

FOUNDATION INVESTIGATION  
AND DESIGN REPORT

PROPOSED REPLACEMENT/EXTENSION OF  
STRUCTURAL CULVERT 12-419-C  
REHABILITATION OF  
HIGHWAY 21 FROM ST. JOSEPH TO BAYFIELD

G.W.P. 406-94-00  
Agreement # 3006-E-0095



I.E.  
Group

Ministry of Transportation/Stantec Consulting Ltd.  
G.W.P. 406-94-00  
Rehabilitation of Highway 21 from St. Joseph to Bayfield  
Agreement # 3006-E-0095

08-1-IEG2-12-419-C  
Final Report  
Appendix A  
November 20, 2009

## Appendix A

### Explanation of Terms Used in Report

#### Record of Borehole Sheet

#### Boreholes 12-419-C1 to C8

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**Prepared for:**

Stantec Consulting Ltd.  
49 Frederick Street  
Kitchener, Ontario  
N2H 6M7

Mr. Gordon Murray, P.Eng.

**Prepared by:**

Infrastructure Engineering Group Inc.  
39-69 Bessemer Road  
London, Ontario  
N6E 2V6

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## **PART A – FOUNDATION INVESTIGATION**

### **1.0 INTRODUCTION**

This report presents the results of a foundation investigation carried out on June 9 and 11, 2008 by Infrastructure Engineering Group Inc. (IEG) on behalf of Stantec Consulting Ltd. (Stantec).

This assignment involves the rehabilitation of Highway 21 from 0.20 km south of Huron County Road 84 (Zurich/Hensall Road, St. Joseph) northerly to 0.28 km south of Huron County Road 3 (Mill Street, Bayfield), including the rehabilitation of the Bayfield River Bridge; Length = 15.92 km.

It includes the replacement/rehabilitation/extension of thirteen (13) existing structural culverts and the Bayfield River Bridge. The project also includes resurfacing, pavement widening, intersection realignments, intersection improvements, minor horizontal and vertical alignment improvements and electrical work.

Foundation investigation and recommendations are required for the design and construction of the replacement/rehabilitation/extension of thirteen (13) existing structural culverts as part of the improvement of Highway 21. This report covers the site of Structure 12-419-C.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes and, based on the findings, to provide geotechnical recommendations for the foundation elements. Partial or full replacement of the culvert may be required pending on the results of the culvert inspection specified under Section 6.3.1 of the RFP document.

Authorization to complete this assignment was given by Mr. Dan Green, P. Eng., of Stantec Consulting Ltd., the TPM Consultant who is completing this assignment for MTO under Agreement # 3006-E-0095.

### **2.0 SITE DESCRIPTION**

#### **2.1 Site Location**

Structure 12-419-C is located on Highway 21, approximately 15.72 km north of the south limit of the project (Station 19+060), located at Station 21+400. The transition in chainage is located between the boundaries of Townships of Hay and Stanley (23+380.613=10+000). Photographs of this culvert site are presented in Appendix "D".

The existing structure is a reinforced concrete, rigid frame box culvert with a span of 3.66 m, a height of 1.22 m and a length of 41.50 m, with an overfill height of approximately 3.5 m, and at a skew of 37° from the centerline of Highway 21. The culvert opening dimensions were provided in the RFP documents and the base plans provided by Stantec.

The culvert site is located within a drainage valley in which the stream flows westerly. The road embankments were built on both the south and north sides of the culvert, with a maximum height of approximately 4.7 m. The embankment slopes are typically 2.5H to 3H:1V and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

A grey silty clay to clayey silt till deposit was noted at the streambed. There was approximately 0.5 m of water running in the creek during our field work between June 9 and 11, 2008.

## **2.2 Physiography and Topography**

The thirteen (13) culvert sites investigated for this project are located within the physiographic region referred to as the Huron Slope (Chapman and Putnam, 1984), which runs along the east side of Lake Huron between Sarnia and Tobermory and is situated between the Algonquin shorecliff and the Wyoming Moraine. The area is characterized by a relatively flat topography, heavy textured soil and poor drainage. The surficial deposits consist of brown, calcareous clayey tills which contain very few cobbles and boulders. The tills are known to be underlain by grey stratified clays of lacustrine origin.

The asphalt pavement surface over the existing culvert is near Elevation 205.7 m, while the ground surface at the base of the embankment and adjacent to the stream is located approximately Elevation 201.0 m.

## **3.0 INVESTIGATION PROCEDURES**

### **3.1 Field Investigation**

Between June 9 and 11, 2008, a track-mounted Morooka drill rig was supplied by London Soil Test Ltd. and used on site for drilling and Standard Penetration Testing (SPT, following the procedures of ASTM D 1586). Eight (8) boreholes were drilled and sampled to obtain data for foundation design of the proposed culvert replacement/extension and potential embankment widening. The locations of the boreholes are shown on Drawing 1.

The boreholes were numbered 12-419-C1 to C8 for the subject culvert and the depths of sampling were as follows:

<b>Borehole No.</b>	<b>Depth of Sampling (m)</b>
12-419-C1	7.32
12-419-C2	11.13
12-419-C3	8.08

Borehole No.	Depth of Sampling (m)
12-419-C4	4.27
12-419-C5	5.79
12-419-C6	11.13
12-419-C7	5.79
12-419-C8	4.27

Boreholes 12-419-C1, C2, and C3 were put down for the culvert rehabilitation/replacement. Boreholes 12-419-C4, C5, C6, C7 and C8 were put down at 50 m spacing, spanning between 100 m north and south of the culvert for the potential of embankment widening for temporary detour during construction. Since completion of the field work, it has been decided by the project team that embankment widening will no longer be required. Borehole 12-419-C6, located near the centerline of the culvert, will be used along with Boreholes 12-419-C1, C2, and C3 in establishing the soil profile presented in Drawing 1. The remaining borehole data are provided as geotechnical data only for future use, if required.

The boreholes were drilled using continuous flight solid stem augers. Soil samples were retrieved at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT). Samples were generally taken at intervals of depth of 0.75 m to the maximum depth of exploration.

Field pocket penetrometer was used on the retrieved SPT samples, where applicable, to determine the undrained shear strength of the cohesive soil deposits. It is noted that the measured shear strength value would be slightly lower than the actual value due to sampling disturbance.

Seepage and water levels were noted in each borehole during and at the completion of drilling and sampling. All boreholes were grouted with a bentonite/cement mix at completion of sampling in accordance with Ontario Regulation 903.

Our field engineer, Mr. Ralph Billings, P. Eng., supervised the fieldwork and worked under the direction of the project engineer, Mr. Eric Chung, P. Eng. Our field staff cleared the location of buried utilities and logged the boreholes. The soil samples obtained were placed in labeled containers and transported to IEG's London laboratory for further examination and laboratory testing.

The stations, offsets and ground surface elevations at the as drilled borehole locations were surveyed by Stantec Consulting Ltd. and provided to IEG for the purpose of this report.

The results of the drilling, sampling, in-situ testing and groundwater observations are summarized on the Record of Borehole sheets and enclosed in Appendix "A".

### **3.2 Laboratory Analysis**

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all retrieved soil samples. In addition, grain size analyses, Atterberg Limit tests and unit weight tests were performed on selected samples.

The results of the laboratory testing are presented on the Record of Borehole sheets (Appendix "A"), and Laboratory Test Results (Figures 1 to 9, Appendix "B").

## **4.0 SUBSURFACE CONDITIONS**

### **4.1 General Subsurface Conditions**

Reference is made to the Record of Borehole sheets (Appendix "A") and Laboratory Test Results (Appendix "B") for detailed subsurface soil and groundwater conditions encountered in the boreholes. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, consequently, represent transitions between soil types rather than exact planes of geological change. The soil profiles depicting the subsurface conditions on Drawing 1 will vary between and beyond the borehole locations.

It should be noted that only Boreholes 12-419-C1, C2, C3 and C6 are being used to formulate the soil profile and the summarized subsurface conditions at the culvert location. The remaining boreholes and laboratory data are provided as geotechnical data for future use, if required.

In general, the subsurface deposits at the site consist of very loose to compact embankment fill placed on a major stratum of stiff to hard, grey silty clay to clayey silt till with intermittent layers of compact to dense sand and silt.

#### **4.1.1 Fill, Topsoil and Organic Silt**

Boreholes 12-419-C2 and C6 were put down on the shoulders and encountered shoulder gravel in the order of 610 mm. The shoulder gravel is underlain by a 3.05 and 4.27 m thick layer of sand fill with a trace of silt and gravel, and silty pockets. Traces of organics were found within the bottom 0.6 m of the sand fill in Borehole 12-419-C2 and peaty layers with silty sand pockets were encountered within the bottom 1.0 m of the sand fill in Borehole 12-419-C6. The sand fill in Borehole 12-419-C2 is underlain by a 1.52 m thick layer of mixed fill consisting organics, sand, silt and silty clay layers.

The ground surface of Borehole 12-419-C1 is covered with a 300 mm thick layer of organic topsoil. The ground surface of Borehole 12-419-C3 is covered with a 300 mm thick layer of organic topsoil underlain by a 540 mm thick layer of organic silt.

Five (5) grain size distributions of the embankment fill, topsoil and organic silt layers are shown on Figure 1 of Appendix "B".

Standard penetration tests yielded "N"-values from 0 to 22 blows per 0.3 m. This fill is brown to grey in colour and the measured natural moisture contents range from 5 to 61%.

Based on the above field and laboratory test results, together with visual and tactile examination, the fill, topsoil and organic silt layers exhibited generally very loose to compact compactness condition.

#### **4.1.2 Near Surface Silt**

The organic silt in Borehole 12-419-C3 is underlain by a 0.53 m thick layer of silt. A single grain size distribution analysis was carried out on the near surface silt and the results are presented in Figure 2 of Appendix "B".

A single standard penetration test yielded an "N"-value of 16 blows per 0.3 m. The near surface silt is brown to grey mottled in colour and the measured natural moisture content of a single sample is 18%.

Based on the above field and laboratory test results, together with visual and tactile examination, the near surface silt layer exhibited a compact compactness condition.

#### **4.1.3 Clayey Silt to Silty Clay**

The upper silt layer in Borehole 12-419-C3 is underlain by a 1.53 m thick layer of clayey silt to silty clay, with localized frequent silt and clay layers. A single grain size distribution analysis was carried out on the clayey silt to silty clay and the results presented in Figure 3 of Appendix "B".

A single Atterberg Limits determination yielded a Plastic Limit, Liquid Limit and Plasticity Index of 15%, 20% and 5%, respectively, and the results are presented in Figure 4 of Appendix B".

Two standard penetration tests yielded "N"-values of 14 and 17 blows per 0.3 m. The clayey silt to silty clay is grey in colour and the measured natural moisture contents of two samples are 16 and 23%.

A single unit weight determination carried out on the silty clay to clayey silt yielded a value of 21.0 kN/m<sup>3</sup>.

Based on the above field and laboratory test results, together with visual and tactile examination, the clayey silt to silty clay exhibited a stiff to very stiff consistency.

#### **4.1.4 Upper Silt to Sandy Silt**

The fill layers in Boreholes 12-419-C2 and C6 and the organic silt layer in Borehole 12-419-C3 are underlain by an upper layer of silt to sandy silt, with occasional silty clay seams and layers. The thickness of the upper silt to sandy silt are 2.29 m, 2.28 m and 1.37 m in Boreholes 12-419-C2, C3 and C6, respectively; with bottom elevations of 197.98 m, 197.28 m and 199.47 m, respectively.

Two (2) grain size distribution analyses were carried out on the upper silt to sandy silt and the results are presented in Figure 5 of Appendix "B".

A single Atterberg Limits determination yielded a Plastic Limit, Liquid Limit and Plasticity Index of 12%, 16% and 4%, respectively, and the results are presented in Figure 6 of Appendix "B".

Three (3) unit weight determinations carried out on the upper silt to sandy silt yielded values between 21.6 and 21.8 kN/m<sup>3</sup>.

Standard penetration tests yielded "N"-values from 11 to 20 blows per 0.3 m. Natural moisture contents of the upper silt to sandy silt ranged from 9 and 15%.

Based on the above field and laboratory test results, together with visual and tactile examination, the upper silt to sandy silt exhibited a compact compactness condition.

#### **4.1.5 Silty Clay Till**

The topsoil in Borehole 12-419-C1 and the upper silt to sandy silt in Boreholes 12-419-C2, C3 and C6 are underlain by a silty clay till deposit. The silty clay till deposit in Boreholes 12-419-C1, C3 and C6 extends beyond the vertical limits of the boreholes at a maximum depth of 11.13 m below the present ground surface, elevation 194.59 m. The silty clay till in Borehole 12-419-C2 is 1.5 m thick and further underlain by a lower silt deposit.

Six (6) grain size distribution analyses were carried out on the silty clay till and the results presented in Figure 7 of Appendix "B".

Six (6) samples were tested and exhibited the following Atterberg Limits. These results are shown in Figure 8 of Appendix "B" and summarized below:

Liquid Limit ( $W_L$ )	24 to 26%, average at 25.2%
Plastic Limit ( $W_P$ )	14 to 16%, average at 14.4%
Plasticity Index ( $I_p$ )	9 to 12%, average at 10.8%

The natural moisture contents were in the range of 10 to 22%. These results are characteristic of clayey soils of low plasticity (CL). The measured natural moisture contents are generally



between the measured plastic and liquid limits with localized lower values below the Plastic Limit, and indicate that the deposit is pre-consolidated.

The unit weight of the silty clay till was determined on seven (7) samples, and yielded values in the range of 20.4 to 22.7 kN/m<sup>3</sup> with an average of 21.9 kN/m<sup>3</sup>.

Undrained shear strength as determined from field pocket penetrometer on retrieved SPT samples ranged from 75 to over 300 kPa. Standard penetration tests yielded "N"-values from 11 to 48 blows per 0.3 m.

Based on the above field and laboratory test results, together with visual and tactile examination, the silty clay till deposit exhibited generally stiff to hard consistency.

#### **4.1.6 Lower Silt**

The silty clay till in Borehole 12-419-C2 is further underlain by a lower silt deposit which extends beyond the vertical limit of the borehole at a maximum depth of 11.13 m below the present ground surface, Elevation 194.32 m.

Standard penetration tests yielded "N"-values from 24 to 57 blows per 0.3 m. Natural moisture contents of the upper silt to sandy silt ranged from 13 and 15%.

A single grain size distribution analysis was carried out on the silty clay seams within the silt to sandy silt layer and the results presented in Figure 9 of Appendix "B".

Based on the above field and laboratory test results, together with visual and tactile examination, the lower silt deposit exhibited generally compact to very dense "compactness condition".

#### **4.2 Groundwater Conditions**

The groundwater condition was monitored during and upon completion of sampling.

Water level was measured in Borehole 12-419-C3 at a depth of 0.7 m below the existing ground surface (Elevation 201.75 m). Wet cave-in was observed at a depth of 2.75 m below the present ground surface of Borehole 12-419-C6 (Elevation 202.97 m). Slow water ingress was observed in Borehole 12-419-C2 at a depth of 5.1 m (Elevation 200.35 m), near the native/fill interface. Borehole 12-419-C1 was bored dry and remained dry and open for the short duration that the borehole was kept open.

The water level in the creek was approximately 0.5 m above the creek bottom at the time of field investigation (between June 9 and 11, 2008) and reflected a low flow condition.

It should be noted that the groundwater level will fluctuate seasonally and in response to weather events. Under adverse conditions, water could be perched within the embankment fill and on top

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of the silty clay till. It is reasonable to assume that groundwater could be similar to the water level in the creek during high flow conditions.

## **PART B – FOUNDATION DESIGN**

### **5.0 DISCUSSION AND RECOMMENDATIONS**

#### **5.1 General**

This section of the report provides our recommendations on the geotechnical aspects of foundation design of the proposed replacement/rehabilitation/extension of Structure 12-419-C, based on our interpretation of the factual information obtained during this investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

Structure 12-419-C is located on Highway 21, approximately 15.72 km north of the south limit of the project (Station 19+060), located at Station 21+400. The transition in chainage is located between the boundaries of Townships of Hay and Stanley ( $23+380.613=10+000$ ). Photographs of this culvert site are presented in Appendix "D".

The existing structure is a reinforced concrete, rigid frame box culvert with a span of 3.66 m, a height of 1.22 m and a length of 41.50 m, with an overfill height of approximately 3.5 m, and at a skew of  $37^\circ$  from the centerline of Highway 21. The culvert opening dimensions were provided in the RFP documents and the base plans provided by Stantec.

The culvert site is located within a drainage valley in which the stream flows westerly. The road embankments were built on both the south and north sides of the culvert, with a maximum height of approximately 4.7 m. The embankment slopes are typically 2.5H to 3H:1V and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

A grey silty clay to clayey silt till deposit was noted at the streambed. There was approximately 0.5 m of water running in the creek during our field work between June 9 and 11, 2008.

The asphalt pavement surface over the existing culvert is near elevation 205.7 m, while the ground surface at the base of the embankment and adjacent to the stream is located approximately at Elevation 201.0 m.

In general, the subsurface deposits at the site consist of very loose to compact embankment fill placed on a major stratum of stiff to hard, grey silty clay to clayey silt till with intermittent layers of compact to dense sand and silt.

The Preliminary PDR recommended culvert rehabilitation, subject to Stantec's culvert inspection report. The replacement culvert, if required, will consist of a precast concrete box culvert.

Alternatively, the replacement culvert could be constructed as a rigid frame, open-footing culvert which will be over-built to encompass the existing culvert. This alternative will allow working in the dry and removal of the existing culvert after completion of the new culvert.

It is understood from Stantec that the final culvert design involves replacement of Culvert 12-419-C with a precast concrete box culvert along with construction of RSS walls, wing walls and/or gabion walls to support the existing roadway embankment.

## 5.2 Closed Box Culvert

The soils encountered at the subject site are considered suitable for the support of a box culvert foundation. Results of all boreholes put down along the proposed culvert alignment indicate that the founding subgrade consists of compact silt to sandy silt over stiff to hard silty clay till.

The culvert should be designed to CAN/CSA-S6-06 and to withstand the appropriate weight of overfill, traffic loadings (CL-625-ONT), temporary construction loads and critical loading effects during construction. If the base slab does not have adequate frost cover/protection, it should be designed for frost pressures.

The invert of the existing culvert is located approximately between Elevations 200.95 m and 200.98 m. Based on the assumption of a 0.3 m thick base slab, and a 0.4 m thick layer of culvert bedding, the bottom of the culvert bedding will be placed approximately between Elevations 200.25 m and 200.28 m.

Based on the borehole results, the box culvert should be designed to bear on the native, undisturbed compact silt to sandy silt, stiff silty clay to clayey silt or stiff silty clay till at the elevation and bearing resistances shown below:

Borehole	Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
12-419-C1	200.2	250	150
12-419-C2	200.2	250	150
12-419-C3	200.2	250	150
12-419-C6	200.2	250	150

The SLS value given above is based on a maximum settlement of 25 mm. This can be achieved provided the founding subgrade is not disturbed during construction.

Minor dewatering with strategically located sumps and trenches will likely be required to facilitate foundation construction.

As per CAN/CSA-S6-06, Clause 1.9.5.6, a cut-off wall of sufficient depth and strength shall be provided at the ends of the culvert to prevent undermining. The depth of the cut-off wall should be designed cognizant of the hydraulic condition (CAN/CSA-S6-06, Section 1.9) and the frost depth of 1.2 m (OPSD 3090.101).

Foundation preparation for any cast-in-place construction should be carried out in accordance with Sub-section 902.07.05.02 of OPSS 902 and Sub-section 902.07.02.02 of SSP902S01. Under wet weather and or site condition, the sandy silt to silt and silty clay to clayey silt subgrade could easily be disturbed. In this regard, a 50 mm thick layer of lean concrete should be placed on the subgrade immediately after subgrade preparation to protect its integrity under wet conditions.

Conventional bedding requires a minimum 300 mm thick OPSS Granular A bedding (Group I, SW-GW, Table 1 OPSS 422) and a 75 mm thick levelling granular course as per OPSS422.07 should be placed on the prepared subgrade to achieve a uniform support for a precast concrete culvert. The Granular A layer should be placed in 150 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD). The levelling course should be uncompacted and consist of OPSS 1002 fine aggregates (concrete sand).

### 5.3 Open Footing Culvert (Spread Footing Foundations)

Based on the borehole results, spread footings may be used for the culvert walls, headwalls (wingwalls) and retaining walls, and designed to bear on the native, undisturbed compact silt to sandy silt, stiff silty clay to clayey silt or stiff silty clay till at the elevation and bearing resistances shown below:

Borehole	Highest Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
12-419-C1	200.2	250	150
12-419-C2	200.2	250	150
12-419-C3	200.2	250	150
12-419-C6	200.2	250	150

The SLS value given above is based on a maximum settlement of 25 mm and a footing width of up to 4 m. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with Clause 6.7.4 of CAN/CSA-S6-06.

Immediately upon excavation, the exposed subgrade should be inspected and approved by the geotechnical engineer.

#### 5.4 Lateral Earth Pressures

The lateral earth pressures acting on the culvert walls, headwalls (wing walls), RSS walls and retaining walls (armour stone, gabion etc.) will depend on the type and method of placement of the backfill materials and on the subsequent lateral movement of the structure whether it is restrained or unrestrained. The lateral earth pressures to be used in the design should be computed in accordance with Section 6.9 of the CAN/CSA-S6-06.

Granular backfill should be constructed behind the culvert walls, headwalls (wing walls), RSS walls, and retaining walls as per OPSD-3121.150, with particular attention to the frost taper requirement. The granular backfill should conform to OPSS 1010 for either Granular "A" or Granular "B" Type III. To maintain free draining characteristics in granular fill materials, the maximum percentage passing the No. 200 sieve (75  $\mu$ m) should be limited to 5%.

The backfill should be constructed as per OPSS 902, OPSS 501 and SSP902S01. A perforated subdrain should be installed behind the walls with a positive outlet or wall drains as per OPSD-3190.100 to drain the granular fill above the stream water level. Alternatively, the culvert walls could be designed to resist hydrostatic pressure.

The lateral earth pressure,  $P_h$ , acting on the headwalls (wing walls) or retaining walls may be computed using the equivalent fluid pressures presented in Clause 6.9.2.3 of the CAN/CSA-S6-06, or employing the following equation based on unfactored earth pressure distributions:

$$P_h = K (\gamma h + q)$$

Where:

- $K$  = earth pressure coefficient, use value from table below  
 $\gamma$  = unit weight of soil, = 21.2 kN/m<sup>3</sup> for Granular "B"  
= 22.8 kN/m<sup>3</sup> for Granular "A"  
 $h$  = depth below top of wall, m  
 $q$  = live load surcharge pressure, equivalent fill height of 0.8 m  
as per Clause 6.9.5 of CHBDC and CAN/CSA-S6-06

Wall Type	Earth Pressure Coefficient (K)	
	Granular "A" $\phi = 35^\circ$	Granular "B" $\phi = 30 \text{ to } 35^\circ$
Restrained Wall ( $K_o$ )	0.43	0.50 to 0.43
Unrestrained Wall ( $K_a$ )	0.27	0.33 to 0.27

The submerged unit weight of the backfill should be used for any submerged portion of the granular backfill when calculating the lateral earth pressure.

The above parameters are based on a horizontal back slope (not exceeding 5 degrees) behind the headwalls. A compaction surcharge equal to 12 kPa should be included in the lateral earth



pressures for the structural design of the headwalls and retaining walls in accordance with Clause 6.9.3 of the CAN/CSA-S6-06.

The sliding resistance of the cast-in-place footings should be checked. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between concrete and undisturbed, competent compact silt to sandy silt founding soils can be calculated using a coefficient of friction (friction factor) of 0.35 as per Table 24.4 CFEM 4<sup>th</sup> Edition, 2006. The unfactored horizontal resistance against sliding between concrete and undisturbed, stiff to very stiff clayey silt to silty clay and silty clay till founding soils can be calculated using an adhesion of 50 kPa. Alternatively, a coefficient of friction (friction factor) of 0.45 may be used for concrete on stiff to very stiff clayey silt to silty clay and silty clay till founding soils.

For a precast concrete culvert, the friction factor and adhesion should be reduced by a factor of 0.67. Vibratory equipment for use behind the culvert walls, headwalls (wing walls) and retaining walls should be restricted in size as per current MTO practices, and should conform to OPSS 501 and SSP105S10.

## **5.5 Embankment Widening**

The existing approach embankments are up to 4.7 m high adjacent to the existing culvert. For the widening of the embankment, the surficial topsoil, organic silt and any deleterious materials should be stripped or excavated prior to placing fill materials. The embankment widening should then be constructed as per OPSD-202.010, 202.030 and 208.010, with emphasis on adequate benching of the subgrade for receiving the embankment fill. The fill to be used for embankment construction can either be imported silty clay or granular materials, but granular materials are preferred for compaction and drainage.

Backfill adjacent to the structure should be carried out in conformance with OPSS 902, SSP902S01 and OPSD-3121.150, and the fill should be placed and compacted in accordance with OPSS 501 and SSP105S10.

Based on the findings of the field investigation, no foundation stability or settlement problems due to widening the approach embankments on the inorganic native soils are anticipated for embankment slope of 2.5H:1V and up to 4.7 m high. The fill placement should begin at the toe of the embankment, in leveled lifts and each lift compacted to at least 98% SPMDD. Benching into the existing embankment slope at 1 m high steps is recommended as per OPSD 208.010.

After stripping, the exposed subgrade should be inspected and approved by the geotechnical engineer. The approved subgrade should then be proof-rolled using a heavy compactor, as directed by the engineer. Unless the excavation is carried out in wet weather conditions, no unusual dewatering is anticipated during stripping and preparation of the subgrade to receive the embankment fills. Where necessary, dewatering can be carried out using gravity drainage and pumping from open filtered sumps in accordance with OPSS 517 and 902, and SSP902S01, with emphasis on the requirements of OPSS 518.

Measures should be incorporated into the design and staging to ensure that the slope surfaces are protected from surface erosion in accordance with the requirements of OPSS 577. Proper erosion control measures should be implemented both during construction of the embankment fills and permanently. Erosion control during construction should be carried out by installing silt fences. Properly designed erosion control blankets could also be placed on any new embankments and adjacent disturbed embankments after completion of fill placement. A vegetative cover should be established as soon as practical upon completion of fill placement to minimize the chances of surface erosion.

Revetments such as rip-rap blanket should be provided at the toe of the slope and the ends of the culvert to prevent erosion/scour by stream action in accordance with OPSS 511 and OPSD 810.010. The design of the rip-rap blanket should be carried out cognizant of the stream hydraulics.

#### **5.6 Excavation, Groundwater Control and Temporary Shoring**

Excavation for this project will involve the construction of the box culvert or footings for the culvert walls, headwalls (wing walls) and retaining walls. Depending on the design that is finally selected, the anticipated maximum depth of excavation below the existing grade of Highway 21 is between 5.5 and 6 m.

Excavation to depths of up to 6 m should not present any special difficulties using heavy excavation equipment, provided it is constructed in accordance with OPSS 501, 517, 518, 539, 577 and 902, SSP902S01 and OPSD-803.010 and 3121.150. However, the buried utilities alongside the embankments will likely be in conflict with the excavation. Excavation and protection procedures shall conform to OPSS 539 and should be reviewed with the utility companies or authorities prior to construction. Based on the subsurface soil and groundwater conditions encountered at this site, a Permit to Take Water (PTTW) in accordance with Ontario Regulation 387/04 will not be required for the purpose of excavation.

The water in the creek can be controlled by temporary diversion or dam and pump method. The anticipated minor groundwater ingress can be controlled using intercept ditches and pumping from filtered sump pits.

It is noted that a "Permit To Take Water" (PTTW, Regulation 387/04) will be required from the MOE (Ministry of Environment) when the total quantity of water to be handled exceeds 50,000 litres/day while employing temporary pumping of water, flow passages through culverts, stream diversion or dam and pump method as groundwater control measures (unwatering). It may take up to 90 days for MOE to review an application and issue a permit.

It should be pointed out that if the founding soil is disturbed, excessive settlements could occur after structural loads are applied. The founding level will be located below the streambed and, therefore, a minimum 50 mm thick lean concrete working mat should be placed immediately

after excavation and subgrade preparation for footings to protect the integrity of the bearing surface and to facilitate placement of reinforcing steel. All foundation excavations, bearing surfaces, and placement of lean concrete mat should be inspected and approved by the geotechnical engineer.

All excavation must be carried out in compliance with the requirements of the Occupational Health and Safety Act (OHSA). For this purpose, the unsaturated upper fill materials, stiff clayey silt to silty clay and compact sandy silt to silt encountered at this site are classified as Type 3 soils and the underlying very stiff to hard silty clay till soils are classified as Type 2 soils. Saturated cohesionless soils are classified as Type 4 soils.

For the Type 2 soils, the excavation shall be cut to near vertical in the bottom 1.2 m and then trimmed back to 1H:1V. Within the Type 3 soils and above the water table, the excavation shall be cut to no steeper than 1H:1V throughout. Side slopes of 3H:1V or flatter shall be used for excavation within Type 4 soils.

Temporary support within the overfill of the existing and the new partially constructed embankment/culvert may be required to facilitate culvert construction and to maintain access for construction and local traffic, and emergency vehicles. The staging of different phases of this work should be examined to determine if roadway protection is required. Roadway protection is generally a contractor design/build item in accordance with SSP105S19 and current MTO practices. A performance level 2 and a maximum allowable lateral movement of 25 mm should be specified in accordance with the requirements of SSP105S19.

Geotechnical parameters for the design of temporary support structures are provided in Sections 5.3 and 5.4. In addition, a unit weight of  $21 \text{ kN/m}^3$  and an internal friction angle ( $\phi$ ) of  $28^\circ$  for the embankment fill can be used for design if the existing embankment is to be supported. Further, the toe support of the temporary support system will be generated in the underlying compact sandy silt to silt and stiff to very stiff silty clay till. For design purposes, the sandy silt to silt has an internal friction angle ( $\phi$ ) of  $30^\circ$  and a unit weight of  $22 \text{ kN/m}^3$ . The silty clay till has a unit weight of  $22 \text{ kN/m}^3$  and an undrained shear strength of 150 kPa (unfactored).

## **5.7 Frost Protection**

This project is located in the Owen Sound Operations District (previously known as District No. 5). The design frost penetration depth for this project is 1.2 m in accordance with OPSD 3400.011. All foundations and spread footings should be provided with at least 1.2 m of soil cover for adequate frost protection. Alternatively, frost protection can be provided by equivalent thermal insulation.

## **5.8 Scour Depth**

The footings should be founded below the anticipated local and general scour depths as per

CAN/CSA-S6-06, Clause 1.9, Hydraulic Design. The permissible velocities of the various soil types which will be exposed at the streambeds (based on American Society of Civil Engineers publication, 1926, reprinted as Design Chart 2.17, MTO Drainage Management Manual 1997) are provided in the following table:

<b>Soil Type</b>	<b>Permissible Velocity (m/sec)</b>
Sandy Silt	0.8
Silt	0.8
Clayey Silt	1.5
Silty Clay	1.5
Silty Clay Till	1.5

## 6.0 STATEMENT OF LIMITATION

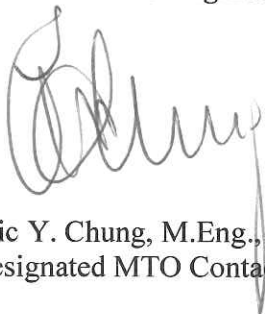
We recommend that once the details of the proposed structure are finalized, our recommendations should be reviewed for their specific applicability.

The Limitations of Report, as Quoted in Appendix "C", is an integral part of this report.

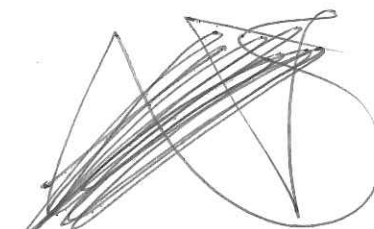
We trust that we have completed the assignment within the Terms of Reference for this project. If there are any questions concerning this report, please do not hesitate to contact our office.

Yours truly,

**Infrastructure Engineering Group Inc.**



Eric Y. Chung, M.Eng., P.Eng.  
Designated MTO Contact



Joseph Law, P.Eng.  
Project Manager



Tom O'Dwyer, P. Eng.  
Quality Review Engineer



Ministry of Transportation/Stantec Consulting Ltd.  
G.W.P. 406-94-00  
Rehabilitation of Highway 21 from St. Joseph to Bayfield  
Agreement # 3006-E-0095

08-1-IEG2-12-419-C  
Final Report  
Drawing 1  
November 20, 2009

Drawing 1  
Borehole Locations  
And  
Soil Strata

**Infrastructure Engineering Group Inc.**

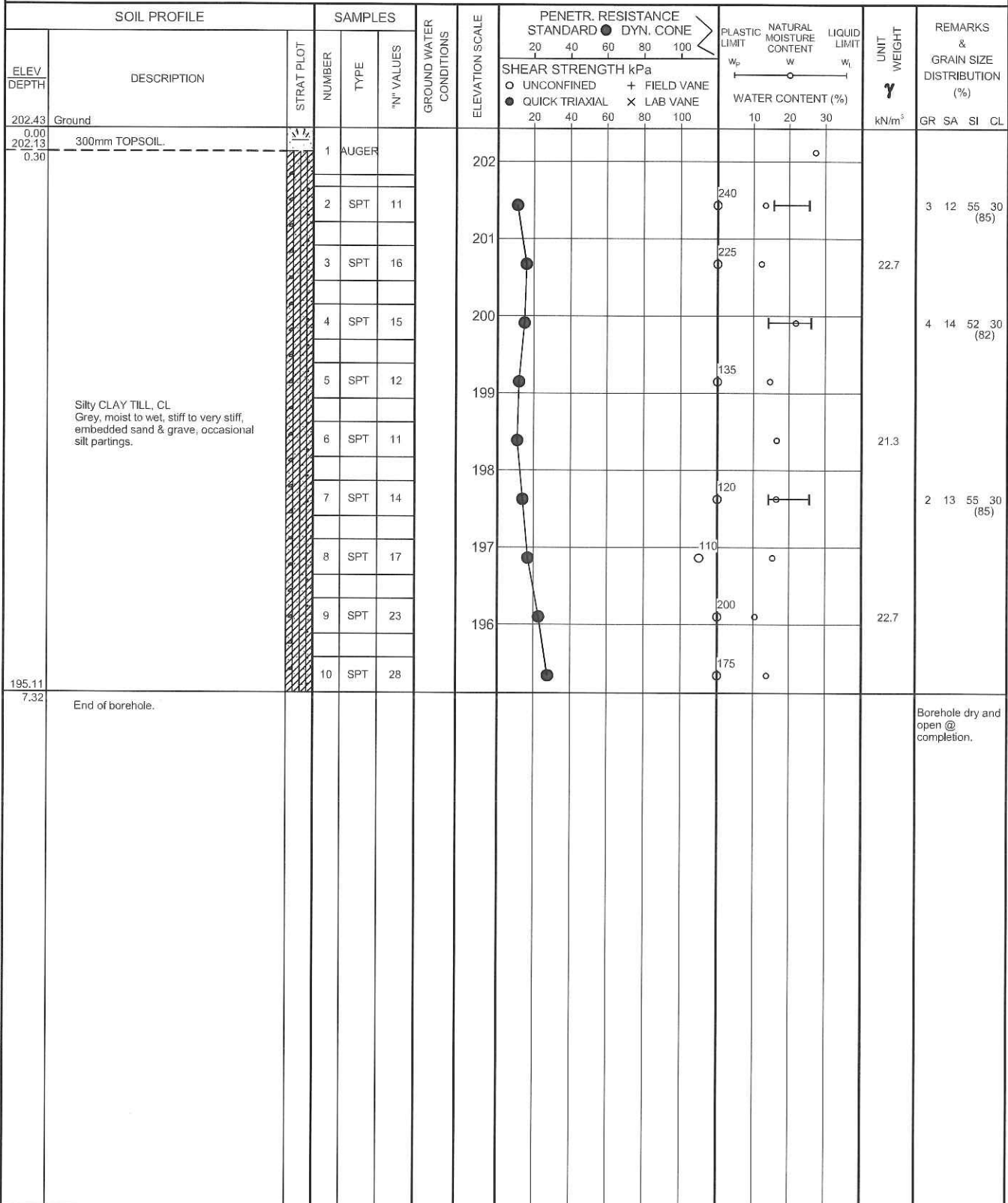


# RECORD OF BOREHOLE No 12-419-C1

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823922, Easting - 370202 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 9.6.08 - 9.6.08 CHECKED BY JL



JOE MTO 08-1-JEG2 HWY 21.GPJ ONTARIO MOT.GDT 22/9/09

+ 3, × 3: Numbers refer to  
Sensitivity

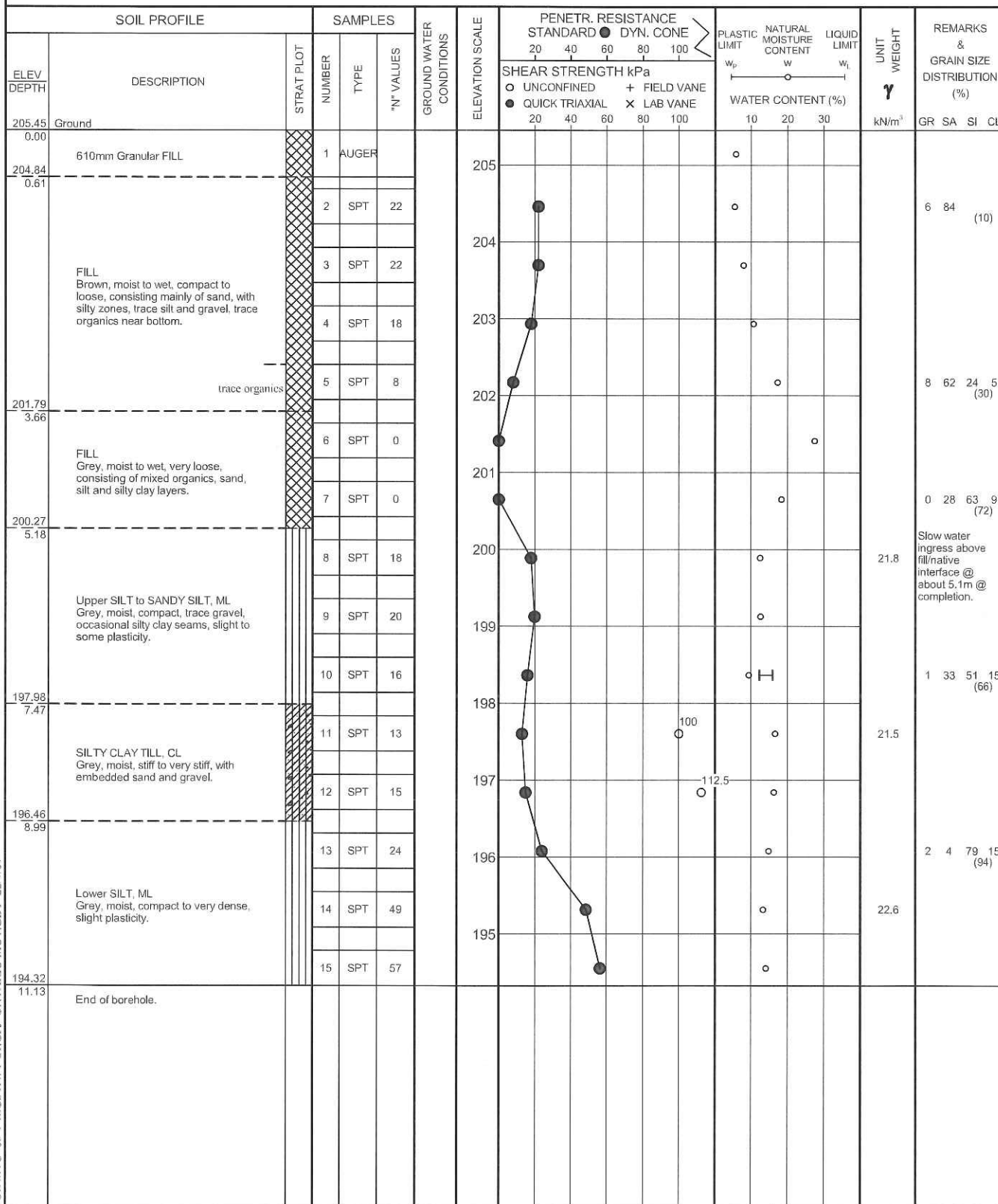
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

# RECORD OF BOREHOLE No 12-419-C2

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823919, Easting - 370213 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 9.6.08 - 9.6.08 CHECKED BY JL



TOE MTO 08-1-IEG2 HWY 21.GPJ ONTARIO NOT.GDT 22/9/09

+ 3, x 3: Numbers refer to Sensitivity

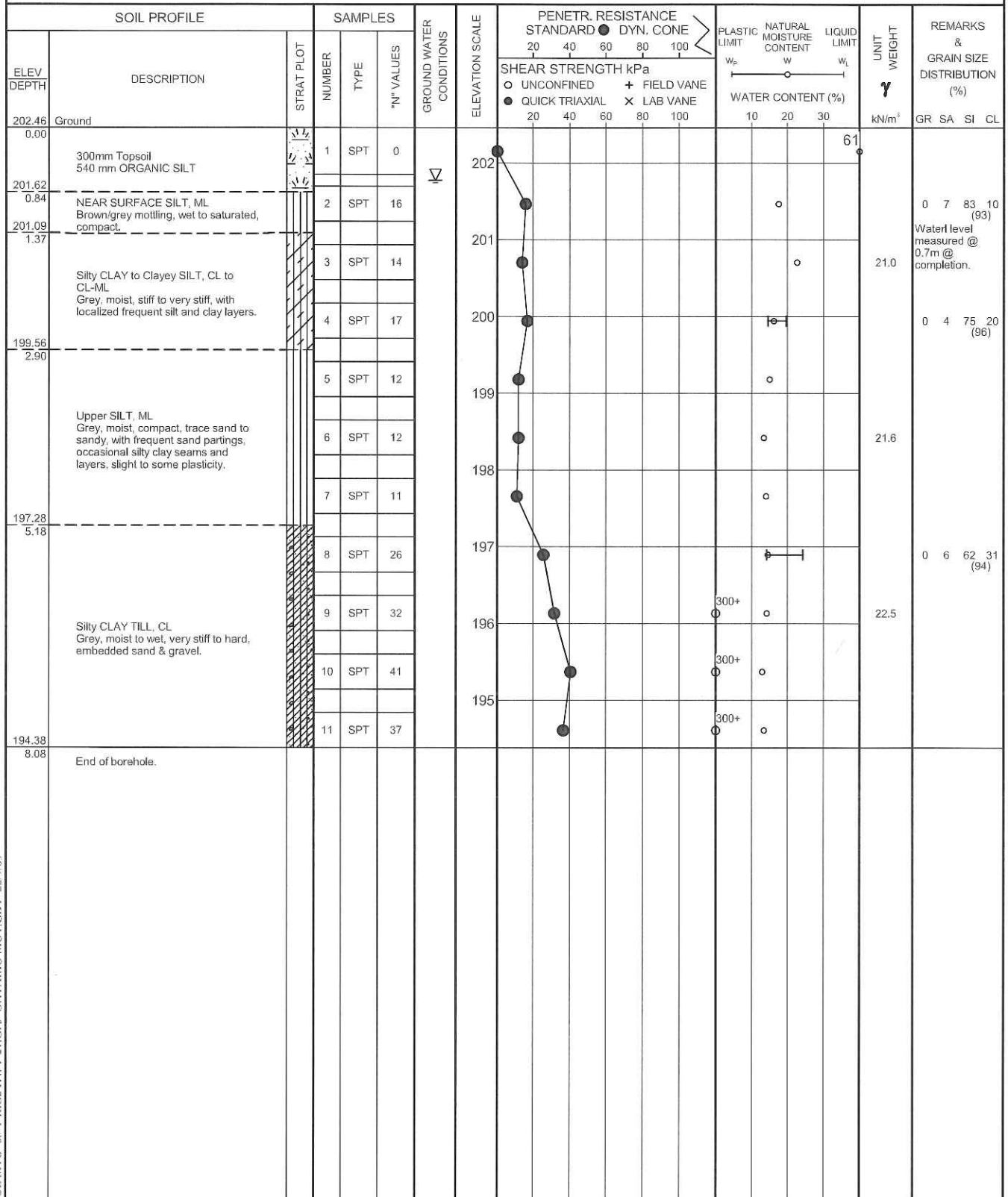
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

# RECORD OF BOREHOLE No 12-419-C3

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823947, Easting - 370252 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 11.6.08 - 11.6.08 CHECKED BY JL



JOE MTO 08-1-HEG2 HWY 21.GPJ ONTARIO MOT.GDT 22/9/09

+ 3, x 3: Numbers refer to  
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

# RECORD OF BOREHOLE No 12-419-C4

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823838, Easting - 370235 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 11.6.08 - 11.6.08 CHECKED BY JL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
204.16	Ground													
0.00	300mm TOPSOIL.		1	SPT	3		204							
203.86			2	SPT	15		203							
0.30			3	SPT	18		202							
	Silty SAND, SM Brown, wet to saturated, very loose to compact, trace gravel.		4	SPT	26		201							
201.72			5	SPT	32		200							
2.44			6	SPT	21									
	SILT, ML Grey, wet, compact to dense, with silty clay seams, trace sand.													
199.89	End of borehole.													
4.27														

JOE MTO 08-1-HEG2 HWY 21.GPJ ONTARIO MDT.GDT 22/9/09

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to  
Sensitivity

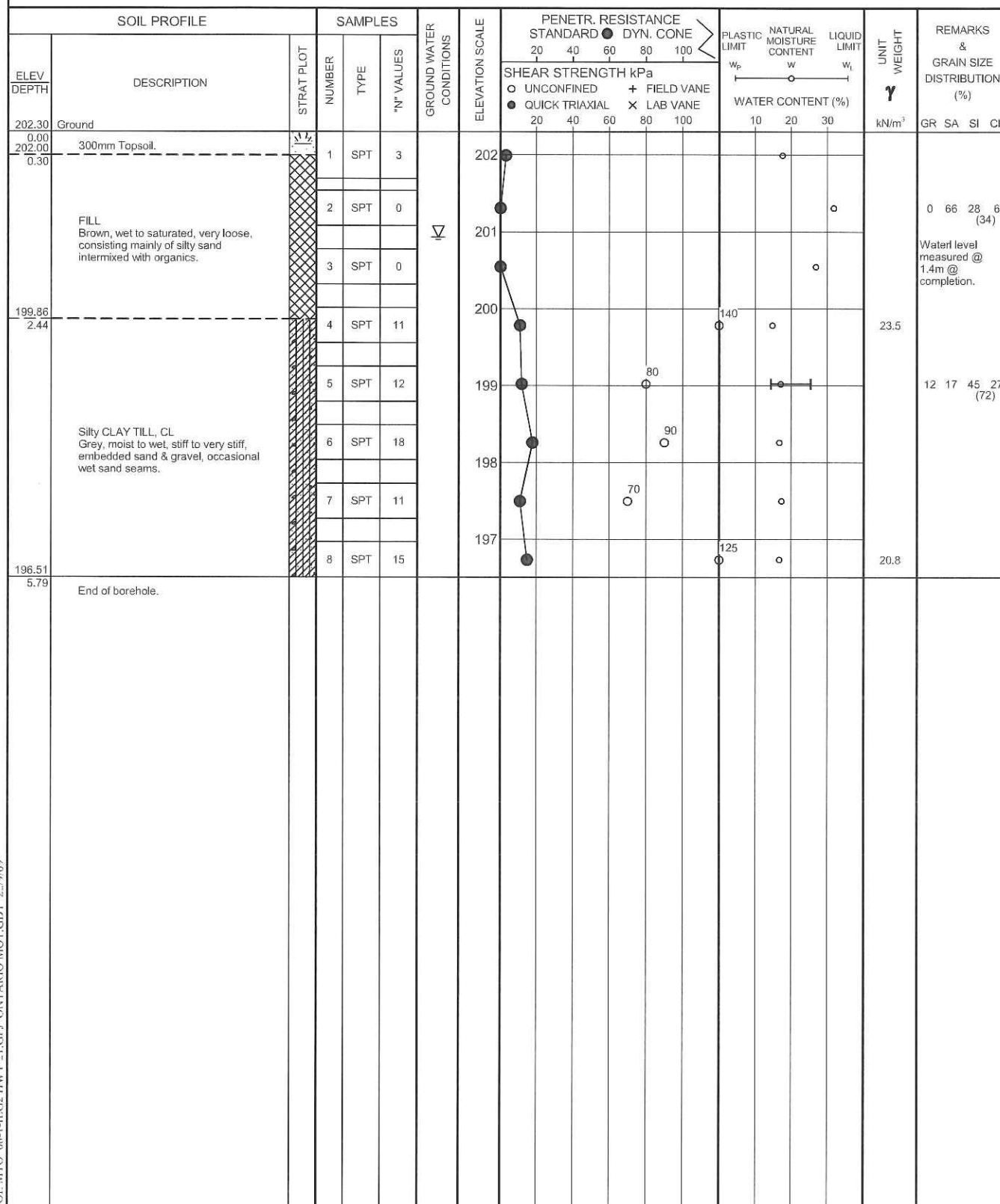
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

# RECORD OF BOREHOLE No 12-419-C5

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823877, Easting - 370200 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 11.6.08 - 11.6.08 CHECKED BY JL



JOE MTO 08-1-HEG2 HWY 21.GPJ ONTARIO MOT.GDT 22/9/09

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

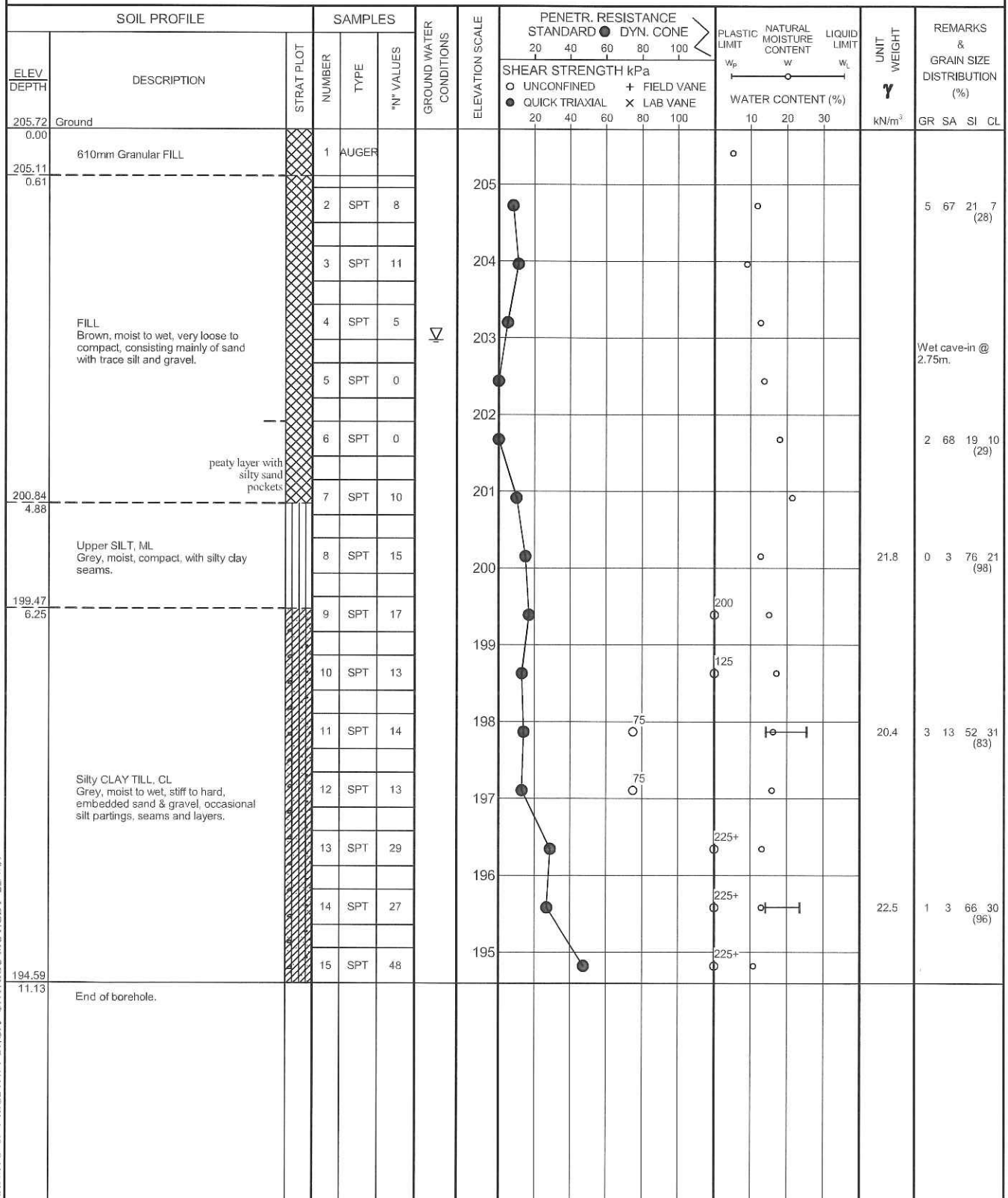
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 12-419-C6

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823948, Easting - 370225 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 9.6.08 - 9.6.08 CHECKED BY JL



JOE MTO 08-1-HEG2 HWY 21.GPJ ONTARIO MOT.GDT 22/9/09

+ 3, X 3: Numbers refer to Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

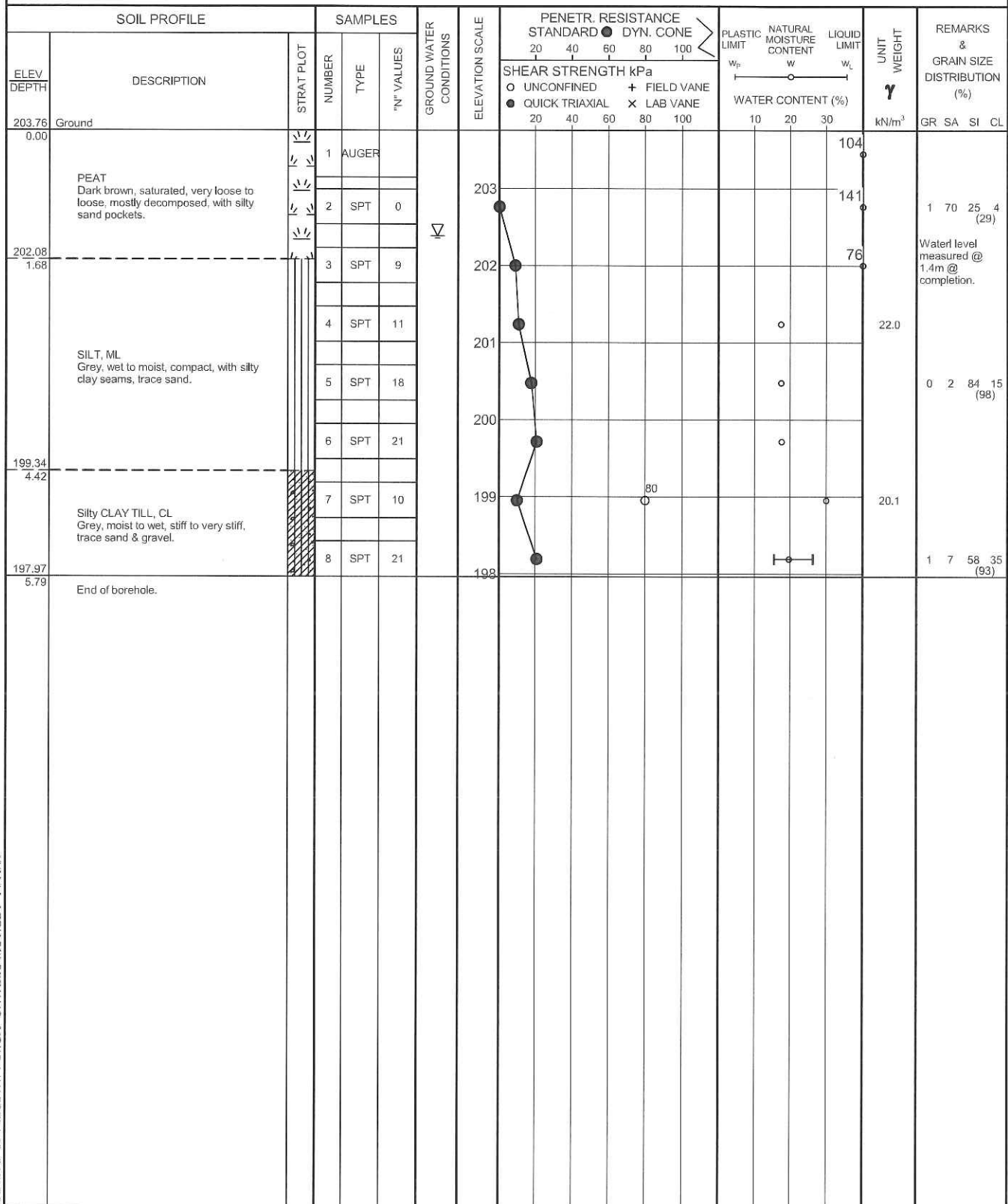


# RECORD OF BOREHOLE No 12-419-C7

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4823978, Easting - 370234 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 11.6.08 - 11.6.08 CHECKED BY JL



JOE MITO 08-1-IEC2 HWY 21.GPJ ONTARIO MOT.GDT 14/11/09

+ 3, x 3: Numbers refer to Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 12-419-C8

1 OF 1

METRIC

W.P. GWP 406-94-00 LOCATION St. Joseph to Bayfield Northing - 4824026, Easting - 370215 ORIGINATED BY RB  
 DIST Owen Sound HWY 21 BOREHOLE TYPE S/S Augering, 110 mm dia. COMPILED BY NN  
 DATUM Geodetic DATE 9.6.08 - 9.6.08 CHECKED BY JL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
206.15 0.00	Ground		1	AUGER										GR SA SI CL		
205.54 0.61	610mm Granular FILL		2	SPT	17	▽	206							8 84 (8)  Water level measured @ 1.2m @ completion.		
			3	SPT	20		205									
			4	SPT	38		204									
			5	SPT	31		203									
			6	SPT	54		202									
202.19 3.96 201.88 4.27	End of borehole.															

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Agreement # 3006-E-0095

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Appendix B  
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## Appendix B

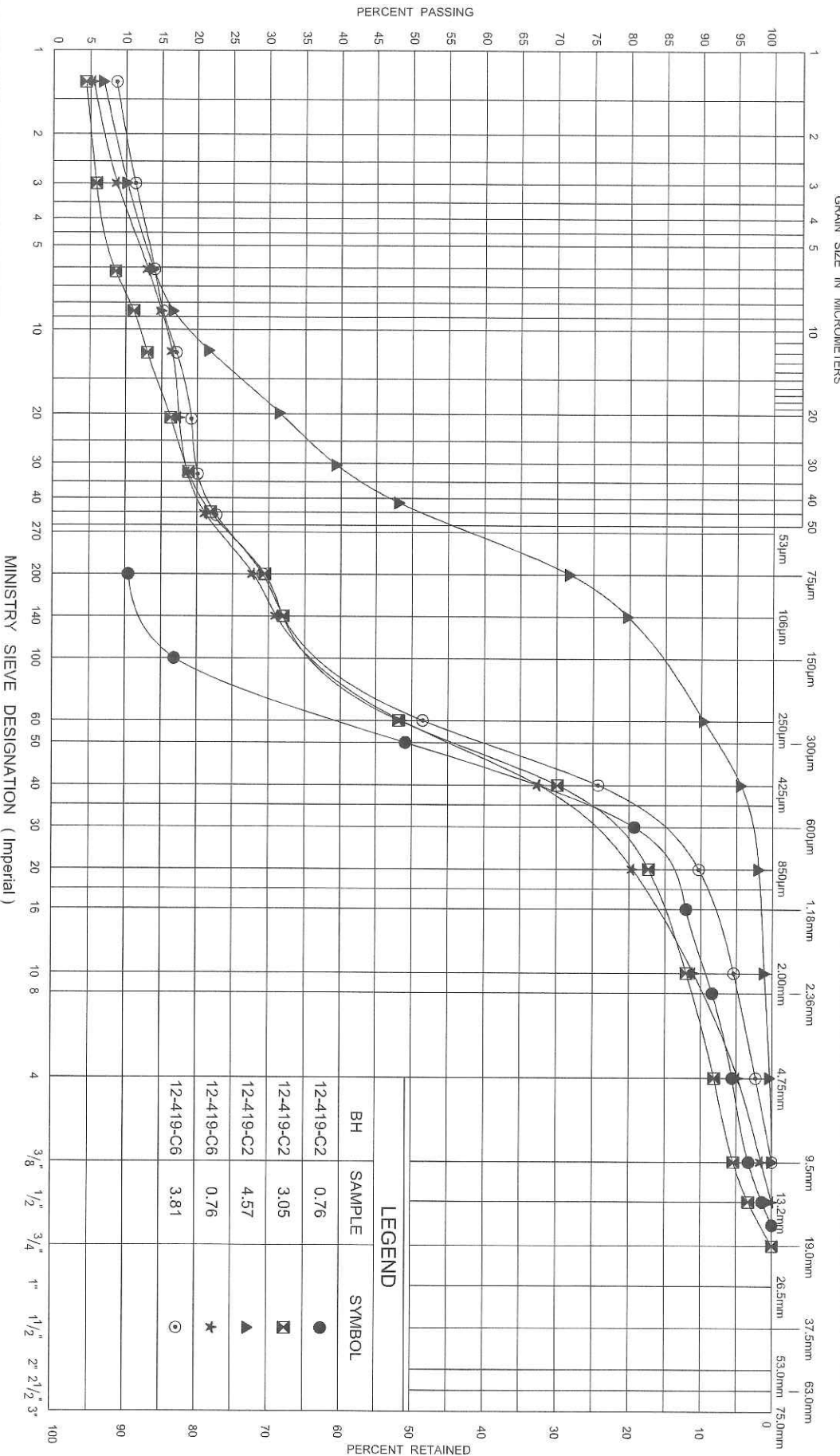
### Laboratory Test Results

Grain Size Distribution	Figures 1, 2, 3, 5, 7, 9
Plasticity Chart	Figures 4, 6 and 8

78 12 M

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



MINISTRY SIEVE DESIGNATION (Imperial)

## GRAIN SIZE DISTRIBUTION



FILL

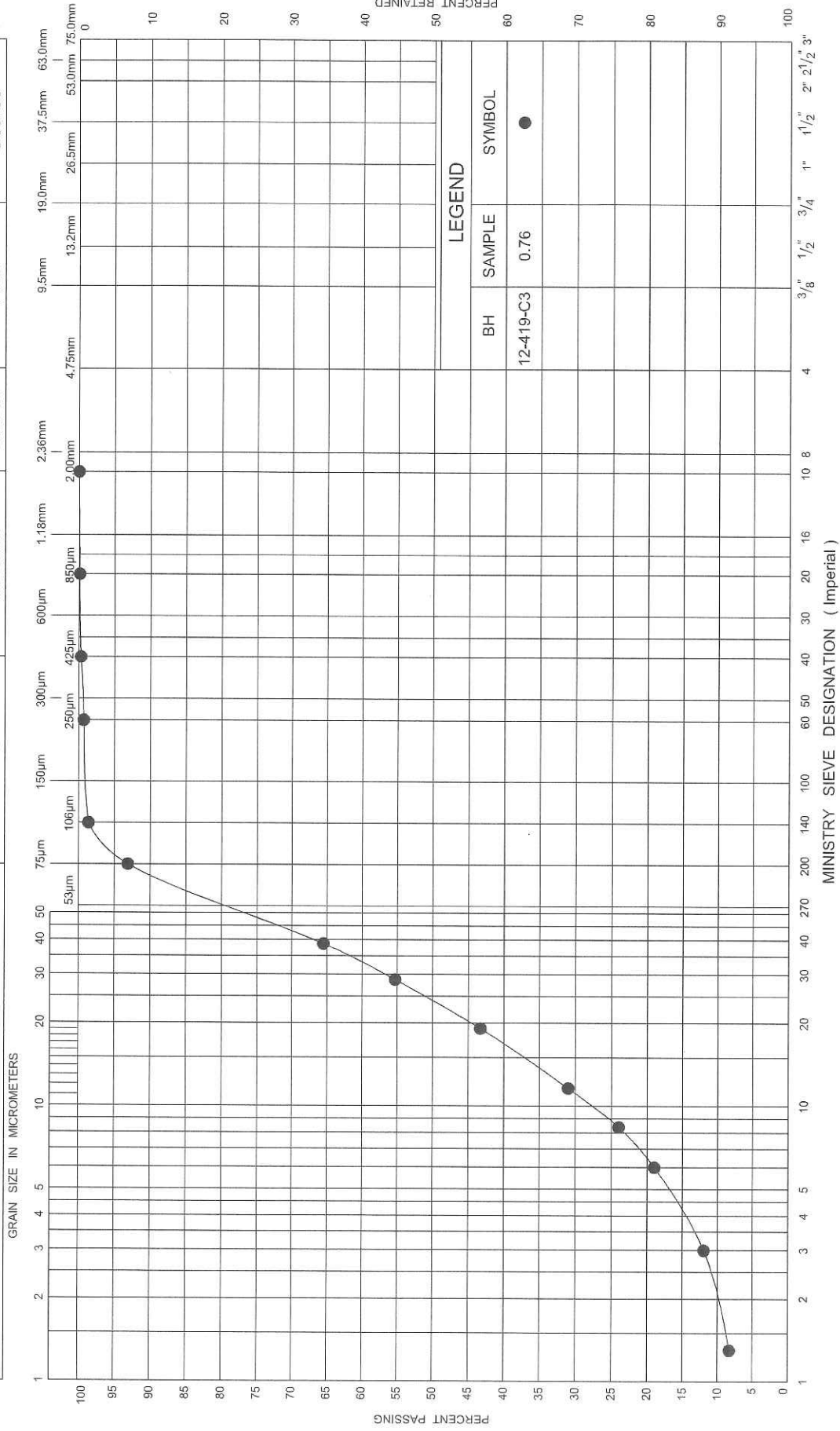
FIG No 1

GWP 406-94-00

St. Joseph to Bayfield

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine		Medium	Coarse	Fine	Coarse



## LEGEND

BH	SAMPLE	SYMBOL
12-419-C3	0.76	●

Ministry of  
Transportation

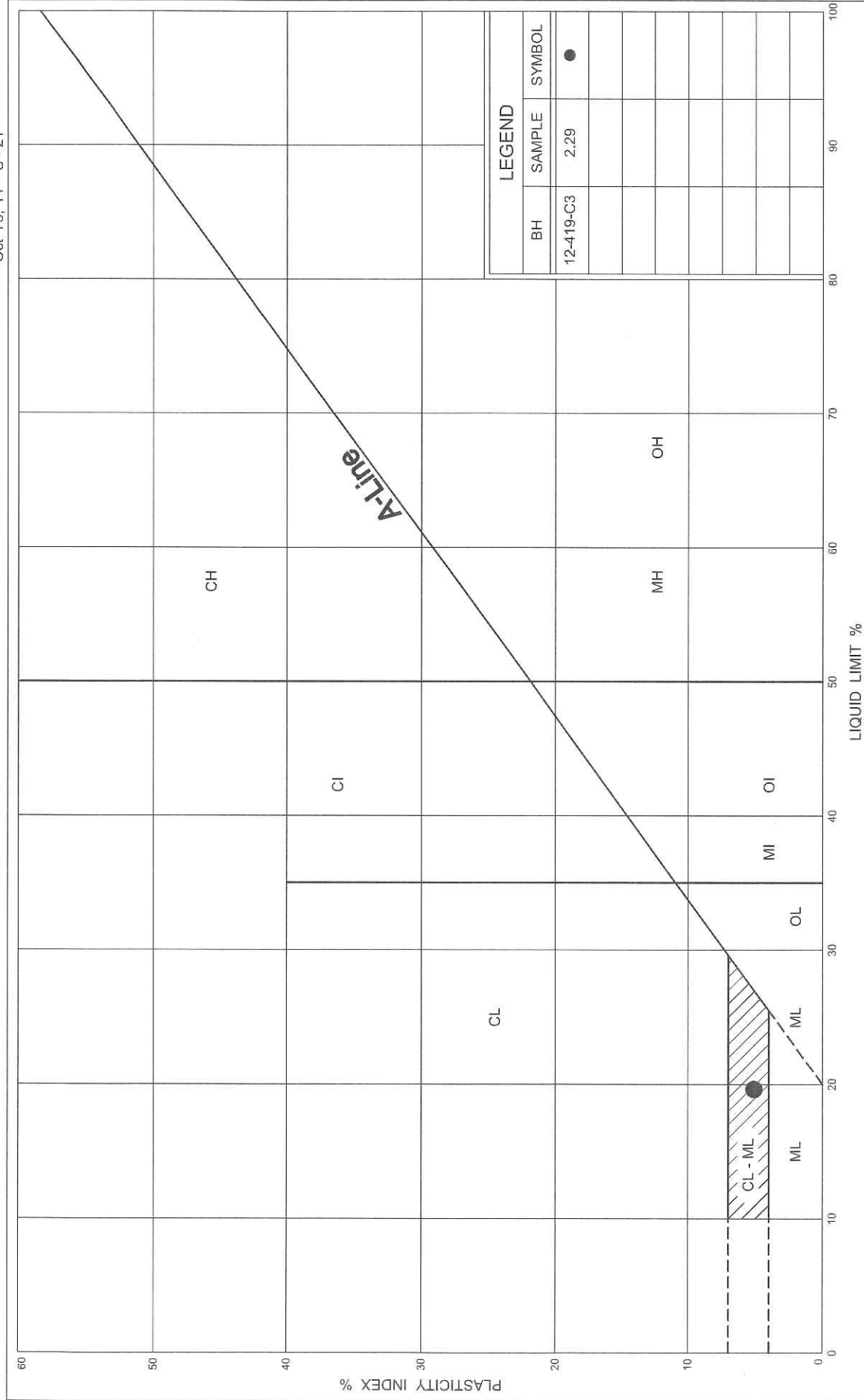


## GRAIN SIZE DISTRIBUTION NEAR SURFACE SILT, ML

FIG No 2

GWP 406-94-00

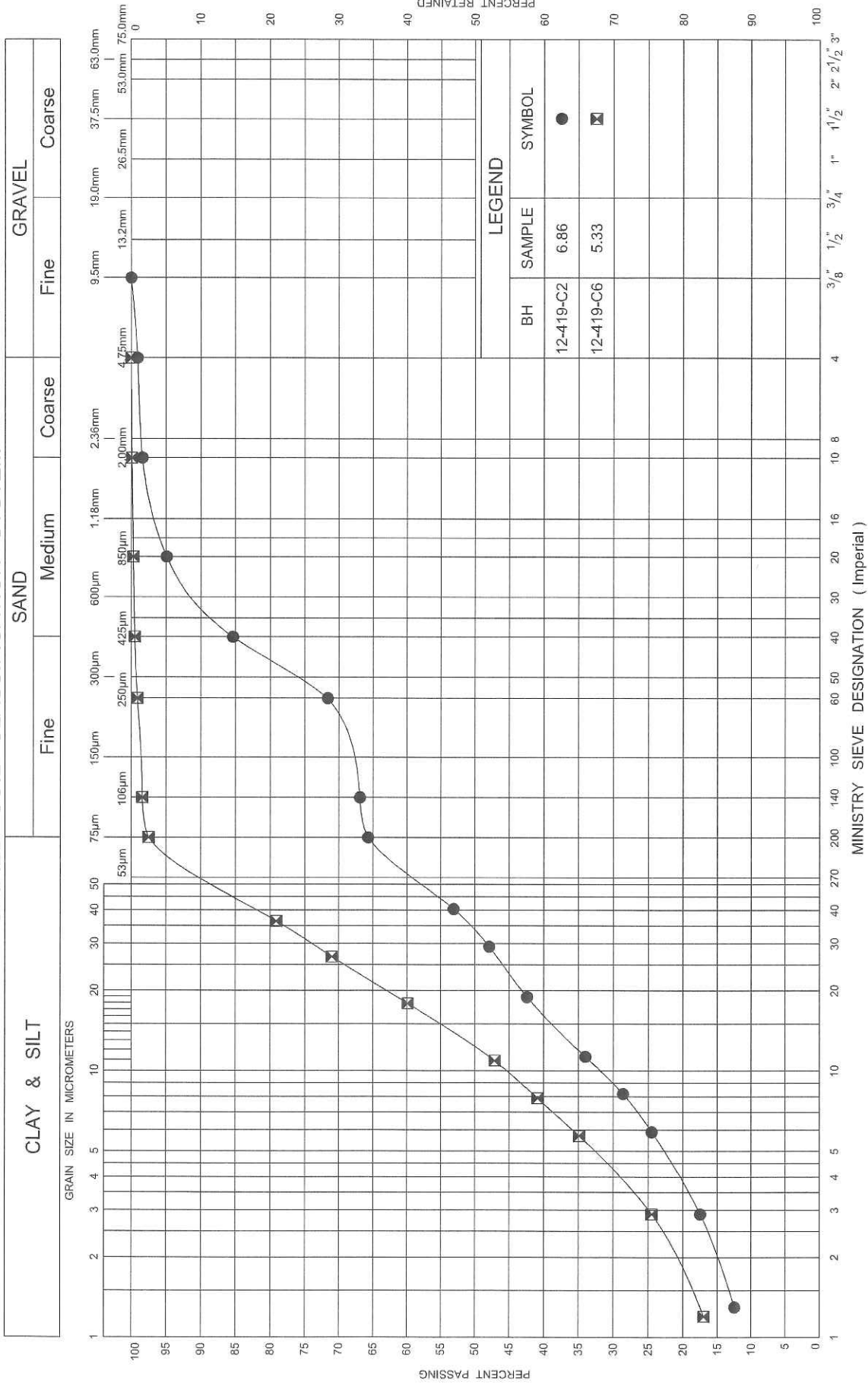
St. Joseph to Bayfield



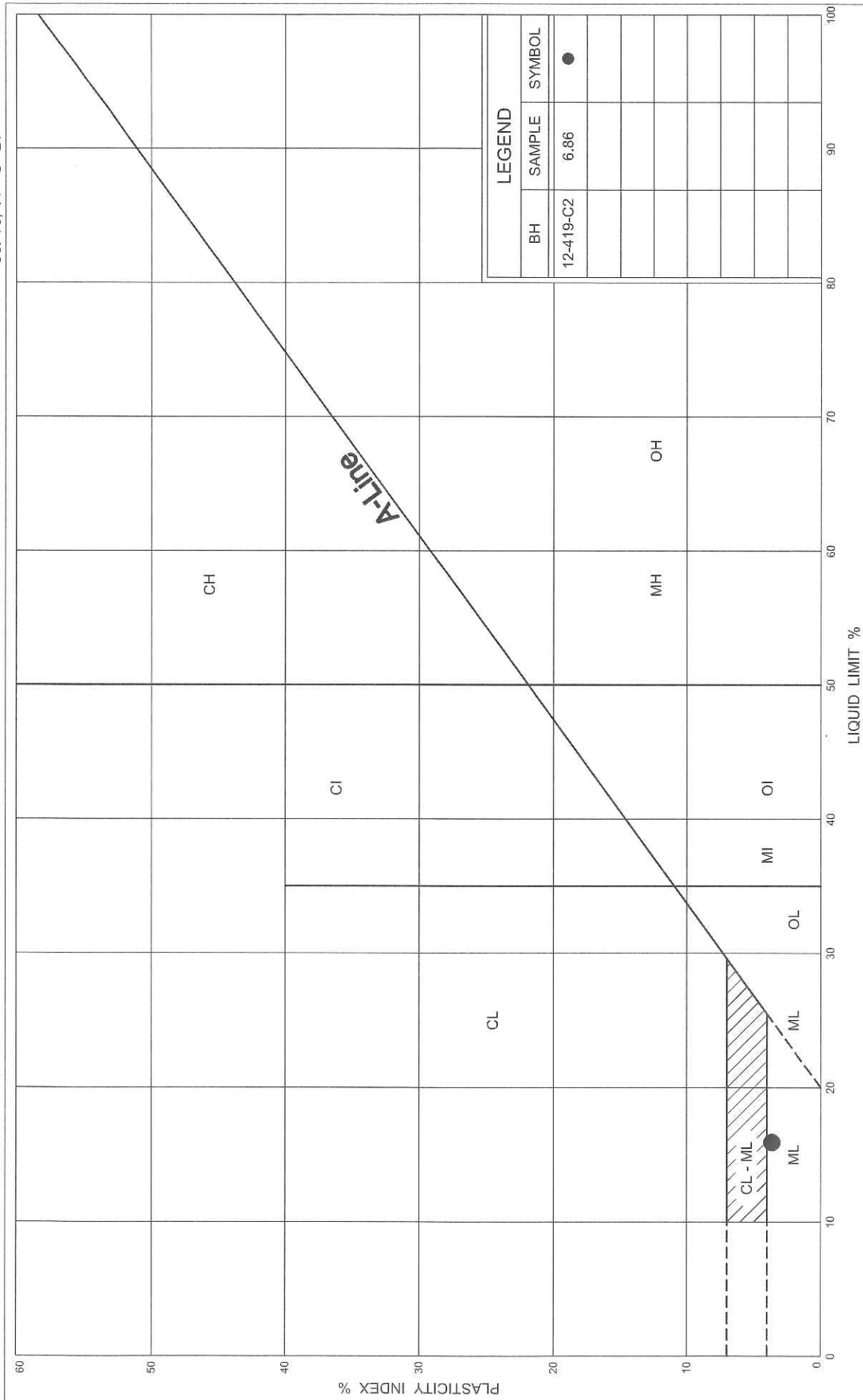
PLASTICITY CHART  
CLAYEY SILT, CL-ML

FIG No 4  
GWP 406-94-00  
St. Joseph to Bayfield

# UNIFIED SOIL CLASSIFICATION SYSTEM







# PLASTICITY CHART

UPPER SILT TO SANDY SILT WITH SILTY CLAY SEAMS, ML

Ministry of  
Transportation



FIG No 6

GWP 406-94-00

St. Joseph to Bayfield

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

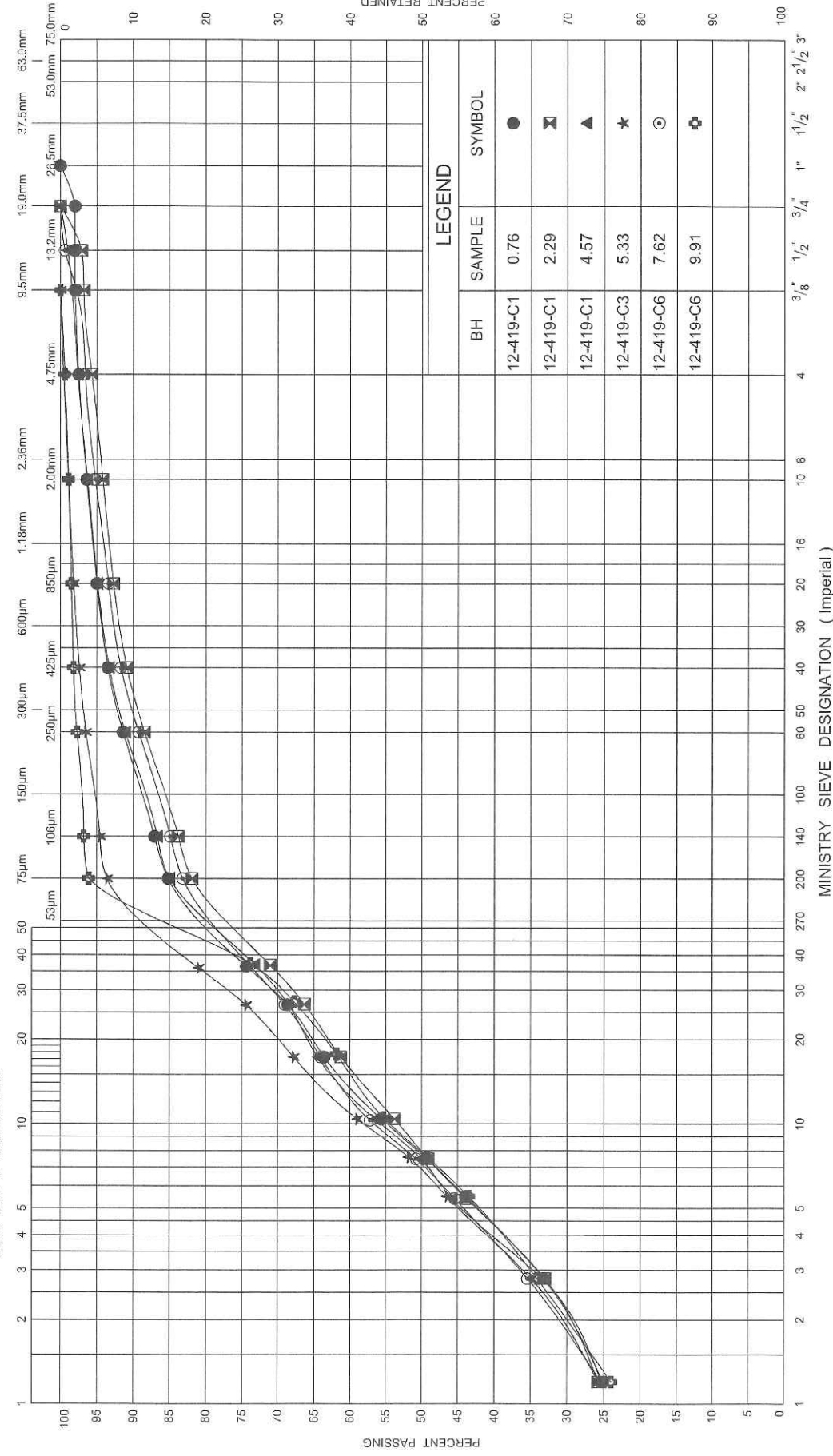


FIG No 7  
GRAIN SIZE DISTRIBUTION  
SILTY CLAY TILL, CL

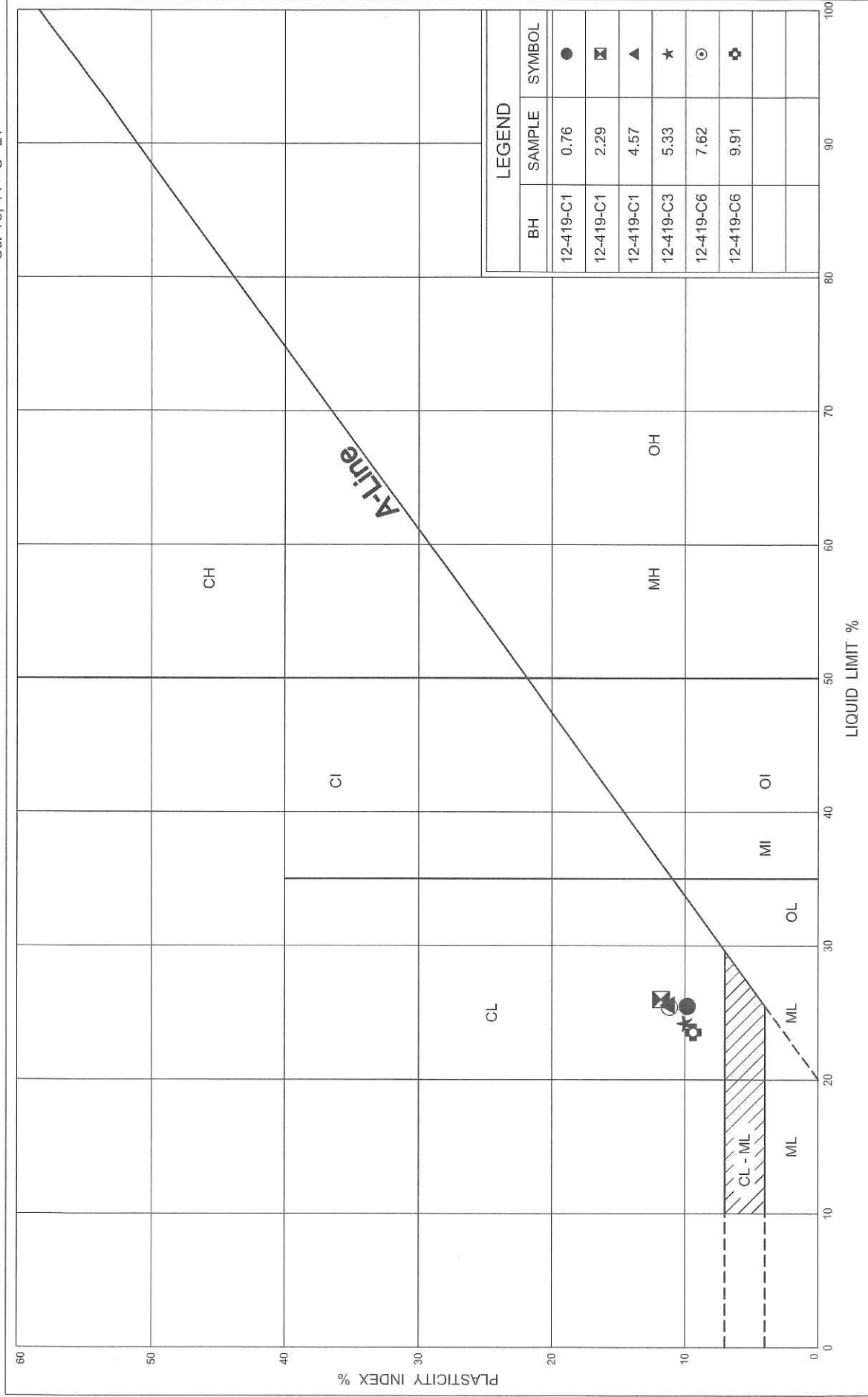


FIG No 8

PLASTICITY CHART

SILTY CLAY TILL, CL

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Coarse

Medium

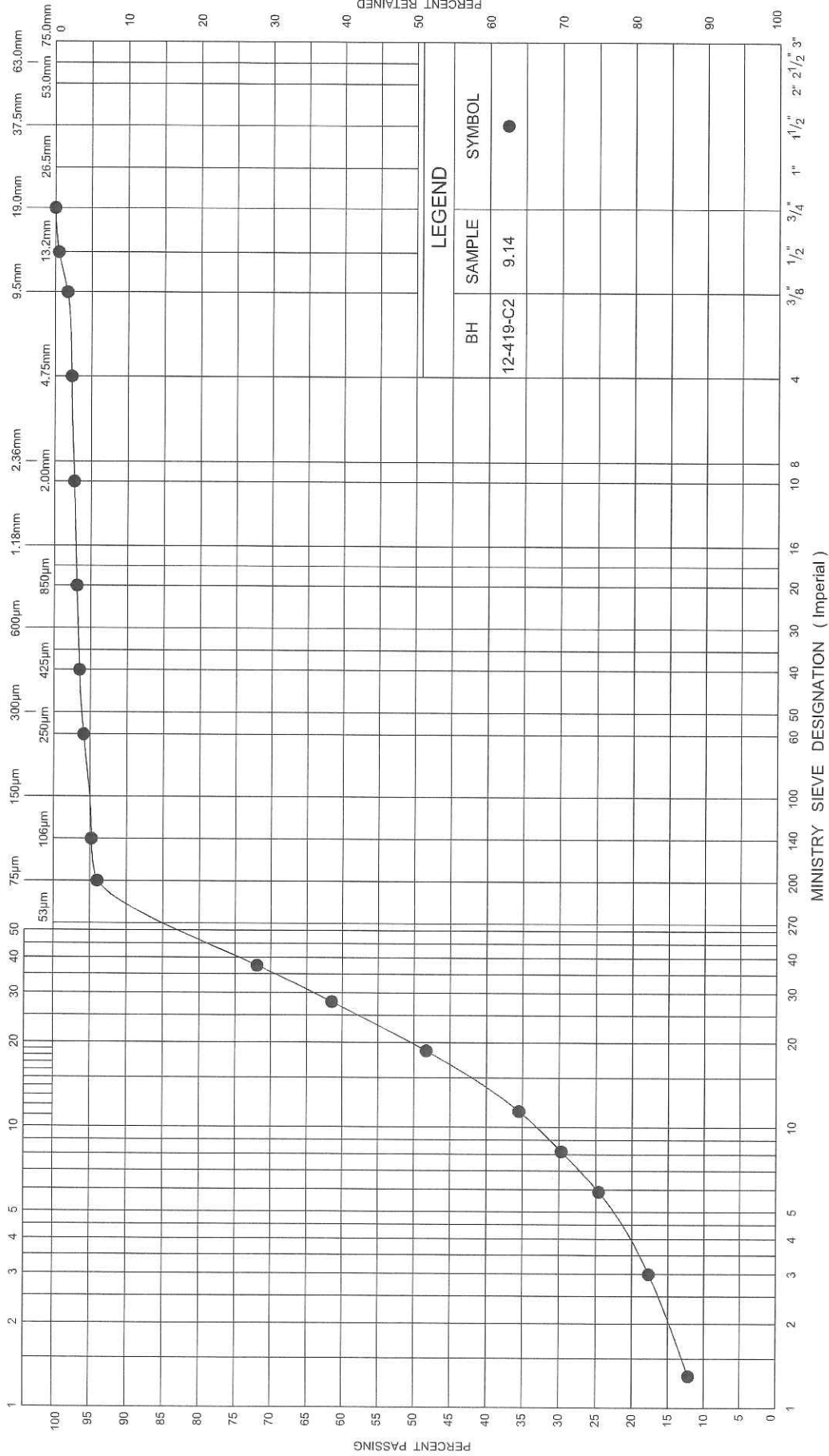
Fine

Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS



## LEGEND

BH	SAMPLE	SYMBOL
12-419-C2	9.14	●

Ministry of  
Transportation



## GRAIN SIZE DISTRIBUTION

LOWER SILT, ML

FIG No 9

GWP 406-94-00

St. Joseph to Bayfield

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Rehabilitation of Highway 21 from St. Joseph to Bayfield  
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08-1-IEG2-12-419-C  
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Appendix C  
November 20, 2009

Appendix C  
Limitations of Report

**Infrastructure Engineering Group Inc.**

## **APPENDIX C**

### **LIMITATIONS OF REPORT**

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

The benchmark and elevations mentioned in this report were obtained strictly for use in the geotechnical design of the project and by this office only, and should not be used by any other parties for any other purposes.

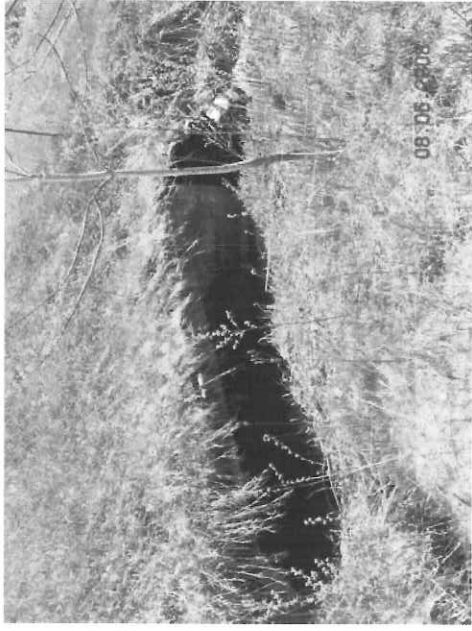
Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Infrastructure Engineering Group Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report does not reflect the environmental issues or concerns unless otherwise stated in the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, IEG recommends that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.



Culvert 12-419-C Looking South



Culvert 12-419-C, Downstream



Culvert 12-419-C, Upstream

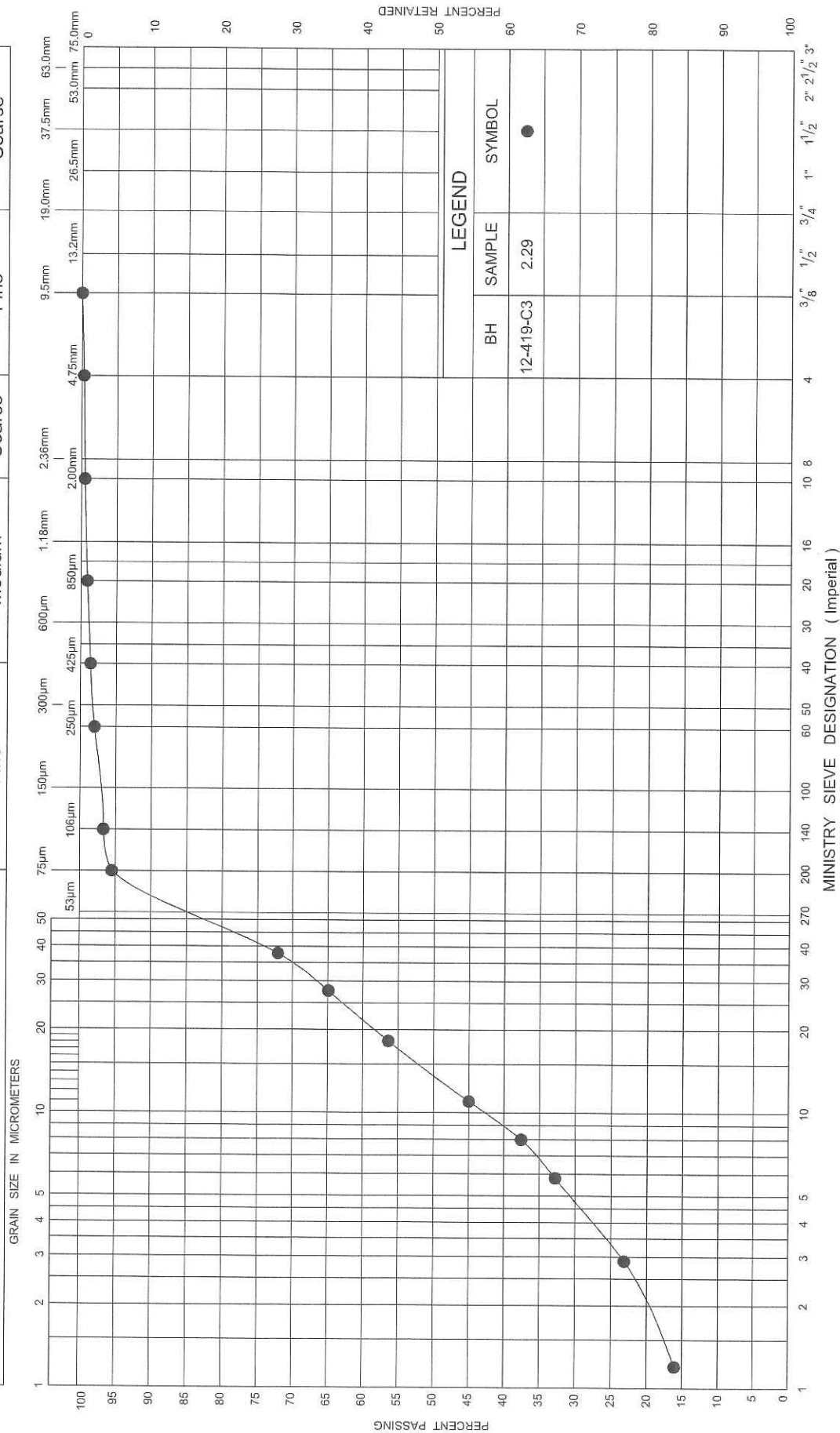


## CLAY &amp; SILT

## SAND

## GRAVEL

## Coarse

Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION

CLAYEY SILT, CL-ML

FIG No 3

GWP 406-94-00

St. Joseph to Bayfield

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

CONT No 2009-3023  
WP No GWP 406-94-15



Culvert # 12-419-C  
Highway 21

SHEET  
xxx

BORE HOLE LOCATIONS & SOIL STRATA

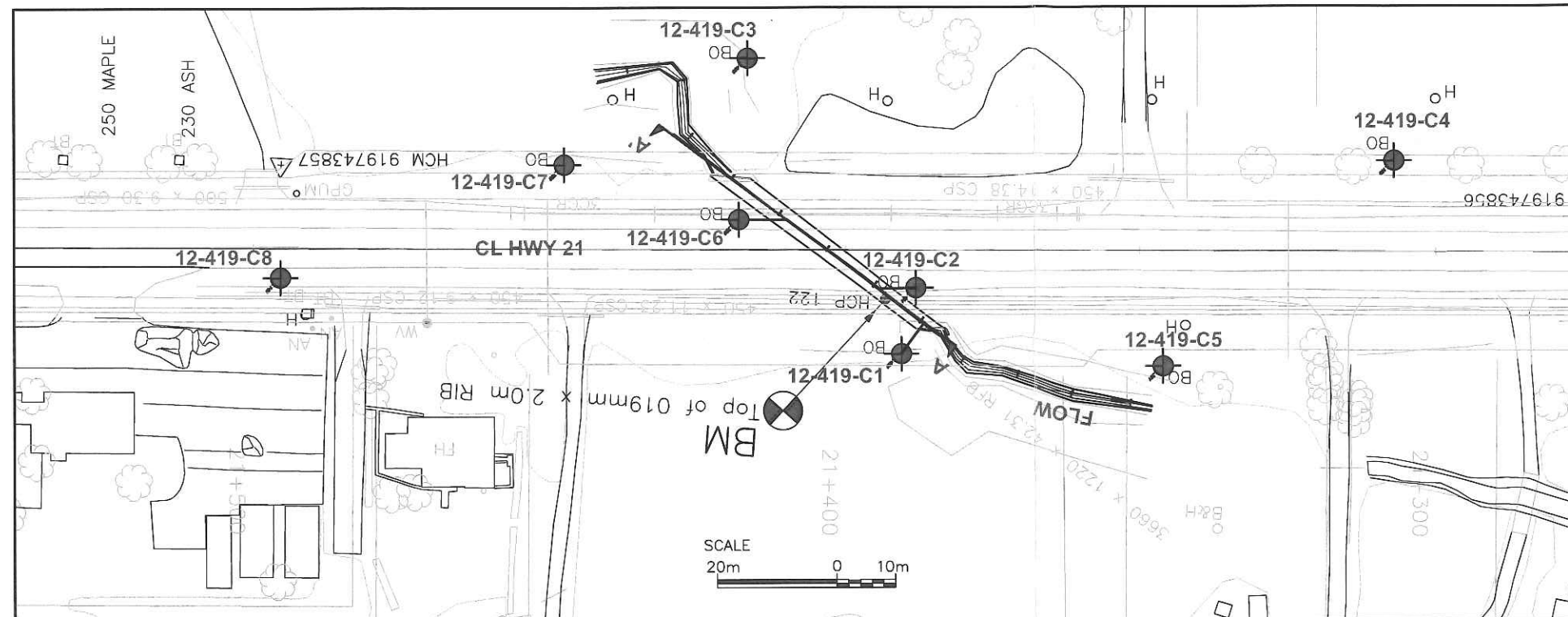
I.E. Infrastructure Engineering Group Inc.  
Pavement & Construction Materials Consulting Engineers  
GTA • Kitchener • London • Windsor

KEYPLAN NTS

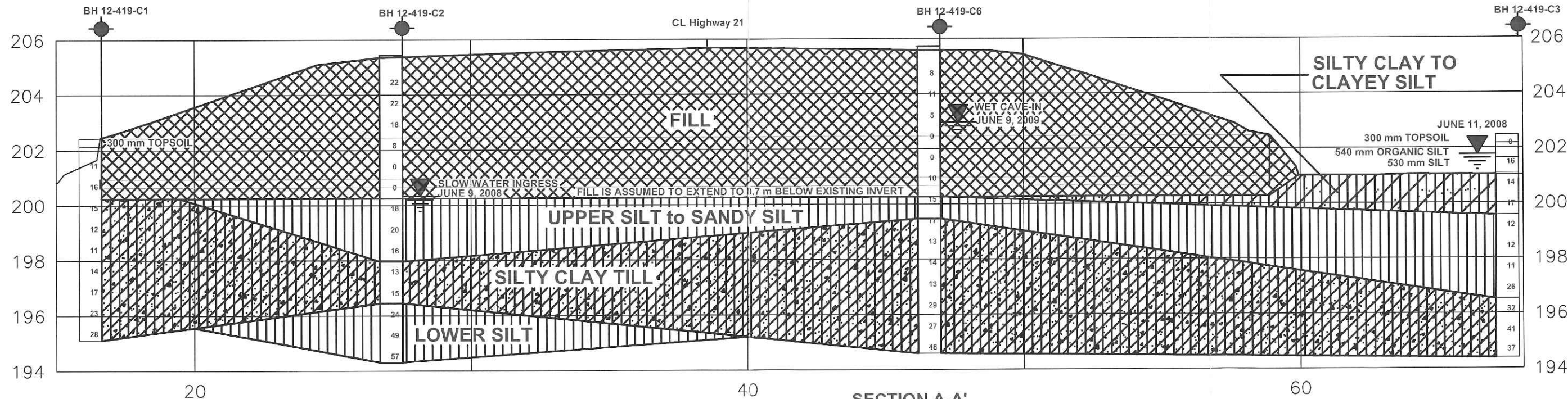


LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Standpipe



BOREHOLE LOCATION PLAN



SECTION A-A'  
CENTERLINE OF CULVERT

SCALE  
5m  
Horizontal and Vertical

BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES NORTH	EAST	BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES NORTH	EAST	BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES NORTH	EAST
12-419-C1	202.43	4823922	370202	12-419-C4	204.16	4823838	370235	12-419-C7	203.76	4823978	370234
12-419-C2	205.45	4823919	370213	12-419-C5	202.30	4823877	370200	12-419-C8	206.15	4824026	370215
12-419-C3	202.46	4823947	370252	12-419-C6	205.72	4823948	370225				

NOTES

- 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 ARE SPECIFICALLY EXCLUDED IN ACCORDANCE WITH THE CONDITIONS OF SECTION GC2.01 of OPS GEN. COND.
- THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- BOREHOLES 4, 5, 7 AND 8 WAS PUT DOWN FOR POTENTIAL EMBANKMENT WIDENING WHICH IS NO LONGER REQUIRED. PROFILES FOR THESE BOREHOLES ARE NOT NEEDED AND THEREFORE NOT PROVIDED.

20/11/09	J.L.	Final
04/11/09	J.L.	Executive Review
01/09/09	J.L.	Draft
DATE	BY	DISCUSSION
Geocres : 40P12-13		
HWY No.	HWY 21	DIST Owen Sound
SUBM'D J.L.	CHECKED E.C.	DATE 01/09/09
DRAWN J.L.	CHECKED J.L.	APPROVED E.C.
DWG	1	

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m, N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_{\alpha}$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	$m^2/s$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_p$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $\frac{c_u}{\tau_f}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$kg/m^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$kN/m^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	$kg/m^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$kN/m^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$kg/m^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$kN/m^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$kg/m^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$m^3/s$	RATE OF DISCHARGE
$\gamma_d$	$kN/m^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	$kg/m^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	$kN/m^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$		CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$kg/m^3$	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$kN/m^3$	SEEPAGE FORCE
$\gamma'$	$kN/m^3$	UNIT WEIGHT OF SUBMERGED SOIL						

Ministry of Transportation/Stantec Consulting Ltd.  
G.W.P. 406-94-00  
Rehabilitation of Highway 21 from St. Joseph to Bayfield  
Agreement # 3006-E-0095

08-1-IEG2-12-419-C  
Final Report  
Appendix D  
November 20, 2009

Appendix D  
Site Photographs