% of the capacity will be achieved 2 weeks after installation and 90% two months after installation.



The factored axial resistance at ultimate limit states (ULS) includes a resistance factor of 0.4 in accordance with section 6.6.1 of the CHBDC (Deep Foundations – Static Analysis – Compression).

Downdrag forces are not expected to act on the piles since no new vertical loads will be imparted to the surrounding soils to cause settlement of the silty clay layer.

All pile driving activity should be inspected by trained geotechnical personnel.

7.1.2 Lateral Resistance

Passive lateral resistance for vertical piles should be calculated as per C6.8.7.2 (Static Analysis i.e. Brom's method) of the CHBDC using the following unfactored geotechnical soil parameters:

Table 7.2: Geotechnical Parameters for Determination of Lateral Resistance of Piles

| Parameter | Existing Trench Fill | Silty Clay | Till |
|---------------------------------------|----------------------|------------|------|
| Bulk Unit Weight, kN/m ³ | 19.0 | 18.0 | 20.0 |
| Effective Friction Angle, degrees | 30 | - | 32 |
| Coefficient of Passive Earth Pressure | 3.0 | - | 3.3 |
| Design Undrained Shear Strength, kPa | - | 40 | - |

Lateral resistance within the existing trench fill and the till should be calculated using the non-cohesive approach and within the silty clay using the cohesive approach, as defined in the CHBDC.

7.1.3 Lateral Deflections

The coefficient of horizontal subgrade reaction, which may be used for deflection calculations, may be estimated as follows:

Granular Soil (existing fill)

$$k_s = n_h z/d$$

where:

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

z = depth

n_h = 1500 kN/m³ and represents a coefficient related to soil density under saturated conditions to reflect spring thaw conditions.



d = pile diameter

Cohesive Soils (silty clay)

$$k_s = 67 C_u/d$$

where:

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

C_u = undrained shear strength of the soil = 40 kPa for this application

d = pile diameter

A design value of 40 kPa should be used for the undrained shear strength of the silty clay soil.

Group Effects on Lateral Deflections

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

The nature of pile-soil-pile interaction is complex, however is generally broken down into the following main components:

- alteration of the soil state due to pile installation and the potential overlap of the alterations when nearby piles are driven; and,
- superposition of strains and alterations of the soil failure zones when nearby piles are simultaneously loaded.

Studies (Reese, Isenhower and Wang, 2006) have reported the following relative flexibility between single piles and pile groups.

- Condition: Load is perpendicular to pile spacing

Table 7.3: Relative Flexibility Ratios for Pile Group Effects

| Pile Spacing c/c | PG/SP Relative Flexibility Ratio |
|------------------|----------------------------------|
| 4d | 1.0 |
| 3d | 0.9 |
| 2d | 0.8 |

Where: PG = Pile Group
SP = Single Pile

Where pile-soil-pile interaction is anticipated, the deflection for a specific lateral load may be estimated by dividing the single pile deflections by the applicable Relative Flexibility Ratio.



7.1.4 Tensile Resistance

Resistance to tensile loads should be calculated based on the shaft resistance of the piles in accordance with Section 6.8.5. A ULS Resistance Factor of 0.3 has been applied to derive the following values for use in design:

Table 7.4: Tensile Resistance of Piles

| Pile Type | Embedment Length (m) | Pile Tip Elevation (m) | Factored Geotechnical Resistance (Tension) at ULS (kN) |
|--------------|----------------------|------------------------|--|
| HP 310 x 110 | 5 | 62.555 | 35 |
| HP 310 x 110 | 6 | 61.555 | 45 |
| HP 310 x 110 | 7 | 60.555 | 60 |

7.1.5 Pile Notes

All piling work should be in accordance with SP903S01.

7.2 Seismic Design Considerations

7.2.1 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.

7.2.2 Soil Profile Type

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

7.2.3 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 is not classified as liquefiable.



7.3 Earth Pressure Design

The retaining wall should be backfilled with free-draining material such as OPSS Granular B Type II or OPSS Granular A connected to a subdrain to prevent hydrostatic pressure build-up.

During construction, it is anticipated that soldier piles with timber lagging will be used as the shoring method to retain the existing embankment fill. It is anticipated that water will be able to drain through the timber lagging. Where the shoring is to be incorporated into the final retaining wall design, backfilling with free-draining material will not be possible. Therefore, the design should include an engineered drainage system on the back face of the wall. The drainage system should connect to a subdrain at the base of the wall. The subdrain should consist of a 100 mm diameter perforated pipe subdrain surrounded on all sides by a minimum of 100 mm of 19 mm clear stone, wrapped with geotextile. The subdrain should be connected to a frost-free outlet.

7.3.1 Earth Pressures - Static

Computation of static 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. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. 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.

Table 7.5: Lateral Earth Pressure Parameters - Static Conditions

| Parameters | OPSS Granular A | OPSS Granular B, Type II | Existing Embankment Fill |
|------------------------------------|-----------------|--------------------------|--------------------------|
| Unit Weight (kN/m^3) | 22.0 | 22.0 | 19.0 |
| Angle of Internal Friction, ϕ | 35° | 35° | 33° |
| Horizontal Backslope | | | |
| Coeff. of Active Earth Pressure | 0.27 | 0.27 | 0.29 |
| Coeff. of Passive Earth Pressure | 3.69 | 3.69 | 3.39 |
| Coeff. of Earth Pressure at Rest | 0.43 | 0.43 | 0.46 |
| 2H:1V backslope | | | |
| Coeff. of Active Earth Pressure | 0.39 | 0.39 | 0.44 |
| Coeff. of Passive Earth Pressure | 10.80 | 10.80 | 9.29 |



Table 7.6: Lateral Earth Pressure Parameters - Combined Static and Seismic Conditions

| Parameter | OPSS Granular A | OPSS Granular B Type II | Existing Embankment Fill |
|---|-----------------------------|-----------------------------|-----------------------------|
| Total Unit Weight, γ (kN/m ³) | 22.0 | 22.0 | 19.0 |
| Effective Friction Angle | 35 degrees | 35 degrees | 33 degrees |
| | Horizontal Backslope | Horizontal Backslope | Horizontal Backslope |
| Active Earth Pressure (K_{AE}) | 0.33 | 0.33 | 0.36 |
| Height of application of P_{AE} from base as ratio of wall height (H) | 0.367 | 0.367 | 0.365 |
| Passive Earth Pressure (K_{PE}) | 3.48 | 3.48 | 3.19 |
| Height of application of P_{PE} from base as ratio of wall height (H) | 0.297 | 0.297 | 0.296 |
| | 2H:1V Backslope | 2H:1V Backslope | 2H:1V Backslope |
| Active Earth Pressure (K_{AE}) | 0.57 | 0.57 | 0.72 |
| Height of application of P_{AE} from base as ratio of wall height (H) | 0.405 | 0.405 | 0.424 |
| Passive Earth Pressure (K_{PE}) | 10.55 | 10.55 | 9.02 |
| Height of application of P_{PE} from base as ratio of wall height (H) | 0.307 | 0.307 | 0.306 |

7.4 Rock Anchors

Prestressed grouted rock anchors can be used to provide horizontal, overturning and uplift resistance. The following recommendations are provided for the design of grouted rock anchors:

- A factored geotechnical resistance at ULS of 750 kPa for the bond between rock and grout assuming a non-shrink grout having a minimum compressive strength of 30 MPa. A resistance factor of 0.4 has been applied to generate this value.
- Minimal deformation of the rock and grout is anticipated. Thus the SLS value should be determined based on elastic deformation of the tendon. It is anticipated that there will be minimal movement if the anchors are prestressed, thus SLS will not likely apply.
- The minimum fixed anchor length should be no less than 3 m.



- The minimum anchor spacing should be 900 mm centre to centre.
- To ensure against the possibility of rock mass failure, the following design parameters should be used:
 - submerged unit weight for bedrock of 14 kN/m^3 , and 8 kN/m^3 for overburden
 - a 60 degree apex angle failure cone in the bedrock with the apex located at the midpoint of the bonded length. Resistance from overburden should be calculated based on a truncated cone with a 20° angle from vertical. The interaction between cones must be included in the overall stability analysis.
- where cementitious grouts are used, the tendon area should not exceed 20% of the borehole area.

Construction of rock anchors should be carried out in accordance with SP999S26.

7.5 Frost Protection

The frost penetration depth for design at this site is 1.8 m. The proposed depth to the underside of the pile cap will not provide adequate frost protection. It will therefore be necessary to provide alternate frost protection such as insulation.



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 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 side slopes should be protected from erosion and should be inspected regularly for signs of instability. Slopes should be flattened as required to maintain safe working conditions.

Open cut excavations for construction of the proposed wall foundations is not required for the planned wall design. The recommended pile foundations will be incorporated into a shoring system. However, if open cut excavations are to be carried out in close proximity to existing structures, the proposed excavation limits will need to be reviewed by both the Structural and Foundation Engineers to assess the need for restrictions such as staging of the excavation and limitations on stage lengths and excavation time duration. In this case an NSSP would be required.

8.2 Supported Excavations – Shoring

The front face of the proposed retaining structure is 1.9 m in front of the front face of the pile cap beneath the Moodie structure abutment. The centreline of an existing storm sewer (914 mm diameter) is located just 1.5 m from the front face of the proposed wall. The invert elevation of the pipe is approximately 5.4 m below the top of pavement. A cross-section is shown on Figure 4 in Appendix C. Shoring will be required to support the existing embankment fill during construction of the retaining wall. In addition, roadway protection may be required for excavations adjacent to the driving lanes.

The potential for conflict with existing piles and the existing sewer will need to be considered in the shoring design.

The lateral earth pressures provided in Section 7.2 and rock anchor recommendations provided in section 7.3 may be used for the design of the shoring system.



It is anticipated that the shoring to support the existing embankment fill may be left in place or incorporated into the final retaining wall design due to the limited space between the existing abutment and proposed retaining wall alignment.

It is noted that the piles will have minimal resistance immediately after driving due to remoulding of the silty clay deposit. It will therefore be necessary to wait for a period of time after installation of the piles before excavating if the piles are to form part of the shoring system. In general, it is expected that 70% of the axial and lateral load resistance of the piles will be achieved within 2 weeks of completion of pile driving.

8.3 Site Preparation

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

8.4 Dewatering

Dewatering of temporary excavations for pile caps may be required. If required, dewatering of structure excavations should be carried out in accordance with OPSS 902.07.06.

It is expected that dewatering can be achieved using conventional sump and pump techniques.

8.5 Frost Protection

As indicated in Section 6.1 and Section 7.5, the design frost depth for this site is 1.8 m. All foundations should be provided with a minimum of 1.8 m of soil cover or equivalent insulation to protect against frost heave.

Temporary frost protection should be provided to all structures if construction is undertaken during freezing conditions. This requirement applies to existing structures such as pile caps that may be exposed during excavation as well as new structures.



8.6 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

| Location | Borehole | Sample | pH (pH units) | Resistivity (ohm.m) | Soluble Sulphate (:g/g) | Chloride (:g/g) |
|--------------|----------|--------|---------------------|------------------------|-------------------------------|--------------------|
| West Side | BH 04-1 | 4 | 7.82 | 11 | 150 | 610 |
| East Side | BH 04-4 | 9 | 8.86 | 40 | 190 | 50 |

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



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APPENDIX A

Symbols and Terms Used on Borehole and
Test Pit Records

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:

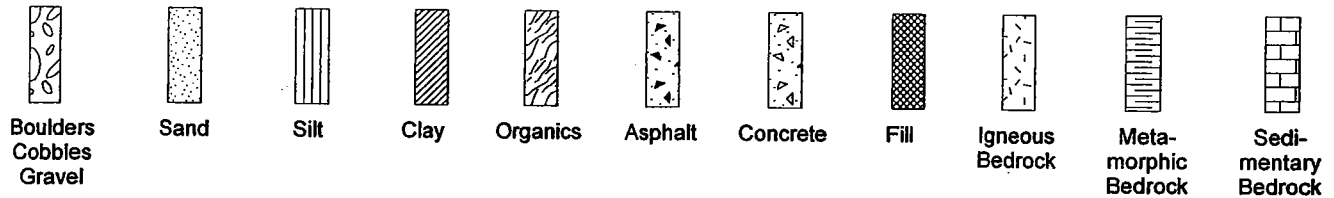
| Strength Classification | Unconfined Compressive Strength (MPa) |
|-------------------------|---------------------------------------|
| <i>Extremely Weak</i> | < 1 |
| <i>Very Weak</i> | 1 – 5 |
| <i>Weak</i> | 5 – 25 |
| <i>Medium Strong</i> | 25 – 50 |
| <i>Strong</i> | 50 – 100 |
| <i>Very Strong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

RECORD OF BOREHOLE No BH04-1

1 OF 1

METRIC

W.P. 302-89-00 LOCATION HWY 417 Moodie Drive Interchange Station 13+995, CL of Retaining Wall ORIGINATED BY DF
 DIST 42 HWY 417 BOREHOLE TYPE HS Augers, Split Spoons COMPILED BY MM
 DATUM Geodetic DATE 04.01.12 - 04.01.12 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 | | | | | | | | | |
| 69.0 | | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 68.9 | 140 mm of ORGANICS/TOPSOIL | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | |
| 0.1 | Silty clay, brown grey (FILL) | | | | | | | | | | | | | | | | |
| 67.7 | | | 1 | SS | 21 | | 68 | | | | | | | | | | |
| 1.3 | Sand, some gravel, trace clay, trace silt (FILL) | | 2 | SS | 11 | | 67 | | | | | | | | | 12 68 | (20) |
| | | | 3 | SS | 1/300 | | 66 | | | | | | | | | | |
| 66.0 | SILTY CLAY, grey, firm | | 4 | SS | 2 | | 65 | | | | | | | | | 46.5 | |
| 3.0 | | | 5 | SS | 2 | | 64 | | | | | | | | | 50.6 | |
| | | | 6 | SS | 3 | | 63 | | | | | | | | | | |
| | | | 7 | SS | 1/300 | | 62 | | | | | | | | | 44.4 | |
| | | | 8 | SS | 2 | | 61 | | | | | | | | | | |
| 60.5 | Silty sand, trace clay, trace gravel, very loose (TILL) | | 9 | SS | 3 | | 60 | | | | | | | | | | 10 44 |
| 8.4 | | | 10 | SS | 1/300 | | 59 | | | | | | | | | | (46) |
| | | | 11 | SS | 4 | | 58 | | | | | | | | | | |
| | | | 12 | SS | 2 | | | | | | | | | | | | |
| 57.6 | Auger Refusal on Inferred Bedrock | | | | | | | | | | | | | | | | |
| 11.4 | Standpipe Installed | | | | | | | | | | | | | | | | |

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

RECORD OF BOREHOLE No BH04-3

1 OF 1

METRIC

W.P. 302-89-00 LOCATION HWY 417 Moodie Drive Interchange Station 14+040 ORIGINATED BY DF
 DIST 42 HWY 417 BOREHOLE TYPE HS Augers, Split Spoons COMPILED BY MM
 DATUM Geodetic DATE 04.01.13 - 04.01.13 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 | | | | | | |
| 68.9 | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | |
| 68.4 | 60 mm of ORGANICS/TOPSOIL | | | | | | | | | | | | | | |
| 68.2 | Sand, some gravel, trace silt, brown (FILL) | | | | | | | | | | | | | | |
| 68.2 | Gravelly sand, trace silt, brown (FILL) | | 1 | SS | 93 | | 68 | | | | | | | | |
| | | | 2 | SS | Refusa | | 67 | | | | | | | | |
| | | | 3 | SS | Refusa | | 66 | | | | | | | | |
| | | | 4 | SS | 11 | | 65 | | | | | | | | |
| 65.0 | SILTY CLAY, grey, firm | | 5 | SS | 1/300 | | 64 | 8.5 | | | | | | | |
| 3.9 | | | 6 | SS | 1/300 | | 63 | 8.5 | | | | | | | |
| | | | 7 | SS | 8 | | 62 | 0.6 | | | | | | | |
| | | | 8 | SS | 2 | | 61 | 8.5 | | | | | | | |
| | | | 9 | SS | 2 | | 60 | 10.6 | | | | | | | |
| | | | 10 | SS | 3 | | 59 | 4.2 | | | | | | | |
| 59.4 | Silty sand, trace clay, trace gravel, grey, loose (TILL) | | 11 | SS | 7 | | 58 | | | | | | | | |
| 9.5 | | | 12 | SS | 9 | | 57 | | | | | | | | |
| 58.2 | Sandstone BEDROCK | | 13 | RC | 88% | | 56 | | | | | | | | |
| 10.7 | | | 14 | RC | 92% | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| 55.2 | End of Borehole | | | | | | | | | | | | | | |
| 13.7 | Standpipe Installed | | | | | | | | | | | | | | |

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

MT0 11674.GPJ ON MOT.GDT 06/12/19

RECORD OF BOREHOLE No BH04-4

1 OF 1

METRIC

W.P. 302-89-00 LOCATION HWY 417 Moodie Drive Interchange Station 14+045 ORIGINATED BY DF
DIST 42 HWY 417 BOREHOLE TYPE HS Augers, Split Spoons COMPILED BY MM
DATUM Geodetic DATE 04.01.12 - 04.01.12 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 | | | | | | | | WATER CONTENT (%) | | |
| | | | | | | | | ○ UNCONFINED | × FIELD VANE | ● QUICK TRIAXIAL | | | | | | | × LAB VANE | |
| 68.9 | | | | | | | 20 | 40 | 60 | 80 | 100 | 10 | 20 | 30 | | | | |
| 68.9 | 80 mm of ORGANICS/TOPSOIL Silty sand, trace clay, trace gravel, brown (FILL) | | 1 | SS | 18 | | | | | | | | | | | | | |
| | | | 2 | SS | 5 | | | | | | | | | | | | | |
| 66.4 | SILTY CLAY, grey, firm | | 3 | SS | 3 | | | | | | | | | 51.6 | | | | |
| 2.4 | | | 4 | SS | 2 | | | | | | | | | 48.2 | | | | |
| | | | 5 | SS | 4 | | | | | | | | | | | | | |
| | | | 6 | SS | 3 | | | | | | | | | | | | | |
| | | | 7 | SS | 3 | | | | | | | | | 47 | | | | |
| | | | 8 | SS | 2 | | | | | | | | | 44.5 | | | | |
| | | | 9 | SS | 1/300 | | | | | | | | | 40.5 | | | | |
| 59.4 | Silty sand, trace clay, some gravel, grey, very dense (TILL) | | 10 | SS | 63 | | | | | | | | | | | | | |
| 9.5 | | | | | | | | | | | | | | | | | | |
| 58.4 | Auger Refusal on Inferred Bedrock | | | | | | | | | | | | | | | | | |
| 10.5 | | | | | | | | | | | | | | | | | | |

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

UNIFIED SOIL CLASSIFICATION SYSTEM

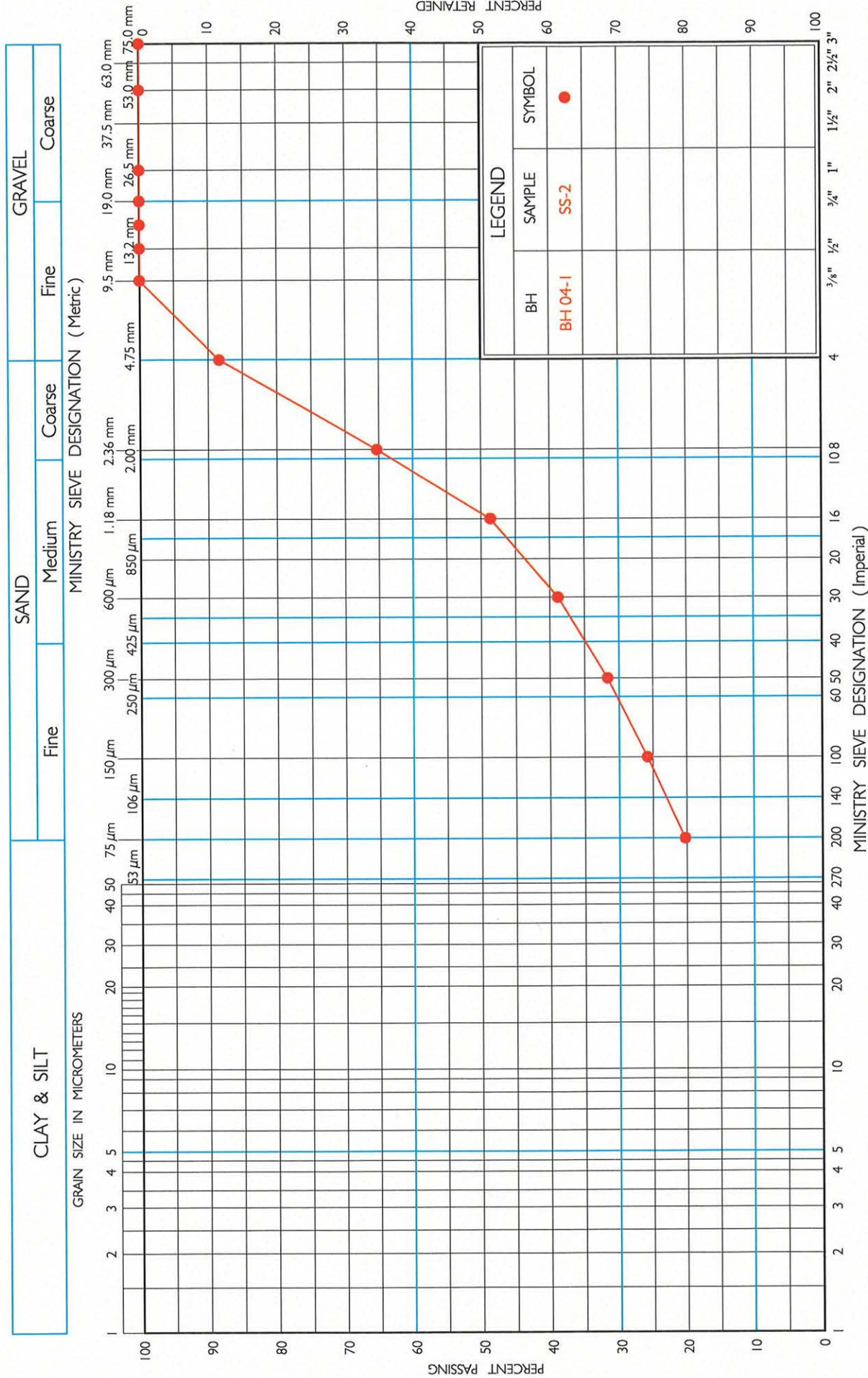


FIG No 1

GRAIN SIZE DISTRIBUTION

SILTY SAND, SOME GRAVEL (FILL)

UNIFIED SOIL CLASSIFICATION SYSTEM

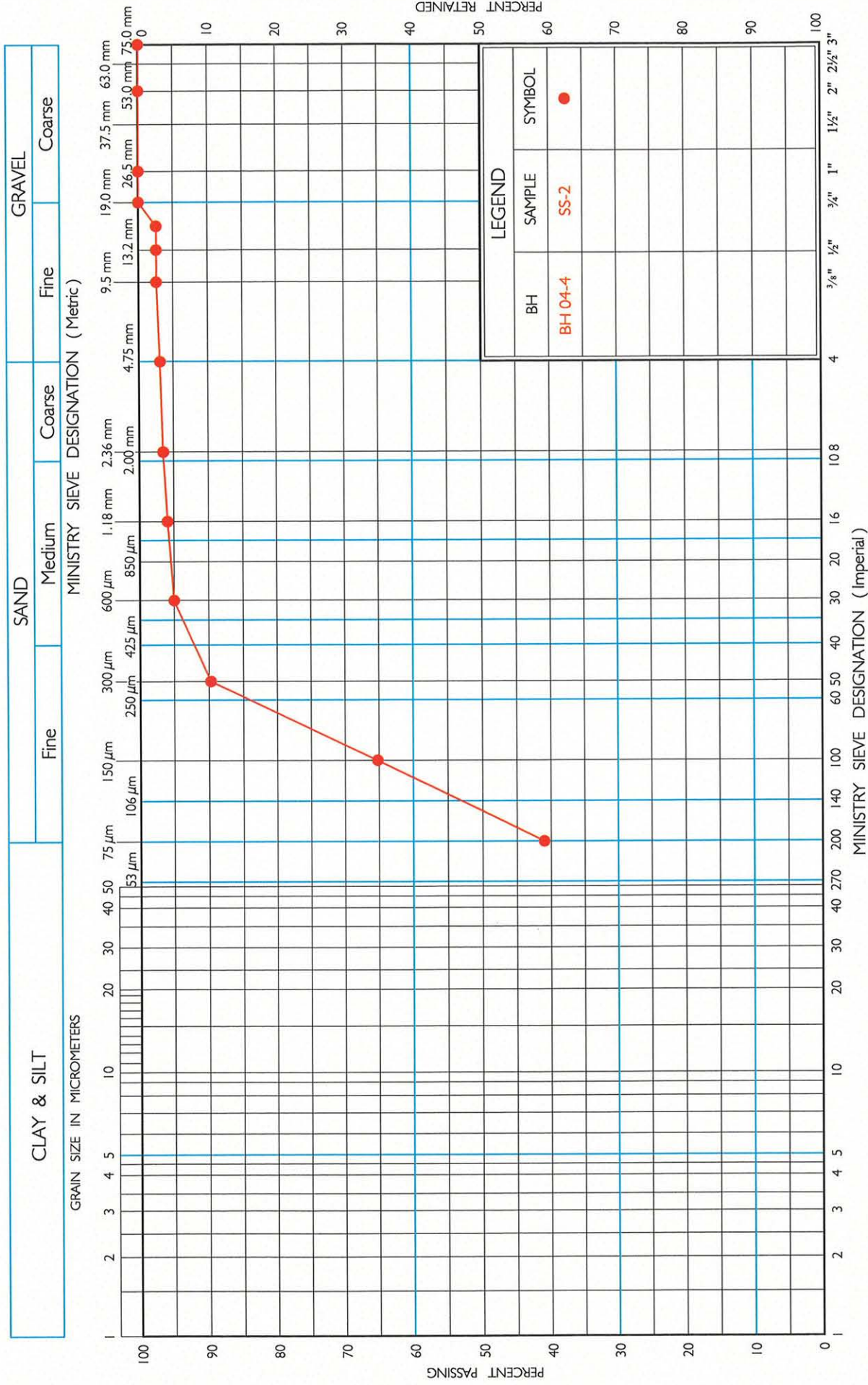


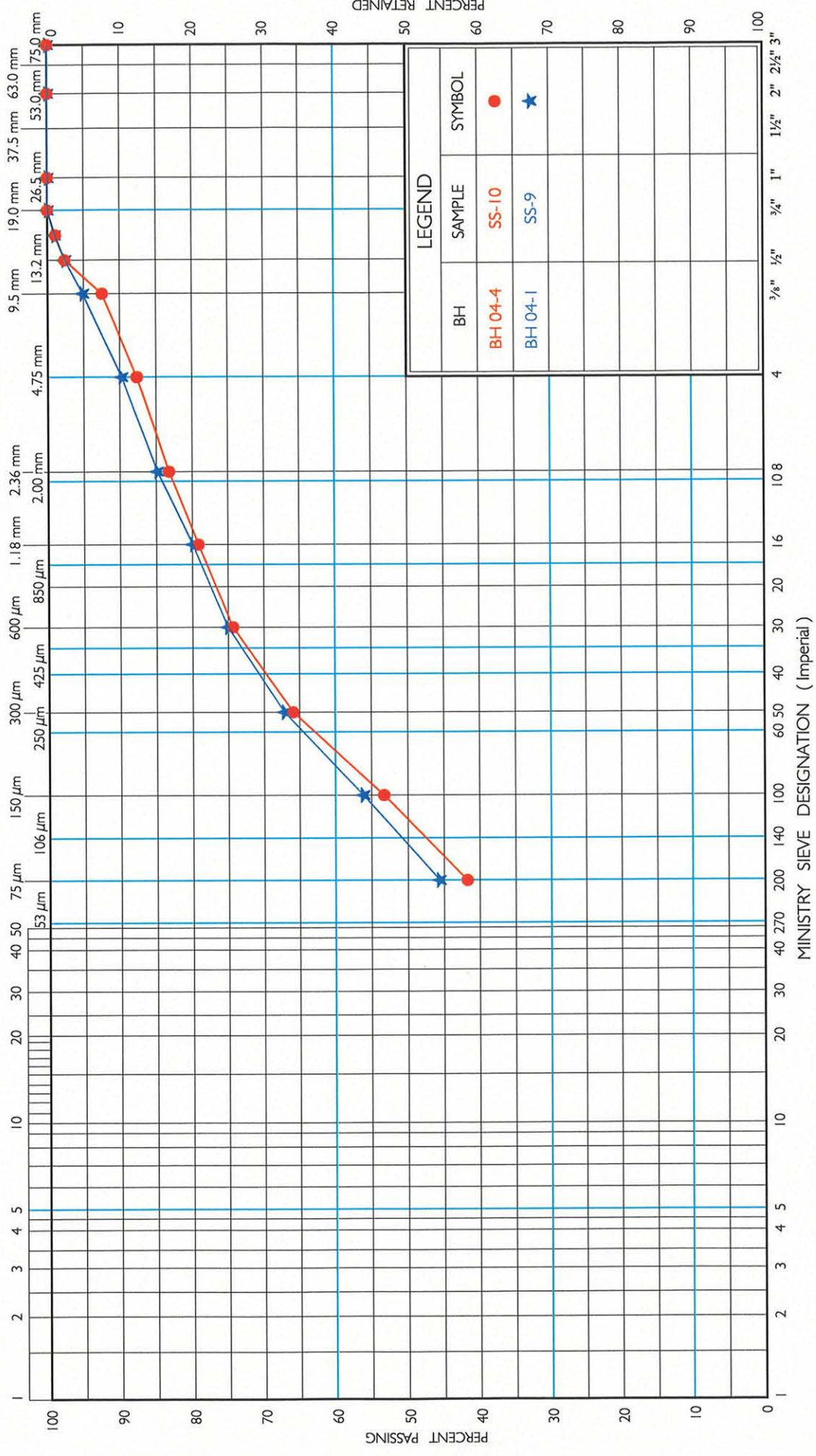
FIG No 2
GRAIN SIZE DISTRIBUTION
SILT AND SAND, TRACE GRAVEL (FILL)

UNIFIED SOIL CLASSIFICATION SYSTEM

| CLAY & SILT | | | SAND | | | GRAVEL | | |
|-------------|--|--|--------|--|--|--------|--|--|
| Fine | | | Medium | | | Fine | | |

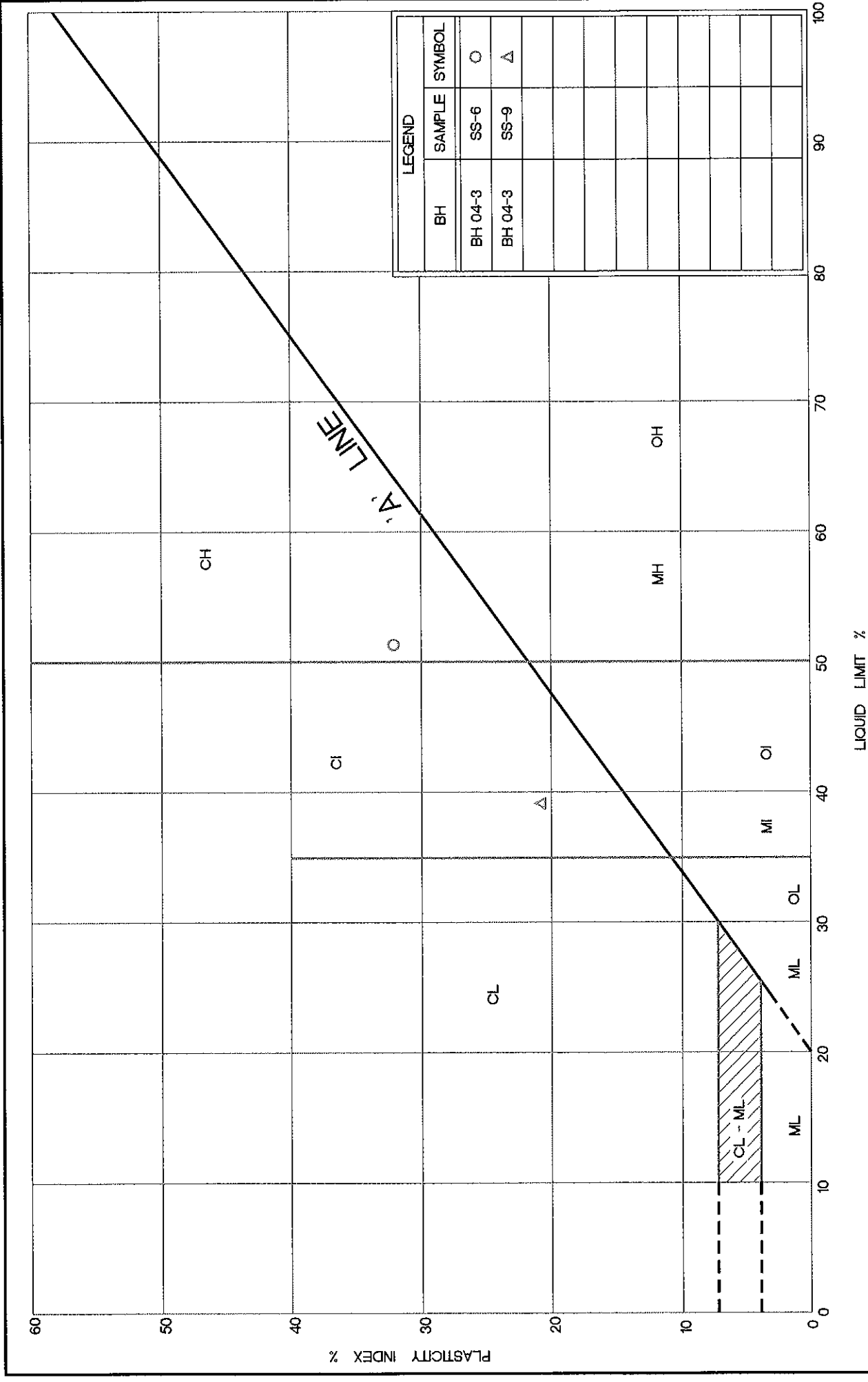
GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



| BH | SAMPLE | SYMBOL |
|----------|--------|--------|
| BH 104-4 | SS-10 | ● |
| BH 104-1 | SS-9 | ★ |

FIG No 3
GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME GRAVEL (TILL)

Ministry
of
Transportation
Ontario

PLASTICITY CHART

FIG No 4

WP 302-89-00



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

| Borehole # | Sample # | Recovery (%) | R.Q.D. (%) | Description |
|------------|----------|--------------|------------|---|
| BH 04-2 | 12 | 88 | 94 | SANDSTONE, white to grey, unweathered; closely spaced fractures: planar to stepped, rough very thin bedding |
| | 13 | 100 | 87 | |
| BH 04-3 | 13 | 100 | 88 | SANDSTONE, white to grey, unweathered; closely spaced fractures: planar to stepped, rough very thin bedding |
| | 14 | 100 | 92 | |

P:\2006\10000\11674 - Hwy 417 Widening\Moodie Walls\Rock Core Summary Table.xls

APPENDIX B

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

T:\Autocad\Drawings\Project Drawings\2006\10000\11674\Retaining Wall\11674-1 (RW).dwg PRINTED: Dec 19, 2006

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

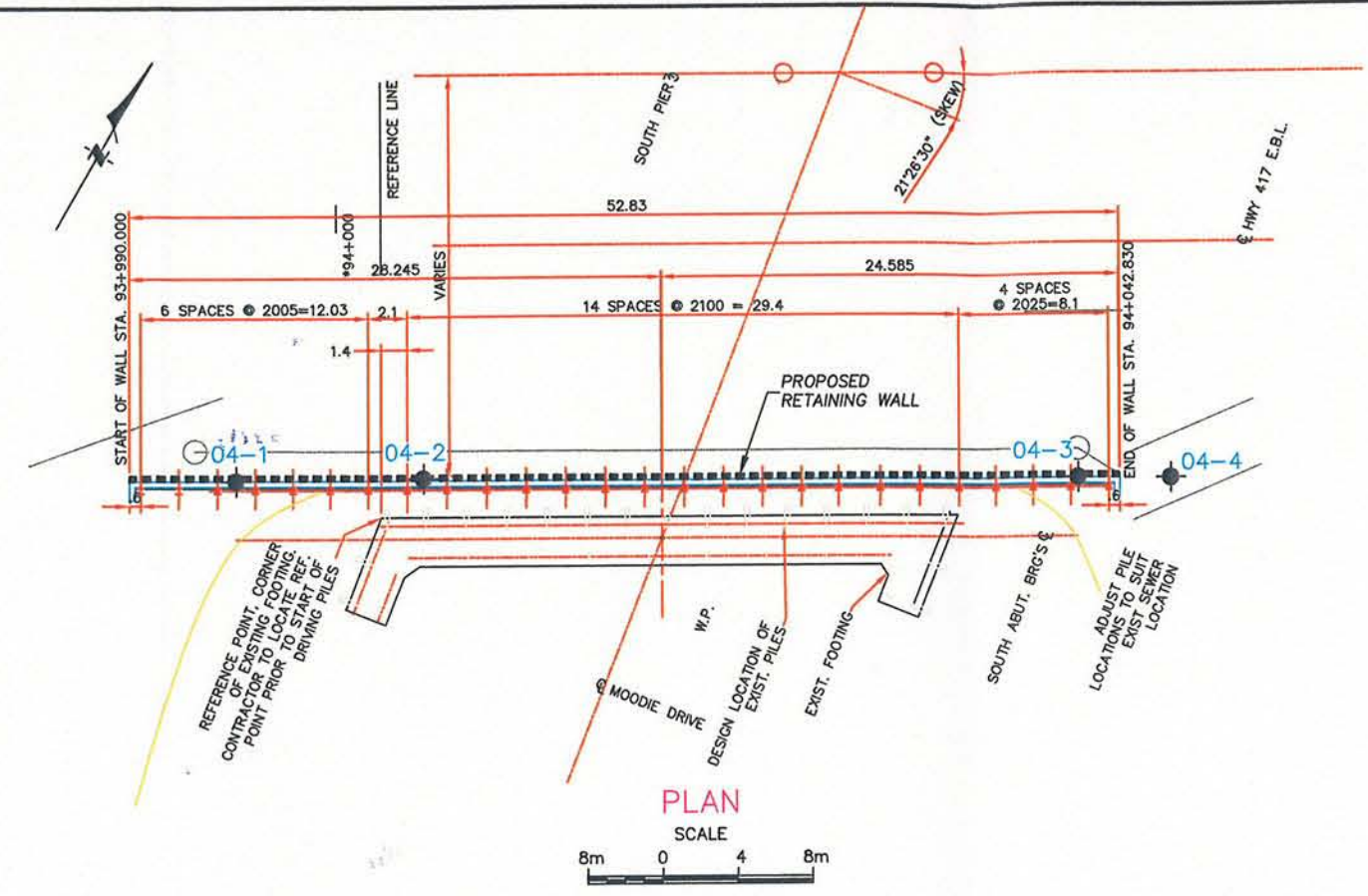
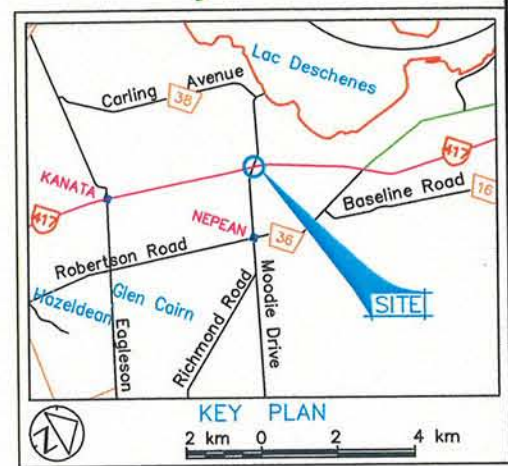
CONT No
WP No 302-89-00

OTTAWA QUEENSWAY
MOODIE DRIVE OVERPASS
SOUTH RETAINING WALL
BORE HOLE LOCATIONS & SOIL STRATA

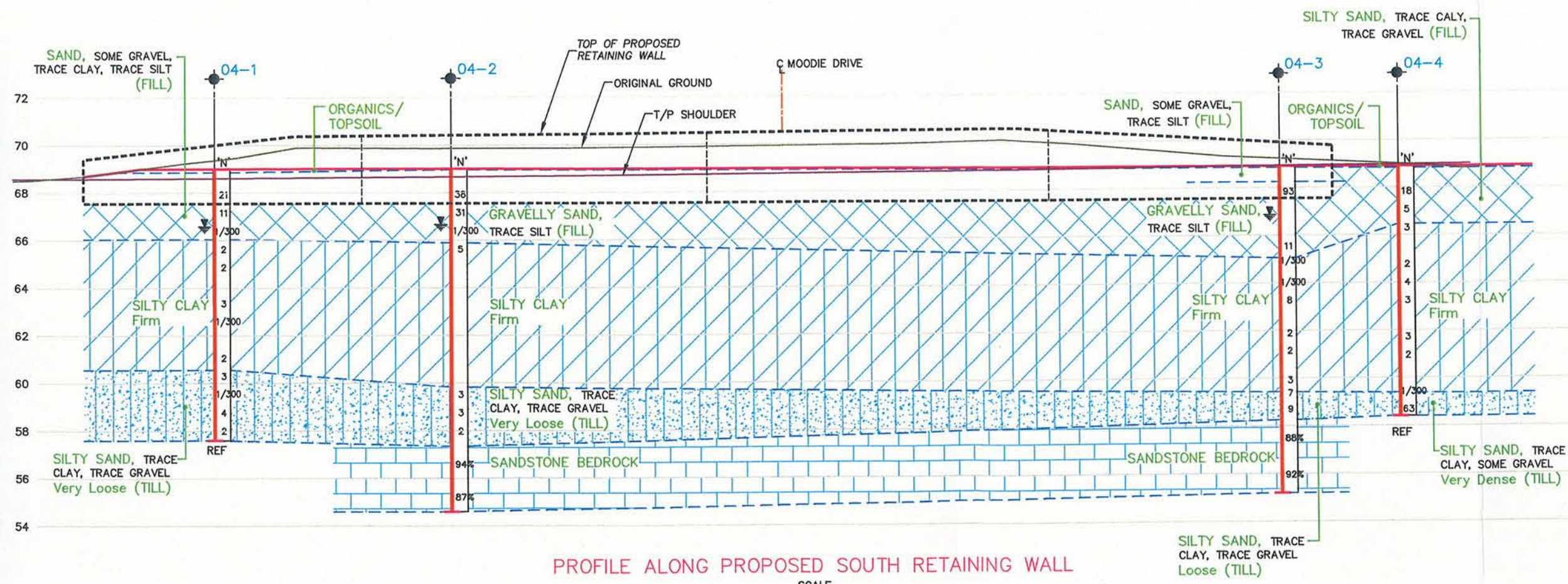


SHEET
—

Jacques Whitford



PLAN
SCALE
8m 0 4 8m



PROFILE ALONG PROPOSED SOUTH RETAINING WALL

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 Dec 2004
- WL in Piezometer
- Piezometer

| No | ELEVATION | COORDINATES | |
|------|-----------|-------------|-----------|
| | | NORTH | EAST |
| 04-1 | 69.0 | 5 022 221.1 | 356 409.0 |
| 04-2 | 69.0 | 5 022 226.2 | 256 417.5 |
| 04-3 | 68.9 | 5 022 243.9 | 356 447.8 |
| 04-4 | 68.9 | 5 022 246.3 | 356 452.1 |

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 | | DESCRIPTION | |
|--------------------|---------|--------------------|---------------|
| DATE | BY | | |
| | | | |
| | | | |
| | | | |
| GEOCRE No 3165-203 | | | |
| HWY No 417 | | DIST 9 | |
| SUBM'D PC | CHECKED | DATE 2006-12-19 | SITE 3-256 |
| DRAWN GBB | CHECKED | APPROVED <i>PC</i> | DWG NO11674-1 |

APPENDIX C

Photographs
Sketch – Cross Section

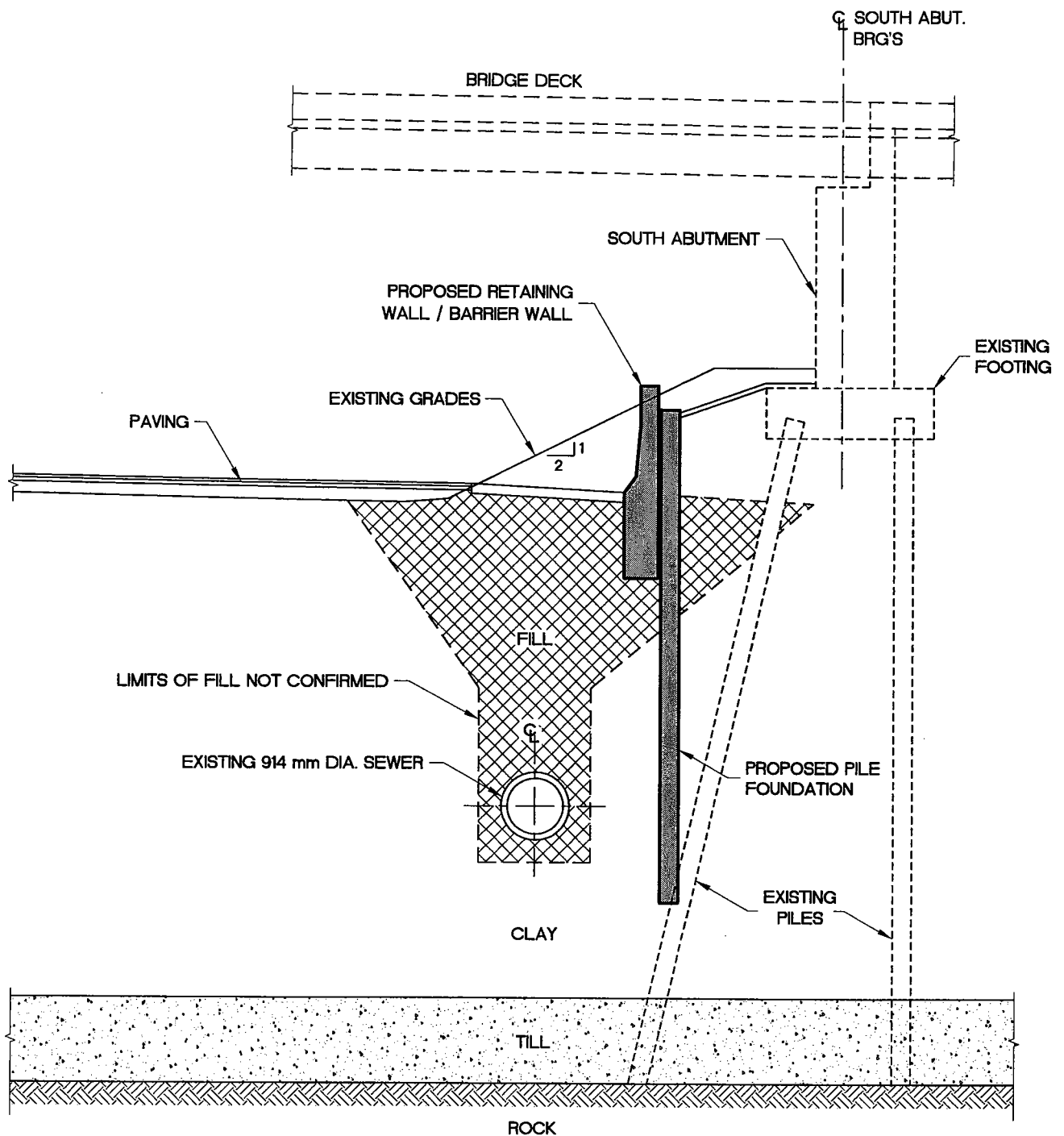




Photo 1: Looking east along slope paving. Note cracking and faulting.



Photo 2: Erosion of soil at east edge of slope paving and resulting settlement and cracking of concrete.



NOTE: LOCATIONS OF EXISTING FEATURES ARE APPROXIMATE.

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A JACQUES WHITFORD LIMITED REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

CROSS-SECTION

HIGHWAY 417 / MOODIE DRIVE SOUTH RETAINING WALL
OTTAWA, ONTARIO

Job No.: NO11674

Scale: N.T.S.

Date: 07/11/20

Dwn. By: GBB

App'd By: *PC*

Fig. No.:

5

