

FINAL REPORT

**Foundation Investigation and Design Report
Gabion Wall Construction near Laronde Creek
Hwy 17, District 54
G.W.P. 5274-08-00
MTO GEOCRES No. 31L-137**

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Table of Contents

1. Part I: FOUNDATION INVESTIGATION	1
1.1 Introduction.....	1
1.2 Site Description and Geological Setting	1
1.2.1 Site Description	1
1.2.2 Geological Setting	2
1.3 Investigation Procedures.....	2
1.3.1 General	2
1.3.2 Laboratory Testing	3
1.4 Subsurface Conditions	3
1.4.1 Asphalt	4
1.4.2 Sand Fill	4
1.4.3 Sandy Silt	5
1.4.4 Silty Sand	5
1.4.5 Clayey Silt	6
1.4.6 Silty Clay	7
1.4.7 Till	9
1.5 Groundwater Conditions.....	9
2. Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS	10
2.1 Introduction.....	10
2.2 Geotechnical Design Considerations	11
2.2.1 General	11
2.2.2 Gabion Wall Base Elevation	11
2.2.3 Geotechnical Resistance at Ultimate Limit States	12
2.2.4 Geotechnical Reaction at Serviceability Limit State	12
2.2.5 Lateral Earth Pressures	13
2.2.6 Frost and Erosion Protection	15
2.3 Excavation Considerations.....	16
2.3.1 Excavation	16
2.3.2 Foundation Bedding Preparation	16
2.3.3 Site Dewatering	17
2.3.4 Backfill	17
2.4 Stability Analysis	17
2.4.1 Local Stability Analysis (Gabion Wall)	18
2.4.2 Global Stability Analysis (Embankment)	19
2.5 Possible Alternatives.....	21
2.6 Construction Schedule	21
2.7 Closure	22

Appendices

APPENDIX A: PHOTOGRAPHS

APPENDIX B: DRAWING

APPENDIX C: BOREHOLE LOGS

APPENDIX D: LABORATORY DATA

APPENDIX E: GLOBAL STABILITY ANALYSES

APPENDIX F: CROSS SECTIONS PROVIDED BY MTO

1. Part I: FOUNDATION INVESTIGATION

1.1 Introduction

This report presents the results of a geotechnical investigation completed by Trow Associates Inc. (Trow) for the Gabion wall construction near Laronde Creek. The Gabion walls will be located on the east and west side of a cantilevered pedestrian walkway at Laronde Creek Bridge on Hwy 17 about 20 km west of North Bay.

The work was undertaken under Agreement # 5006-E-0094, Assignment No. 3. The terms of reference were as presented in MTO letter dated July 30, 2009.

The purpose of the investigation is to examine the existing soil conditions within the proposed construction limits. The site specific geotechnical investigation consisted of test borings, borehole logging, and field and laboratory testing. This foundation investigation report has been prepared specifically and solely for the project described herein. It contains the factual results of the investigation and the laboratory testing.

1.2 Site Description and Geological Setting

1.2.1 Site Description

The site is located near Laronde Creek Bridge on Hwy 17, approximately 20 km west of North Bay, where Hwy 17 crosses Laronde Creek. Hwy 17 runs approximately east-west and the Laronde Creek flows north to south towards Lake Nipissing. The site plan and cross-section profiles are as shown on the drawings in Appendix B.

The existing Laronde Creek Bridge is a one span structure approximately 28.7 m long and 9.4 m wide. The bridge conveys one westbound lane and one eastbound lane of Hwy 17 over the Laronde Creek, as shown in Photograph No. 1 in Appendix A. The side slope at the south side of Hwy 17 embankment is approximately 2H:1V to 1.5H:1V. The surface at the existing bridge is at Elevation about 201.7 m and the water level in the creek is at Elevation about 195.9 m at the time of geotechnical investigation (Sept./11/2009).

The bank on the east side of the creek is relatively steep. On the west side, the bank is much flatter, as shown in Photographs 2 and 3 Appendix A. The vegetation in the area consists of shrubs and grass. However, there is a dense clump of trees and brush on the south side of Hwy 17, east of the Laronde Creek, as shown in Photograph 2, Appendix A.

The drainage in the area generally consists of road side open ditches which drain into Laronde Creek. The ditches are lined with gravel/cobble and sand at the east side of Laronde Creek, and with sand and silt at the west side, as shown in Photographs 2 and 3, Appendix A.

1.2.2 Geological Setting

The site lies within the Canadian Shield in an area where the bedrock is overlain by deep overburden. According to Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site consists of Mesoproterozoic Precambrian rocks (approximately 900 to 1600 million years old), primarily felsic igneous tonalite, granodiorite, monzonite, granite, syenite, and derived gneisses. According to Ontario Department of Mines and Northern Affairs Map 2216 (North Bay Area), the overburden consists of boulder clay, clay, varved clay, minor gravel, sand, and silt.

In general, the overburden consists of surficial sands and silts overlying thick deposit of clay.

1.3 Investigation Procedures

1.3.1 General

The field work for this investigation was performed between September 03, 2009 and September 11, 2009. The field work consisted of drilling seven (7) sampled boreholes (BH-1, BH-2, BH-3, BH-4, BH-5, BH-6, and BH-7) and installing three (3) monitoring wells in (BH-1, BH-2, and BH-5). Drawing No. 1 in Appendix B shows the locations of the seven boreholes. Boreholes BH-1, BH-6 and BH-7 were drilled on the east side of the creek, whereas boreholes BH-2, BH-3, BH-4, and BH-5 were drilled on the west side. Boreholes BH-1, BH-2, BH-4, and BH-6 were advanced near the edge of pavement in the east bound lane, approximately at Stations 13+660, 13+565, 13+585, and 13+626, respectively. Boreholes BH-3, BH-5, and BH-7 were drilled near the ditch bottom at Stations 13+578, 13+590, and 13+626. The boreholes were advanced to depths ranging from about 15.4 m to 22.3 m.

Boreholes BH-1, BH-2, BH-4, and BH-6 were advanced using a bombardier mounted CME 55 drill rig, equipped with continuous flight hollow stem augers (4-1/4" HAS). The other boreholes including BH-3, BH-5, and BH-7 were advanced using a tri-pod wash-type boring hollow stem auger (2.5" inside diameter). All borehole drilling/sampling were operated by a specialist drilling contractor, LandCore Drilling Co. Ltd.

During the drilling, soil samples were obtained using thin wall tubes (Shelby), and a 51 outside diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Tests (SPT) procedures (ASTM D 1586), at intervals shown on the attached borehole logs (Appendix C). The SPT "N" values were recorded and used to provide an assessment of in-situ consistency or relative density of non-cohesive soils. In-situ field vane testing (ASTM D 2573) was performed in the cohesive deposits to measure the in-situ undrained shear strength. The torque was measured using two calibrated scales on a lever arm threaded to the drill rod.

Following completion of the boreholes, water level measurements were obtained from the boreholes in accordance with Ministry of Transportation guidelines. Monitoring wells were installed in Borehole BH-1, BH-2, and BH-5 to permit monitoring of groundwater levels at the site. After completion, boreholes were sealed with bentonite pellets.

The fieldwork was supervised by a member of Trow's engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO Soils Classification System for foundation report, and retrieved soil samples for subsequent laboratory testing and identification. All of the recovered soil samples were placed in moisture-proof bags and returned to Trow's Sudbury and Brampton laboratories for additional visual, textual and olfactory examination.

Details of the soil strata encountered in the boreholes are included in attached borehole log sheets in Appendix C, and plotted on the profiles in Appendix B.

The borehole locations and the ground surface elevations along the cross sections were surveyed by Trow personnel, with reference to the benchmark at the south-west end corner of the concrete bridge slab at the south concrete guard rail. The final geodetic locations and elevations were established based on the site survey map provided by MTO.

1.3.2 Laboratory Testing

All samples returned to the laboratory were subjected to detailed visual examination and classification. The laboratory testing program included natural water content of all samples (LS-701), and routine classification testing of approximately 25% of the selected soil samples. The routine tests included Atterberg Limits (LS-702), grain size distribution (LS703/704), and specific gravity tests.

The laboratory test results are provided on the attached borehole log sheets in Appendix C. The results of the Atterberg Limits tests and grain size analyses are presented in Appendix D.

1.4 Subsurface Conditions

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C, and the laboratory test results are provided in Appendix D. The "Explanation of Terms Used in Report" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and cross section soil profiles are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole log and cross section soil profiles are inferred from non-continuous sampling, observations of drilling progress, results of Standard Penetration Tests, and in-situ vane shear tests. These boundaries

typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions may vary between and beyond the borehole locations.

In general, the stratigraphic sequence at the site typically consists of surficial sand fill, followed by silty sand, sandy silt, clayey silt, and a thick layer of silty clay overlying tills. The silty clay is the dominant deposit in this location.

A summary of the soil and groundwater conditions encountered in the boreholes is provided below.

1.4.1 Asphalt

At BH-1, BH-2, BH-4, and BH-6, asphalt was encountered at ground surface. The thickness of the asphalt layer ranges from 50 mm to 300 mm, and the elevation of this layer are between 201.5 m and 202.2 m.

1.4.2 Sand Fill

In all boreholes, sand fill was encountered. At BH-1, BH-2, BH-4, and BH-6, the sand fill was found directly below the asphalt. At BH-7, the sand fill was encountered at ground surface. At BH-3 and BH-5, the sand fill was overlaid by a 15 mm to 76 mm thick topsoil layer at ground surface. The thickness of the sand fill ranges from 0.6 m to 2.8 m.

The composition of this layer is sand, trace to some gravel, and trace to some silt. The fill is brown in color, and damp to wet. Uncorrected STP “N” value ranges from 2 to 25 blows per 300 mm, classifying the material as very loose to compact in compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 2% to 22%

Grain Size Distribution:

- 24% to 64% gravel;
- 71% to 30% sand; and
- 5% to 6% fines

The result of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The results of the grain size distribution tests on the sand fill are also provided on Figure 3 in Appendix D.

1.4.3 Sandy Silt

Beneath the sand fill, sandy silt was encountered BH-6. This sandy silt layer has a thickness of 4.4 m and extends to a depth of about 5.3 m below the existing grade (approximately Elevation 196.5 m). The deposit consists of silt, sand, trace to some gravel, and trace clay. The sandy silt is brown in color, and wet. Uncorrected SPT “N” values range from 2 to 11 blows per 300 mm, classifying the sandy silt as very loose to compact in compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 16 % to 28%

Grain Size Distribution:

- 10% gravel;
- 33% sand;
- 50% silt; and
- 7% clay

The result of the moisture content and grain size distribution tests are provided on the record of borehole sheet (BH-6) in Appendix C. The result of the grain size distribution test on the sandy silt is also provided on Figure 4 in Appendix D.

1.4.4 Silty Sand

Silty sandy was encountered in BH-4 underneath the sand fill layer. This silty sand has a thickness of 3 m, and extends to a depth of 6.1 m (approximately at Elevation of 195.7 m). The deposit consists of silt, sand, and trace clay. The sandy silt is brown in color, and damp to wet. Uncorrected SPT “N” values range from 2 to 7 blows per 300 mm, classifying the silty sand as very loose to loose in compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 16% to 24%

Grain Size Distribution:

- 0% gravel;
- 56% sand;

- 31% silt; and
- 13% clay

The result of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The result of the grain size distribution test on the sandy silt is also provided on Figure 5 in Appendix D.

1.4.5 Clayey Silt

A layer of clayey silt was encountered underlying the fill in BH-1 and BH-2. The thickness of the clayey silt at these locations is between 1.1 m to 1.6 m. The deposit extends to depths between 2.3 m and 3.1 m, corresponding to Elevations of approximately between 199.9 m and 198.8 m, respectively. The clayey silt contains trace to some sand, and trace to some clay. The deposit is grey in color, and damp to wet. Uncorrected SPT “N” values range from 2 to 10 blows per 300 mm, classifying the clayey silt as very loose to loose in compactness condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture Content:

- 21.4 % to 31%

Grain Size Distribution:

- 0% gravel;
- 1% sand;
- 52 % silt; and
- 47% clay

Atterberg Limits:

- Liquid Limit: 32%;
- Plastic Limit: 24%; and
- Plasticity Index: 8%

The result of the moisture content and grain size distribution tests are provided on the record of borehole sheet in Appendix C. The results of the Atterberg Limits and grain size distribution tests on the clayey silt are also provided on Figure 2 and Figure 6, respectively, in Appendix D.

1.4.6 Silty Clay

A deposit of silty clay was encountered in all boreholes. The top of this deposit ranged from Elevation 195.7 m to 199.9 m. The silty clay has thickness between about 18.2 m to 20 m, and extends to Elevation ranging from about 180.8 m to 180.5 m at BH-1 and BH-2, respectively. The other boreholes, BH-3, BH-4, BH-5, BH-6, and BH-7, were terminated in the silty clay deposit at Elevations in the range of about 179.6 m to 185.7 m.

The silty clay is grey in color and saturated. It is varved with clayey silt. The thickness of individual layers or laminations varies from a few millimeters to a few centimeters, but in general is about one centimeter. The portion of silty clay and clayey silt varies from about 2:1 to 5:1, and the clay portion in general dominates.

Uncorrected SPT “N” values range from 0 (weight of hammer) to 18 blows per 300 mm of penetration. *In-situ* field vane tests were performed to measure undrained shear strengths of the silty clay. The results of the *in-situ* field vane tests measured in the boreholes are shown on the record of borehole sheets in Appendix C and Figure 1.1. The *in-situ* vane shear strength ranges from 15 kPa to 90.1 kPa, indicating a soft to stiff consistency. Sensitivity ranges from 2 to 5 (the average sensitivity is about 3), classifying the silty clay as low sensitivity according to Canadian Foundation Engineering Manual 2006 (CFEM, Chapter 3, page 18).

Figure 1.1 summarizes the measured field vane shear tests that were carried out as part of the current investigation. The results show that the silty clay has a crust extending to about 5 m depth below the ground surface. The typical vane strength is about 35 kPa for the crust layer. Beneath the crust, the typical vane strength reduces to 30 kPa at a depth of 10 m and then increases with depth at a rate of about 4 kPa/meter.

Laboratory testing performed on selected samples consisted of moisture content, grain size distribution, and Atterberg Limits. The test results are as follows:

Moisture Content:

- 22% to 68%

Grain Size Distribution:

- 0% gravel;
- 0% to 2% sand;
- 25% to 70% silt; and
- 28% to 73% clay

Atterberg Limits:

- Liquid Limits: 31% to 38%

- Plastic Limits: 19% to 25%
- Plasticity Index: 8% to 16%

The results of the moisture content, grain size distribution, and Atterberg Limits are provided on the record of borehole sheets in Appendix C. The results of the grain size distribution tests on the silty clay are provided on Figures 7 and 8 in Appendix D. The results of the Atterberg Limits tests are provided on Figure 1 in Appendix D.

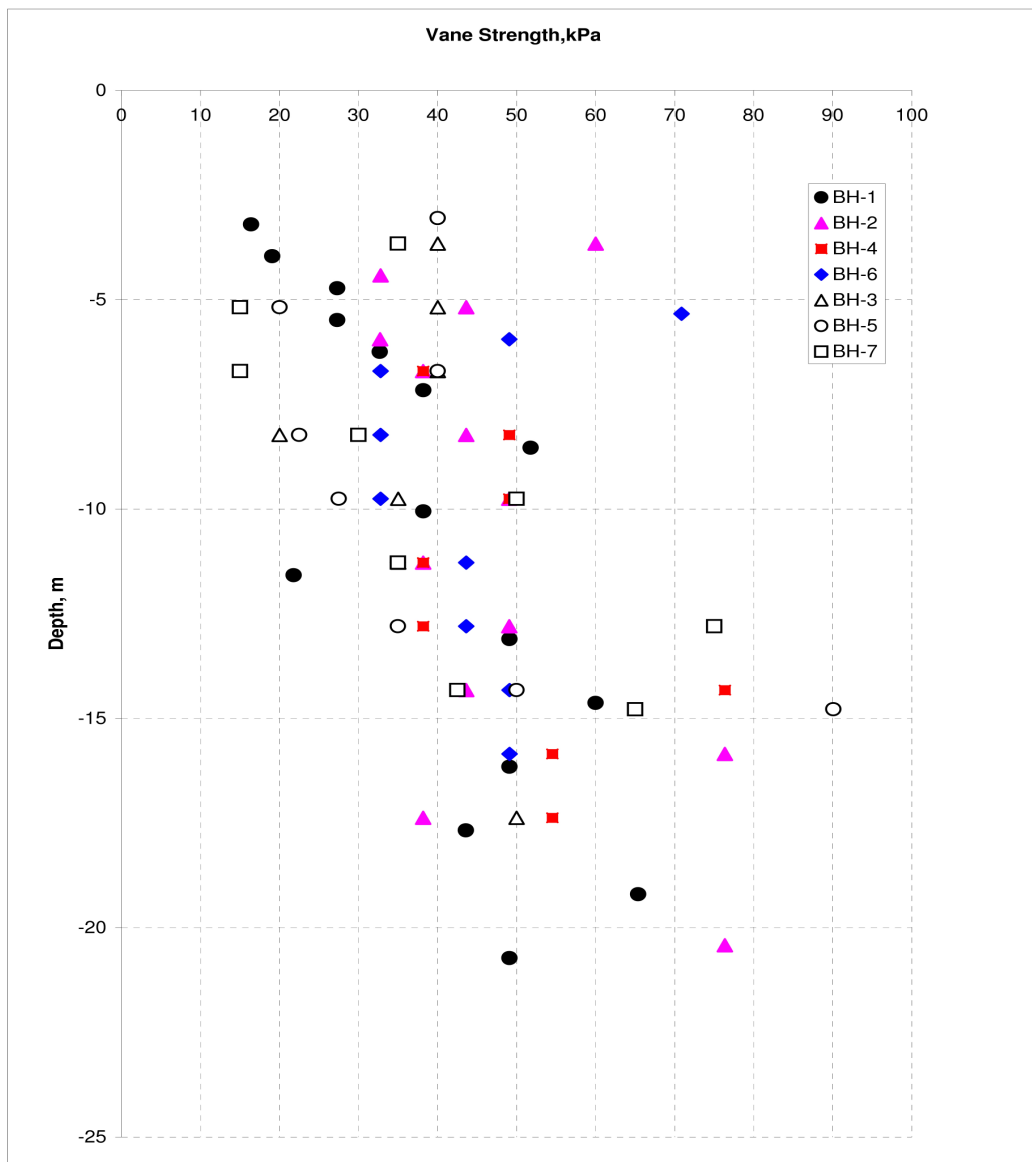


Figure 1.1 Vane strength of the silty clay measured on the site

1.4.7 Till

Beneath the silty clay, silty sandy till and suspected sand and gravel till were encountered at BH-1 and BH-2, respectively. The top elevation of the till ranges from Elevation 180.5 m to 180.8 m, corresponding to a depth of 21.3 m below ground surface. The till extends to about 22.3 m depth at the borehole termination due to auger refusal. The till is grey in color and wet.

1.5 Groundwater Conditions

Information regarding to the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and in monitoring wells installed in Boreholes BH-1, BH-2, and BH-5. The ground water levels encountered in the boreholes are also shown in Table 1.1.

The monitoring wells consists of a 6 m long slotted screen embedded in a sand pack and bentonite seals above and below the sand pack.

The difference in groundwater level between boreholes could be due to disturbance in the holes a time of drilling and that the boreholes had not stabilized prior to backfilling. It should be noted that the groundwater level is subject to seasonal fluctuations.

Table 1.1 Groundwater levels recorded at the site

Borehole No.	Date of drilling	Well tip depth, (m)	Water level	
			Depth, (m)	Elevation, (m)
BH-1*	09/03/2009	12.2	0.9	201.3
BH-2*	09/04/2009	12.2	1.52	200.4
BH-3	09/08/2009	Open hole	0.84	197.4
BH-4	09/09/2009	Open hole	3.05	198.7
BH-5*	09/09/2009	11.2	0.2	196.6
BH-6	09/10/2009	Open hole	0.91	200.9
BH-7	09/10/2009	Open hole	1.5	196.9

*Monitoring well GWL reading was taken on 09/11/2009

2. Part II: ENGINEERING DISCUSSIONS AND RECOMMENDATIONS

2.1 Introduction

The purpose of the following subsections is to provide recommendations for the design and construction of the retaining structure to support the proposed pedestrian walkway. The proposed 1.5 m wide walkway on the south side of the Hwy 17 requires an extension of embankment and consequently Gabion retaining walls to support the widened road platform have been proposed. The Gabion walls will be on the east and west sides of Laronde Creek to facilitate the walkway construction.

As indicated, in order to facilitate construction of the walkway, it is necessary to widen the embankment. The current proposal from MTO is to use Gabion baskets up to 2.5 m high, to avoid encroachment into the ditch and/or adjacent properties. For much of the alignment, this height of Gabion wall or less will result in the walkway being at or near the top of the Gabions. Adjacent to the bridge structure the embankment is higher and two possibilities exist to accommodate the proposed walkway extension. (a) The Gabion walls can follow the ditch line, in which case there would be a slope down to the top of Gabion from the guardrail containing the walkway at the top of slope. The side slope must be at least 2H:1V or flatter and fit the geometric constraints (likely from station 13+570 to 13+590 at the west side of the creek, and from 13+620 to 13+630 at the east side). (b) Alternatively, but less desirably, the top of Gabions can be kept level with the walkway and be founded on the side slope. For bearing capacity and stability, this is less positive and would likely require additional support in the form of Helical piles such as Chance anchors advanced below the Gabions for a section of the walkway zone near the bridge (from station 13+570 to 13+590 at the west side, and from 13+620 to 13+630 at the east side).

As an alternative to the current proposal, it would be feasible to accommodate the walkway using a retaining structure such as a permanent soldier pile and lagging scheme. This would incorporate galvanized soldier piles concreted in auger holes at interval of about 1.5 m with suitably designed concrete lagging. Penetration of the soldier piles would be expected to be about 2.5 to 3 times the retained height if the structure is cantilevered. Costs would likely be higher than the Gabion scheme, but this may present a more elegant solution. The guard rail could be incorporated directly onto the soldier piles.

For purposes of the report, the intended scheme, i.e., setting the Gabions at the ditch line is developed in more detail.

This report will address the geotechnical design of the retaining wall by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the Canadian Highway Bridge Design Code (November 2006), the

Canadian Foundation Engineering Manual (2006), and good practice. Pertinent construction issues from a geotechnical standpoint are examined in general accordance with the Terms of Reference from MTO letter dated 30 July, 2009.

2.2 Geotechnical Design Considerations

2.2.1 General

The geotechnical investigation and its findings pertaining to the subsurface soil characteristics have been covered in **Part I - Foundation Investigation Report** which contains details of the field and laboratory aspects of the investigation.

In the context of the Canadian Highway Bridge Design Code (CHBDC), a satisfactory retaining wall design would require, in terms of Limit States Design, the factored geotechnical resistance of its foundation to withstand and not exceed the imposed Ultimate Limit State loads - (ULS) Design Approach, and its ability to deform acceptably under the Service Limit State loads - (SLS) Design Approach. These associated loads are typically known as unfactored and factored loads, respectively.

In terms of the Strength Limit States design, the retaining wall foundation will derive its geotechnical resistance to the superimposed factored loads from the subgrade soils over a depth of subsoil below the foundation equivalent to the width of the wall base. The depth of soil to be mobilized in relation to a classical bearing capacity failure is approximately the base width of the retaining wall.

Under the existing roadway, the soil has been pre-consolidated by the pressures from the embankment and has not been where the proposed walkway is constructed outside of the current embankment. Consequently, there is potential for differential settlement to occur at the proposed walkway where the embankment width is increased.

2.2.2 Gabion Wall Base Elevation

The base of Gabion walls will be located at the level of the bottom of the open ditch or embedded to a certain depth below the ditch bottom sufficient to avoid disturbance from erosions. The ditch bottom elevation decreases from about Elev. 201 m to about Elev. 197 m as the creek is approached; similarly, the height between ditch bottom and road surface increases.

The elevation of the retaining wall base will likely be located on the sand fill, silty sand, sandy silt, clayey silt, and silty clay, as shown in the soil strata cross-sections of A-A, B-B, and C-C (Drawings 2 and 3 in Appendix B). At the three boreholes close to the ditch centre (BH-3, BH-5 and BH-7), the sand fill and sandy silt overlying the silty clay has a thickness ranging from 1.5 m to 2.5 m. As a result, the foundation soil may encompass both silty clay

and cohesionless subsoils (sand fill, silty sand, sandy silt), assuming a 2 m width of the Gabion wall base.

2.2.3 Geotechnical Resistance at Ultimate Limit States

Based on the results of the geotechnical investigation, for an assumed 2 m wide footing the design bearing capacities recommended as per the CHBDC are as follow:

- Ultimate Geotechnical Resistance at Ultimate Limit State of the foundation soil is about 100 kPa
- Factored Geotechnical Resistance is 50 kPa using a Geotechnical Resistance factor of 0.5.

2.2.4 Geotechnical Reaction at Serviceability Limit State

Serviceability Limit States generally consider the unfactored loads being used to determine total and differential settlements of the structure with the magnitude of unfactored loads and tolerable total and differential settlement limits being established by the Structural or Design Engineer.

In determining the settlement characteristics of the retaining wall, the unfactored loads are required to be provided by the Structural or Design Engineer. However, if we assume that 25 mm of settlement is acceptable, then the geotechnical reaction at the Serviceability Limit States can be determined from the method recommend by Bowles (Foundation Analysis and Design, by E.J. Bowles, the 4th Edition) to provide a Serviceability Limit State Reaction of 50 kPa. It is also noted that Gabion wall is typically considered as a flexible structure, which can sustain some settlement.

2.2.5 Lateral Earth Pressures

The retaining walls should be designed for lateral earth pressures according to the following expression, assuming a triangular pressure distribution:

$$p = k(\gamma h + q)$$

where p = the pressure in kPa acting against the wall surface at depth, h , below the ground surface

k = lateral earth pressure coefficient;

γ = the bulk unit weight of the retained backfill;

h = depth below the ground surface at which the pressure, p , is to be computed; and

q = the value of any adjacent surcharge in kPa which may be acting close to the wall (including traffic loads).

The above equation assumes that a sub-drainage is provided at the founding level, together with free-draining granular backfill adjacent to the wall, to prevent the build-up of hydrostatic pressure behind the wall.

Backfill should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation (MTO) Standards.

For design purposes, the following physical properties can be used.

Compacted Granular "A"

Angle of Internal Friction (ϕ) = 35° (unfactored)

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular "B"

Angle of Internal Friction (ϕ) = 32° (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31$$

$$K_o = 0.47$$

NOTE: K_a is the active earth pressure coefficient for a soil loading an unrestrained structure; and

K_o is the earth pressure coefficient at rest for a soil loading a restrained structure.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. For this case the active condition will apply since it is assumed that some movement would be permissible. These values also assume a horizontal backfill condition.

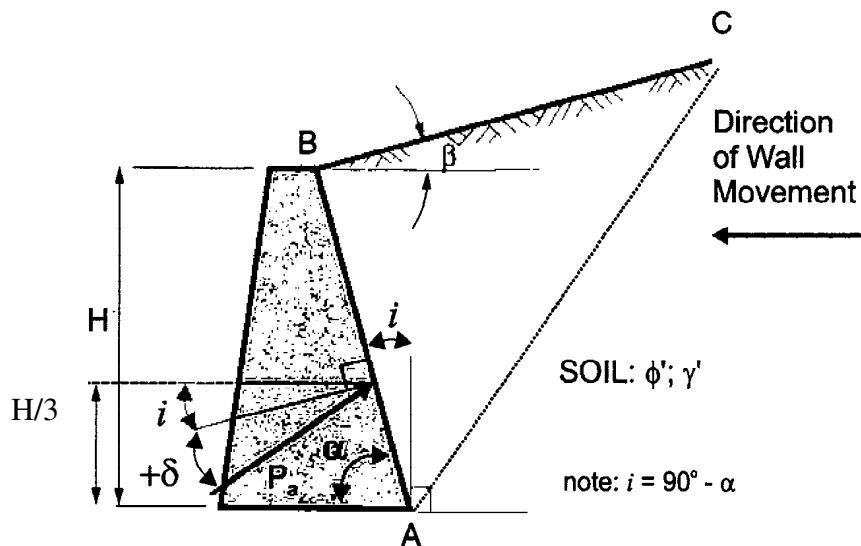


Figure 2.1 The illustration of Coulomb's theory (from page 376, CFEM, the 4th Edition).

For sloping backfill, the following expression can be used (see page 376 in Chapter 24, CFEM, 4th edition).

$$K_a = \cos(\delta + i) \frac{\sin^2(\alpha + \phi')}{\sin^2 \alpha \sin(\alpha - \delta) \left[1 + \frac{\sin(\delta + \phi') \sin(\phi' - \beta)}{\sin(\alpha - \delta) \sin(\alpha + \beta)} \right]^2} \quad \text{Note: } K_a \text{ in horizontal direction}$$

where δ is the friction angle between retaining wall and backfill soil, which can be taken as 10 degree in this case. $P_a = 1/2 \gamma H^2 K_a / \cos(\delta + i)$ in direction of δ as shown in Figure 2.1

The following provides a specific sloping backfill case to demonstrate the application of the above expression.

Parameters assumed:

$\beta = 26$ degree (i.e., about a 2H:1V slope);

$i = -6$ degree (i.e., a 6 degree of batter angle of the Gabion wall); and

$\delta = 10$ degree

Compacted Granular “A”

Angle of Internal Friction (ϕ) = 35° (unfactored)

Unit Weight = 22 kN/m³

Coefficient of Active Lateral Earth Pressures (*if the above parameters are assumed*):

$$K_a = 0.32 \text{ (for the sloping back fill case)}$$

Compacted Granular “B”

Angle of Internal Friction (ϕ) = 32° (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Active Lateral Earth Pressures (*if the above parameters are assumed*):

$$K_a = 0.39 \text{ (for the sloping back fill case)}$$

The effects of compaction surcharge should be taken into account in the calculations of active and at rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

Vibratory compaction equipment for use behind the retaining wall should be restricted in size as per current MTO practice. Traffic load on the road should be taken into account.

2.2.6 Frost and Erosion Protection

The Gabion wall is flexible, allowing frost movement without fractures.

Rip-rap protection should be provided at the bottom of the ditch, which may be realigned in accordance with the construction of the Gabion wall. As a rule of thumb the thickness of the rip-rap should be a minimum of twice the median particle size, and 300 mm thick as a minimum. Rip-rap placed at 1V:1H will be stable. Rip-rap blanket may be used if necessary.

For the slope from the walkway shoulder to the top of Gabion wall (if applicable), erosion protection using deep rooted vegetation or a gravel blanket should be provided in accordance with MTO practice.

2.3 Excavation Considerations

2.3.1 Excavation

Since the Gabion wall will be placed to retain the soil to support the walkway, excavation of the existing embankment will be necessary.

All excavations must be conducted in accordance with the Occupational Health and Safety Act and Regulations for Construction (OHSARC). The sand and silt material may be classified as a Type 3 soil above the groundwater table and a Type 4 soil below the groundwater table, in conformance with the OHSARC. It should be feasible to excavate these soils using hydraulic equipment.

For the purpose of this construction, the existing cohesionless material may be classified as Type 3 soils, provided that there is no significant groundwater seepage. Temporary excavation side slopes for Type 3 soils should not be steeper than 1H:1V. Temporary excavation side slopes for Type 4 soils should not be steeper than 3H:1V. Where practical the excavation face should be benched. For the anticipated geometries, it should be practical to undertake the work safely with the anticipated temporary cuts. For the Case 2 scenario where there will be sloping ground above the top of Gabion, it is assumed that the slope would be at least 2.5H to 1V or flatter. The Gabion would have to be set back to accomplish this. In this case, the cut to facilitate construction of the wall should be about 1.5 m. Any seepage from the cut face should be controlled by appropriate dewatering or the face should be covered with geotextile and granular fill to avoid minor sloughing. This condition will be better if the work is done in the drier summer months. The work should be completed as quickly as practical and excavation should not be left exposed for extended periods.

As indicated, there is a potential for sloughing to occur potentially near the bottom if the excavation remains open for an extended period of time (i.e. 24-48 hours) or during a rainfall event. When excavations cannot be safely sloped to maintain stability during construction, temporary shoring suitably designed may be used. Since the proposed work for the construction is located in the vicinity of the highway embankment, it will be the Contractors responsibility to design a suitable temporary support system. This design and installation procedures are to be provided to MTO for review prior to installation. In addition, the Contractor is to follow SPN.105S19 regarding protection systems.

2.3.2 Foundation Bedding Preparation

Gabion walls are typically placed directly on a graded soil foundation. To increase the bearing capacity, minimize differential settlement, and/or allow for additional drainage, a base layer of granular stone fill with consistent gradation according to the design may be placed and compacted to 0.15 m to 0.8 m in depth as a founding course for Gabion wall placement. Prior to placement of any granular material, the area should be stripped of all top

soil, organic and other deleterious material and the exposed subgrade proofrolled with a vibratory compactor. Vibration should not be used if any pumping is observed. The foundation bedding may consist of suitable fill materials compacted to 95% of the Standard Proctor Maximum Dry Density.

2.3.3 Site Dewatering

Some excavations may be below the groundwater levels measured during this investigation. To avoid disturbance of the founding materials, to facilitate the designed excavation slope, and to allow placement of fill in dry conditions, groundwater must be controlled to below the proposed excavation levels during the construction.

The method used should not undermine the existing road. It is the responsibility of the Contractor to propose a suitable dewatering system based on groundwater levels at the time of construction and the Contractor should submit his proposal for prior approval of the MTO prior to construction. For the shallow excavation proposed, it should be possible to control seepage by pumping or drainage from perimeter ditches in slightly oversize excavations. This should be confirmed during construction.

2.3.4 Backfill

The backfill should consist of Granular “B”, or Granular “A” (OPSS 1010) placed in layers not exceeding 300 mm in thickness for the full width of the trench and each layer should be compacted to 95% of the Standard Proctor Maximum Dry Density before a subsequent layer is placed according to OPSS 514. It should be free-draining backfill to prevent the build-up of hydraulic pressure behind the wall. Alternatively, clear crushed stone (19mm) may be used.

Geotextile filter fabric should be placed between the Gabion wall and the specified backfill material interface to prevent loss of retained soil during drainage. If clear crushed stone is used as backfill, suitable geotextile will have to be extended along the face of the cut, which should be benched wherever practical.

2.4 Stability Analysis

As assessment has been performed to evaluate stability of the Gabion walls and the impact of the proposed construction on the stability of the existing embankment. The analyses were performed using the classic Coulomb’s theory to evaluate Gabion wall stability and the conventional limit equilibrium method (proposed by Morgenstern-Price) to assess the global stability of the embankment.

The embankment slope becomes steeper and ditch bottom gets deeper approaching the creek. To account for the change in cross section profile, two cases were chosen for stability analysis:

- 1) Case 1 : Typical cross-section perpendicular to Hwy 17 where the top of Gabions is kept level with the walkway. In this case, the Gabion wall is assumed to have a height of 2.5 m and a 6 degree batter angle towards retained slope. The geometry for the cross section (Case 1) is shown in Drawing 4 in Appendix B.
- 2) Case 2: Critical cross-section at Station 13+620 on the east side of the creek. The geometric profile of this cross-section was obtained directly from the survey cross-section drawing provided by MTO (Sept. 21, 2009). The Gabion wall is located at the ditch bottom with a 2H:1V slope connecting its top to the edge of the walkway shoulder. The geometry for the cross section (Case 2) is shown in Drawing 4 in Appendix B. Additional assessments are shown for flatter slope angles.

2.4.1 Local Stability Analysis (Gabion Wall)

Stability analysis on the Gabion wall is to check: a) overturning stability; b) sliding; c) the eccentricity of resultant force on the wall base; and d) bearing capacity. The parameters utilized in the analyses are presented in Tables 2.1 and 2.2. Table 2.3 summarizes the results of the local stability analysis for the Gabion wall.

Table 2.1 Parameters utilized in Case 1.

Descriptions	Input Values	Units
Backfill slope angle above wall	0	degree
Angle of internal friction	35	degree
Angle of wall friction	10	degree
Inclination angle to vertical plane	6	degree
Cohesion	0	kN/m ²
Surcharge (including traffic load)	16.8	kN/m ²
Soil density	20.4	kN/m ³
Gabion density	18.8	kN/m ³
Height of wall	2.5	m
Width of base	2.0	m
Embedment (0.2 m as a conservative value for stability analysis)	0.2	m
Allowable soil bearing capacity	50	kPa

Table 2.2 Parameters utilized in Case 2.

Descriptions	Input Values	Units
Backfill slope angle above wall	26.5	degree
Angle of internal friction	35	degree
Angle of wall friction	10	degree
Inclination angle to vertical plane	6	degree
Surcharge	12.0	kN/m ²
Soil density	20.4	kN/m ³
Gabion density	18.8	kN/m ³
Height of wall	2.5	m
Width of base	2.0	m
Embedment (0.2 m as a conservative value for stability analysis)	0.2	m
Allowable soil bearing capacity	50	kPa

Table 2.3 Summary of the local stability analyses

	Case 1	Case 2
overturning stability	FS=4.6 (≥2)	FS=3.8 (≥2)
sliding	FS=1.8 (≥1.5)	FS=1.5 (≥1.5)
eccentricity of resultant force (resultant is in middle one third)	satisfied	satisfied
bearing capacity	satisfied	satisfied

2.4.2 Global Stability Analysis (Embankment)

The global stability analyses were performed using the SLOPE/W computer program developed by GeoSlope International. An optimization algorithm in SLOPE/W was adopted for auto-search or auto location of the most critical potential slip surface. Again, each of the two cases (Case 1 and Case 2) was evaluated.

The SLOPE/W printouts, for Case 1 and Case 2, are included in Appendix E. Soil properties used for analyses are presented on the graphs as well. The stratigraphy at the site was developed based on the results of the geotechnical investigation presented in Part 1

Foundation Investigation. The groundwater level was defined based on its measurements during the geotechnical investigation.

Figure 1 in Appendix E shows the result of the global stability analysis on the typical cross-section (*Case 1*) perpendicular to Hwy 17. It can be seen that the calculated factor of safety is about 1.4.

Figure 2 in Appendix E shows the result of the global stability analysis on the cross-section at Station 13+620 (*Case 2*) perpendicular to Hwy 17. It can be seen that the calculated factor of safety is about 1.2, which is marginal. Additional two scenarios are shown in Figures 3 and 4, Appendix E. Figure 3 indicates that the factor safety (F.S.) would be higher (i.e., F.S.=1.3) if the slip surface extends into the silty clay in *Case 2* stability analysis. Similarly, Figure 4 shows that assuming the wall base founded on the silty clay would lead to a more favorable result in stability analysis (i.e., a higher factor of safety, F.S. =1.5).

Since the calculated factor of safety (1.2) is marginal, other measures should be taken to improve the global stability. This could include measures such as flattening the side slope between the edge of the walkway and the Gabion top at least 2.5H:1V, or incorporate Helical piles such as Chance anchors at the base. The affected section is approximately from Station 13+570 to 13+590 at the west side of the creek and from 13+620 to 13+630 at the east side, based on the geometries interpreted from cross-sections provided by MTO (see Appendix F).

Global stability analysis was performed on the scenario corresponding to the backfill slopes of 2.5H:1V. The calculated factor of safety is 1.4, as shown in Figure 5 in Appendix E.

In addition, stability analyses of temporary excavations for the scenario with the backfill slope of 2.5H:1V were performed to evaluate the short-term stability for the temporary excavation condition. In this case of temporary excavation, the upper soil layer in Figure 6 is assumed to have a cohesion of 5 kPa. The calculated factor of safety is about 1.2, as shown in Figure 6 in Appendix E. It is recommended that the groundwater should be controlled to at least the level presented in Figure 6. The excavation in the critical areas should be carried out in short sections (5m or less) and the geotextile and granular fill following closely and on the same day. Minor excavations can then be done to seat the gabions.

2.5 Possible Alternatives:

The following concepts are presented for possible considerations:

(a) Soldier Piles with Concrete Laggings

An alternative to the Gabion wall is soldier pile wall with concrete lagging (permanent shoring). This will avoid excavating into the embankment slope. The steel soldier piles are set into drill holes close to the edge of the proposed walkway. The drill holes should be filled with concrete to the level of the encountered embankment slope. The concrete laggings are placed between the soldier piles. Any voids behind the concrete laggings should be filled with pit run Granular 'B' or approved free draining material. A filter fabric should be placed behind the concrete laggings.

The soldier piles should be galvanized, and the concrete laggings treated for corrosion protection, using epoxy coated rebar. If this is to be utilized, specific embedment design must be developed. For preliminary guidance and assuming a cantilevered structure, an embedment depth is about 2.5 to 3 times the retained height is anticipated. This should be confirmed by more rigorous analysis if this scheme is to be considered.

(b) Culvert in the Roadside Ditch

Another possibility to consider is to place a suitably designed section of culvert (box or pipe) within the existing ditch and fill over the top to safely accommodate the embankment widening for the walkway. A 2H:1V slope incorporating the walkway is envisioned. A swale would have to be developed at the base to handle surface run-off. This scheme could also require further development for feasibility including cost analysis.

2.6 Construction Schedule

This work can be done best in dry summer months when the ground is firm with no groundwater seepage and placement and compaction of material is easily accommodated. Construction during winter periods is less desirable but should be possible provided that:

- a) At foundation levels for the Gabions, the area is cleared of all ice, snow, and deleterious materials and the base suitable prepared. The work should be done in sections and completed without delay.
- b) Compaction of fill materials during cold weather is difficult and sometimes impractical. Granular material will be required. Some over excavation and wasting of materials, and possibly some maintenance in the spring should be anticipated. Vibratory equipment should only be used if pumping does not occur.

Based on the above, this work should be done during warmer dry periods whenever practical.

2.7 Closure

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations as well as their own interpretations of the factual borehole results so that they may draw their own conclusions as to how the subsurface conditions may affect them

This Foundation Investigation and Design Report has been prepared by S. Micic, Ph.D., P.Eng and G. Qu, Ph.D., and reviewed by S. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact.

We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

Trow Associates Inc.



Silvana Micic, Ph.D, P.Eng.
Geotechnical Engineer



Encl.



S.E. Gonsalves, M.Eng., P.Eng.
Principal Engineer
Designated MTO Foundation Contact



APPENDIX A : PHOTOGRAPHS



Photograph 1 Laronde Creek Bridge on Highway 17, south side



Photograph 2 The east side of Laronde Creek, south side of Highway 17



Photograph 3 The west side of Laronde Creek, south side of Highway 17

APPENDIX B : DRAWING

DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

GWP

No. 5274-08-00

SITE PLAN AND
BOREHOLE LOCATIONS

N

SHEET
1



KEY MAP
Not to Scale

LEGEND

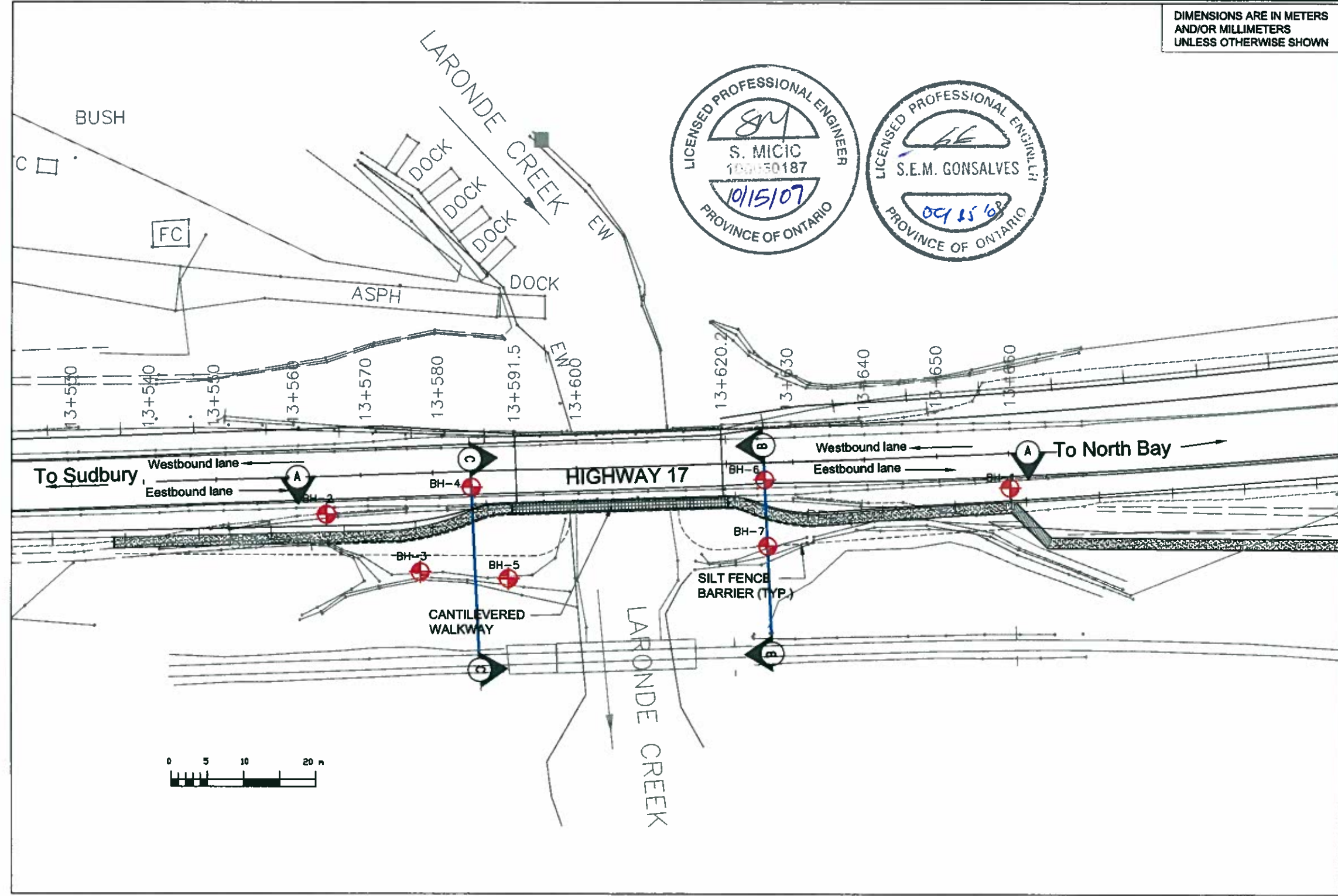
BOREHOLE

Water Level (Piezometer)

Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

REVISIONS	DATE	BY	DESCRIPTION



Trow Associates Inc.

56 QUEEN STREET EAST, SUIT 301
BRAMPTON, ONTARIO, L6V 4M8
(905) 796-3200

PROJECT TITLE AND LOCATION:

Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury

DRAWING TITLE:

SITE PLAN AND
BOREHOLE LOCATIONS

PROJECT NO.

5274-08-00

DWN.:

GQ

SCALE:

AS NOTED

CHKO.:

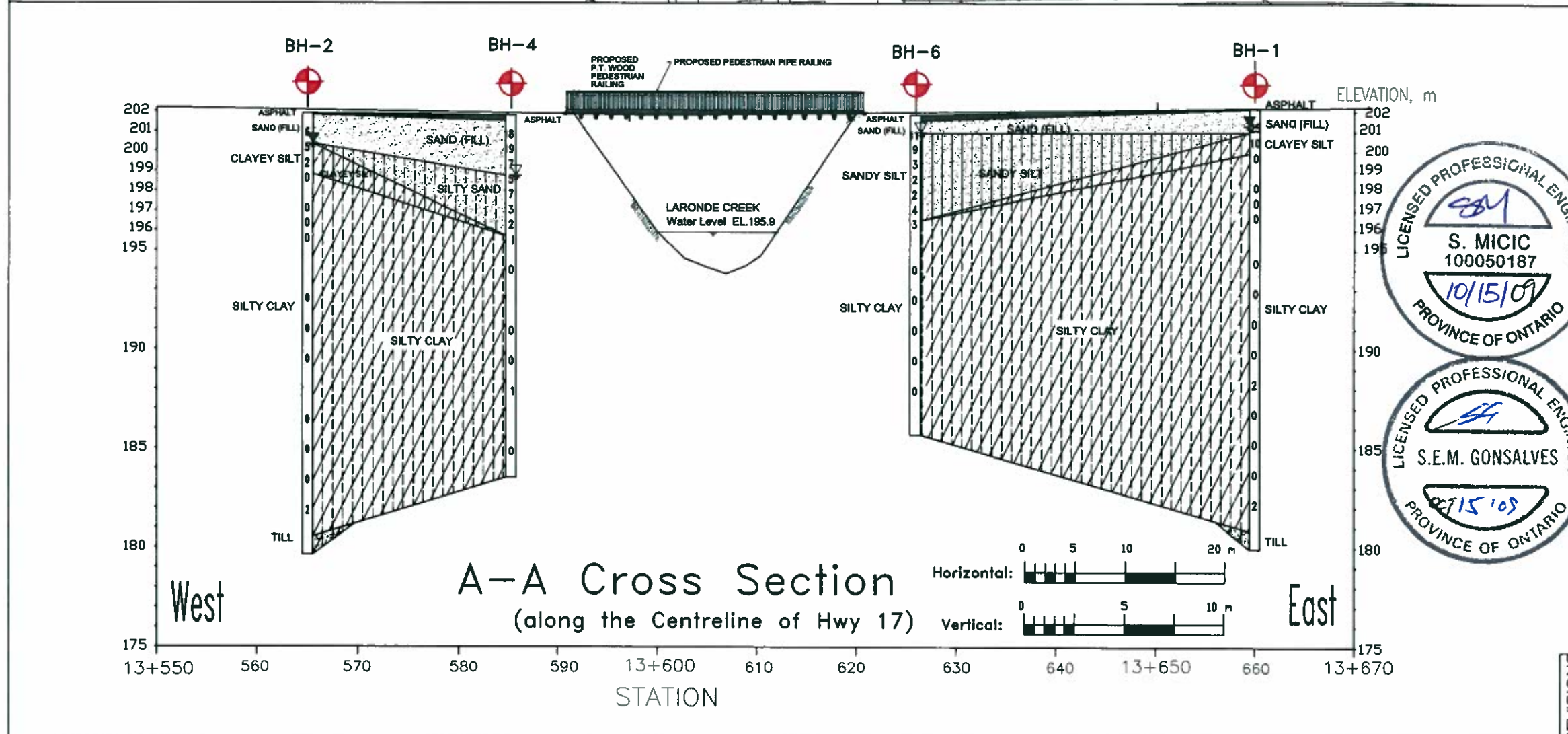
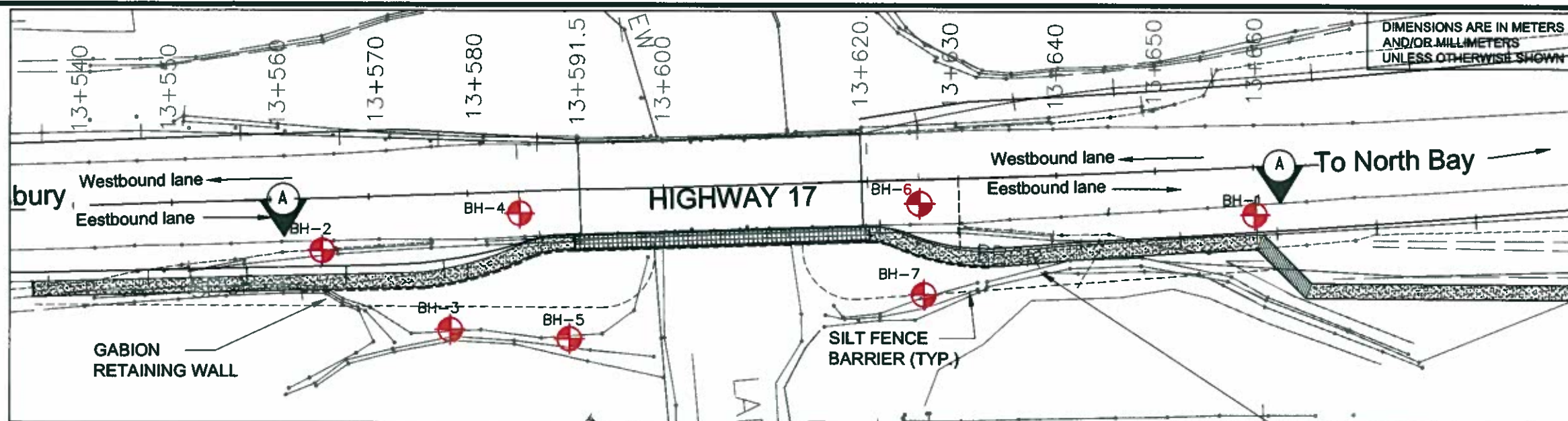
SM

DATE:

Sept. 2009

DWG. No.:

1



KEY MAP
Not to Scale

- LEGEND
- BOREHOLE
 - Water Level (Piezometer)
 - Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

- NOTES
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
 - This drawing is to be read with subject report.
 - This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
 - Borehole locations are approximate.
 - Borehole elevations should not be used to design building(s), or floor slab(s), or parking lot(s) grades.
 - The elevation of the water level in the creek was measured by TROW on 11/Sept./2009

REVISIONS	DATE	BY	DESCRIPTION



DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

GWP No. 5274-08-00



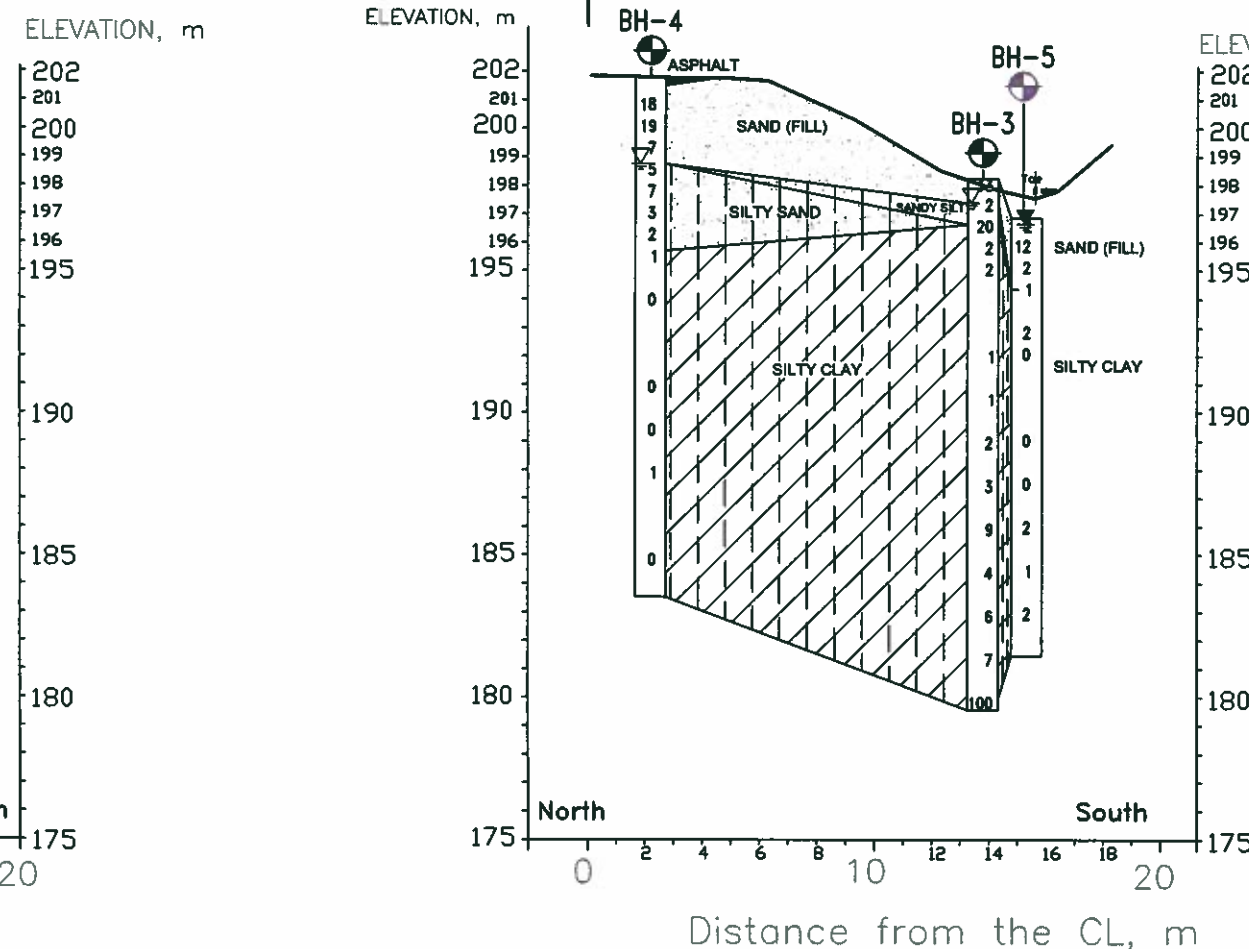
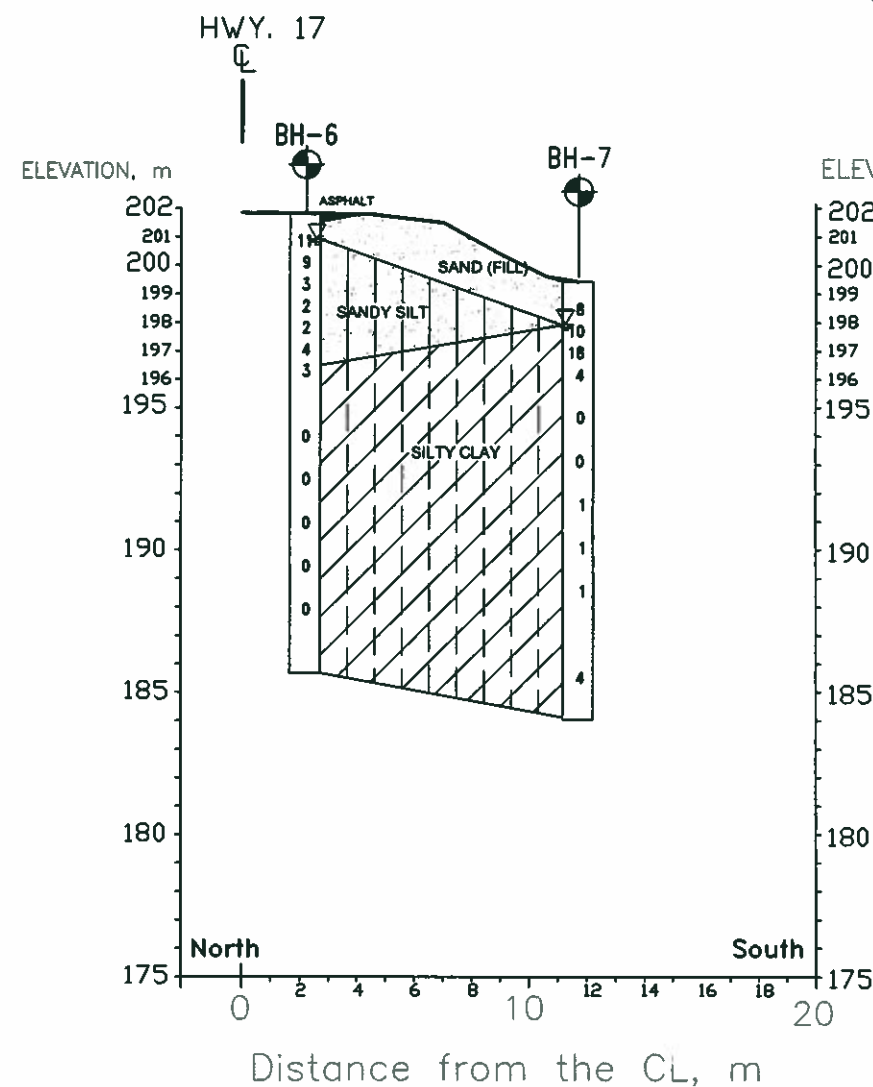
SHEET

3

Cross Section B-B
(at Station ~13+626)



Cross Section C-C
(at Station ~13+585)



KEY MAP
Not to Scale

LEGEND

- BOREHOLE
- Water Level (Piezometer)
- Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

-NOTES-

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- The elevation of the water level in the creek was measured by TROW on 11/Sept./2009

REVISIONS	DATE	BY	DESCRIPTION

SOIL STRATA SYMBOLS:

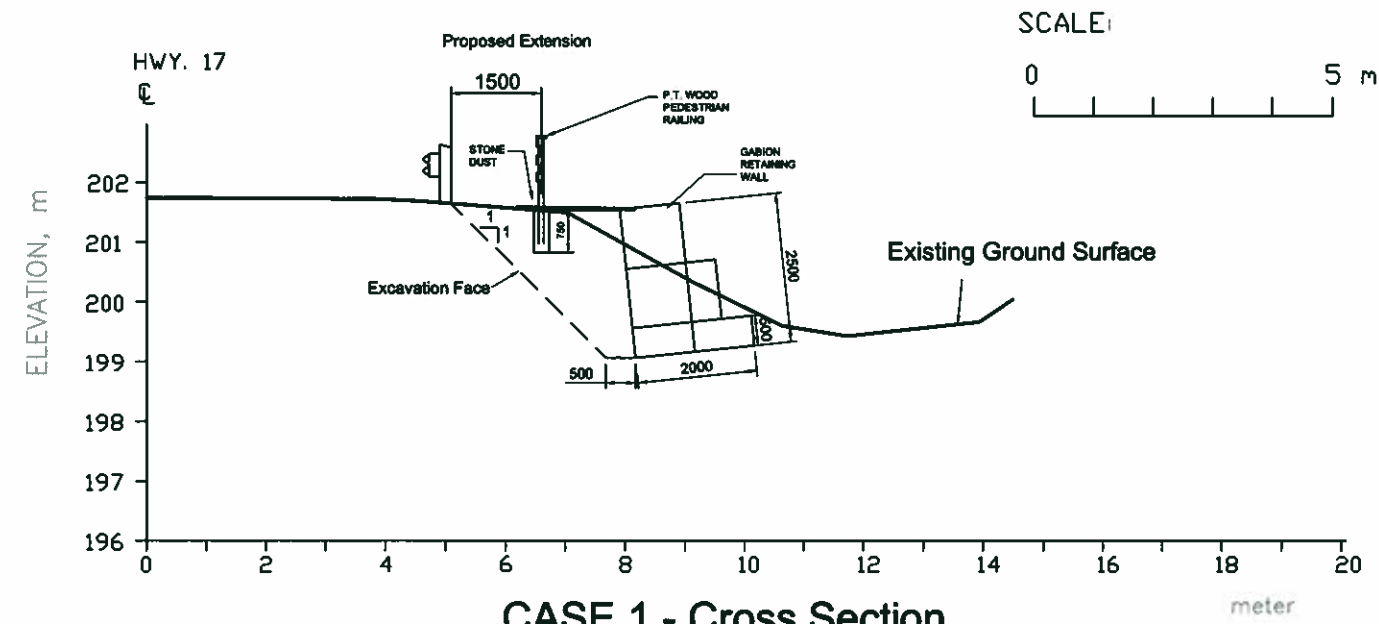
ASPHALT	SAND	SILTY CLAY
SILTY SAND	CLAYEY SILT	TILL

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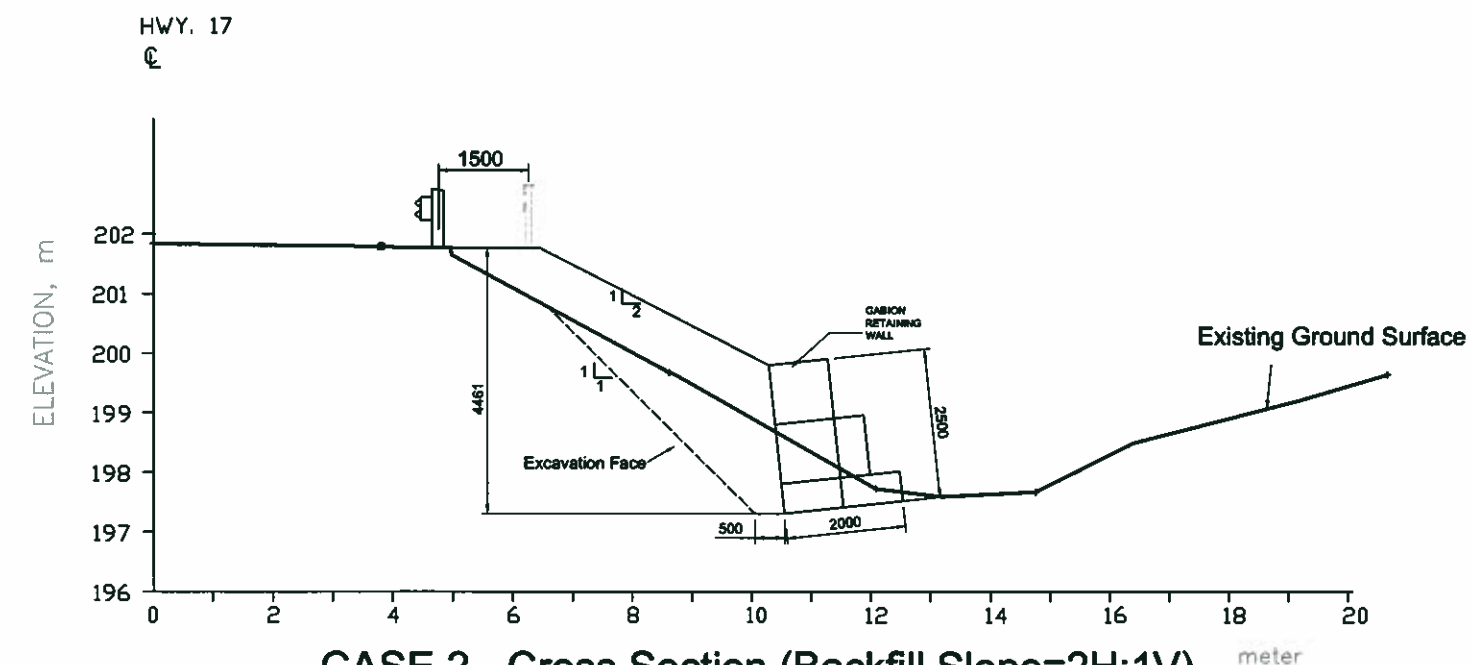
PROJECT TITLE AND LOCATION:
**Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury**

DRAWING TITLE:
**CROSS-SECTIONS
B-B and C-C**

PROJECT NO. 5274-08-00	DWN.: GQ
SCALE: AS NOTED	CHKD.: SM
DATE: Sept. 2009	DWG. No.: 3



CASE 1 - Cross Section



CASE 2 - Cross Section (Backfill Slope=2H:1V)
(at Station about 13+620)

DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN



GWP No. 5274-08-00



SHEET

4



KEY MAP
Not to Scale

LEGEND

- BOREHOLE
- Water Level (Piezometer)
- Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

NOTES

1. The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2. This drawing is to be read with subject report.
3. This drawing is for subsurface information only. Surface details and features are for conceptual illustration only.
4. Borehole locations are approximate.
5. Borehole elevations should not be used to design building(s), or floor slab(s), or parking lot(s) grades.
6. The elevation of the water level in the creek was measured by TROW on 11/Sept./2009

REVISIONS	DATE	BY	DESCRIPTION

Trow Associates Inc.
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PROJECT TITLE AND LOCATION:
**Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury**

DRAWING TITLE:
**CROSS-SECTIONS
CASE 1, and CASE 2**

PROJECT NO. **5274-08-00**
SCALE: **AS NOTED**
DATE: **Sept. 2009**
DWN.: **GQ**
CHKD.: **SM**
DWG. No.: **4**

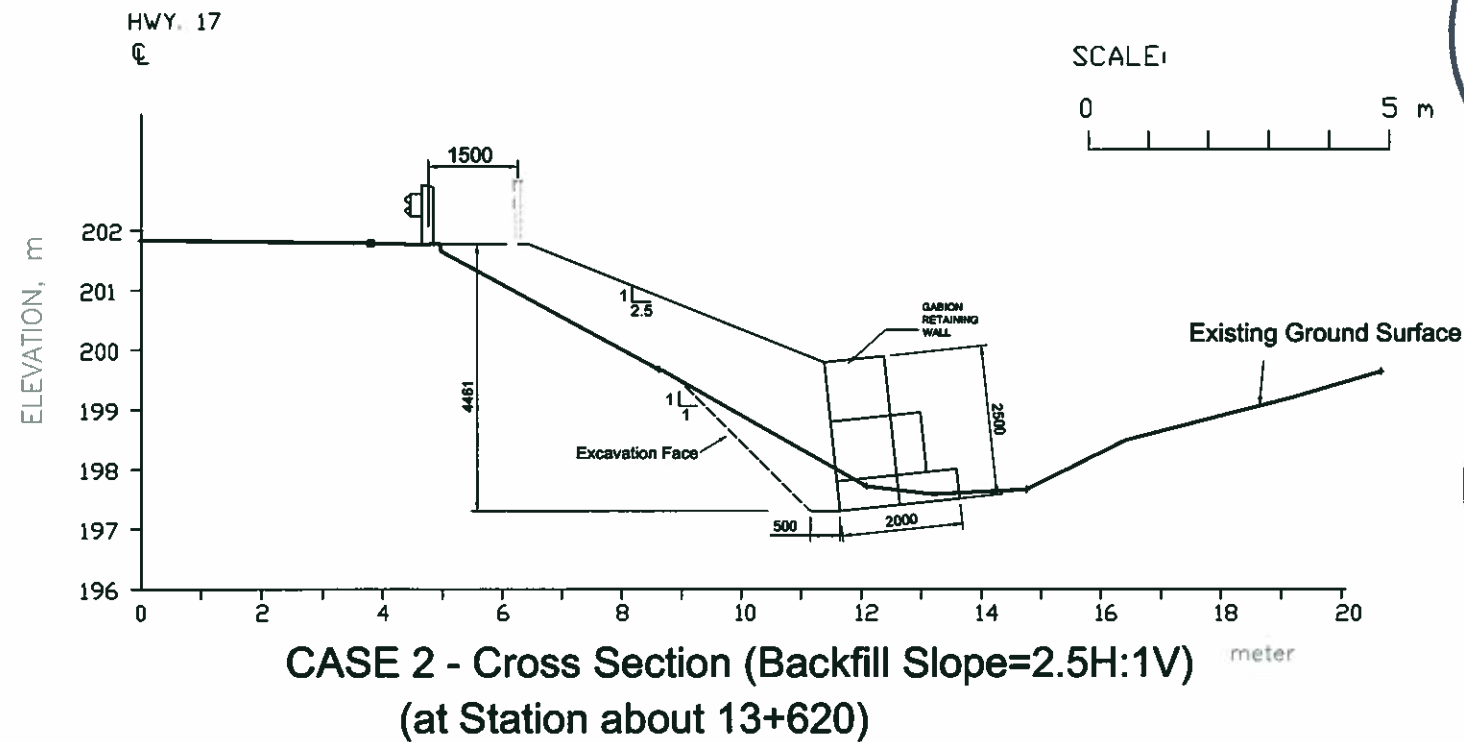
DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

GWP No. 5274-08-00



SHEET

5



KEY MAP
Not to Scale

LEGEND

- BOREHOLE
- Water Level (Piezometer)
- Water Level (Open hole)

No.	ELEVATION	STATION	OFFSET
BH-1	202.183	13+659.9	4.9
BH-2	201.872	13+565.0	5.3
BH-3	198.224	13+577.7	13.8
BH-4	201.799	13+585.3	2.2
BH-5	196.842	13+590.0	15.2
BH-6	201.824	13+625.9	2.2
BH-7	199.429	13+625.9	11.7

NOTES

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- The elevation of the water level in the creek was measured by TROW on 11/Sept./2009

REVISIONS

DATE	BY	DESCRIPTION

 **TROW Associates Inc.**
56 QUEEN STREET EAST, SUIT 301
BRAMPTON, ONTARIO, L6V 4M8
(905) 796-3200

PROJECT TITLE AND LOCATION:
**Gabion Wall Construction
near Laronde Creek
Hwy 17, Sudbury**

DRAWING TITLE:
**CROSS-SECTION
CASE 2
(BACKFILL SLOPE = 2.5)**

PROJECT NO. 5274-08-00	DWN.: GQ
SCALE: AS NOTED	CHKD.: SM
DATE: Sept. 2009	DWG. No.: 5

APPENDIX C : BOREHOLE LOGS

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 475J 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

JOINT 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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	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_a	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
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ'	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No BH-1

1 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.3 CHECKED BY IM

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						
202.2							20 40 60 80 100								
202.0	ASPHALT, (~ 50 mm)		1	AS										64 30 (6)	
201.0	SAND (FILL) (SW), brown, damp, well graded, compact, fine to coarse grained, some gravel, trace to some silt.		2	SS	25										
1.2	Clayey SILT (ML), grey, damp to wet, compact, poorly graded, some fine grained sand.		3	SS	10										
199.9	SILTY CLAY (CL), grey, saturated, low plasticity, soft to stiff.		4	SS	0										
2.3			5	TW											
			6	SS	0										
			7	SS	0										
			8	SS	0										
			9	TW											
			10	SS	0										
			11	SS	0										
			12	SS	0										
			13	SS	0										
			14	SS	2										
			15	SS	0										
			16	SS	0										
			17	SS	0										
								</							

Continued Next Page

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

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT_GDT 09/10/21

METRIC

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

RECORD OF BOREHOLE No BH-2

1 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.4 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE						● QUICK TRIAXIAL	× LAB VANE
201.9																
200.8	ASPHALT, (~ 50 mm)		1	AS												
	SAND (FILL) (SW), brown, damp, loose, well graded, fine to coarse grained, some fine to coarse gravel, trace silt. - some silt below 0.8 m		2	SS	6											
200.4																
1.5	CLAYEY SILT (ML), grey, wet, very loose to loose, trace sand, trace to some clay. very loose below ~ 2.29 m depth.		3	SS	5											
			4	SS	2											
198.8			5	SS	0											
3.1	SILTY CLAY (CI), grey, saturated, medium plasticity, firm to stiff.		6	TW												
			7	SS	0											
			8	SS	0											
			9	SS	0											
			10	TW												
			11	SS	0											
			12	SS	0											
			13	SS	0											
			14	TW												
			15	SS	0											
			16	SS	0											
			17	SS	0											
					</											

Continued Next Page

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

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

RECORD OF BOREHOLE No BH-2

2 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.4 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×						LAB VANE		
			18	SS	2															
180.5							181													
21.3	HARD AUGERING, suspected sand and gravel till.		19	BAG			180													
179.6																				
22.3	BOREHOLE TERMINATED AT ~ 22.25 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK																			
	NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed monitoring well to 12.2 m depth.																			



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

RECORD OF BOREHOLE No BH-3

1 OF 1

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Auger (Wash Boring) COMPILED BY KR
 DATUM Geodetic DATE 09.9.8 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L					
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)						
198.2							20 40 60 80 100	20 40 60 80 100	10 20 30						
0.0	TOPSOIL (~76mm) over		1	SS	13		198								
197.4	SAND (FILL) (SW) , brown, damp,		2	SS	2		197								
0.8	compact, poorly graded, fine to														
	coarse grained, some silt, trace to														
	SANDY SILT (SM) , grey, wet, very		3	SS	20										
196.6	loose, some gravel.														
1.6	SILTY CLAY (CI-MI) , brown,		4	SS	2										
	saturated, medium plasticity, soft to														
	stiff.		5	SS	2										
	grey below ~ 3.05 m depth.														
			6	TW											
			7	SS	1										
			8	SS	1										
		9	SS	2											
		10	SS	3											
		11	SS	9											
		12	SS	4											
		13	SS	6											
		14	SS	7											
		15	SS	100											
179.6															
18.7	BOREHOLE TERMINATED AT ~ 18.67 m DEPTH DUE TO SPT REFUSAL ON SUSPECTED BEDROCK														

(Gs=2.733)
0 0 35 65

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

ON MOT SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON MOT.GDT 09/10/21

METRIC

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

ON MOT SO11878G - LARONDE CREEK BRIDGE BY GREG 6.GPJ ON MOT.GDT 09/10/21

RECORD OF BOREHOLE No BH-4

2 OF 2

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.9 CHECKED BY IM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			
	2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed PVC standpipes to 12.2 m depth.																

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

RECORD OF BOREHOLE No BH-5

1 OF 1

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY GQ
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Auger (Wash Boring) COMPILED BY GQ
 DATUM Geodetic DATE 09.9.9 CHECKED BY VD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
196.8	0.0	TOPSOIL, (~ 15 mm) over SAND (FILL) (SW) , some silt, trace rootlets and wood deris. brown, damp to wet, very loose to compact, fine grained. - a thin (0.15 m) layer of silty clay at a depth of about 0.9 m - become wet below 1.05 m		1	SS	2												
				2	SS	12												
				3	SS	2												
194.4	2.5	SILTY CLAY (CL) , varved, grey, saturated, soft to stiff, low plasticity		4	SS	1												
				5	SS	2												
				6	SS	0												
				7	TW													
				8	SS	0												
				9	SS	0												
				10	SS	2												
				11	SS	1												
				12	SS	2												
181.5	15.4	BOREHOLE TERMINATED AT ~ 15.4 m DEPTH																
		NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed monitoring well to 11.2 m depth.																

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

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

RECORD OF BOREHOLE No BH-6

1 OF 1

METRIC

W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY CS
 DIST 54 HWY 17 BOREHOLE TYPE CME 200mm OI Hollow Stem Auger COMPILED BY KR
 DATUM Geodetic DATE 09.9.10 CHECKED BY IM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE								
						● QUICK TRIAXIAL	×	LAB VANE										
201.8																		
200.9	ASPHALT, (~ 300 mm)																	
0.3	SAND (FILL) (SW), brown, damp, fine to coarse grained, trace fine grained gravel, some silt.		1	AS			201											
200.9			2	SS	11		200											
0.9	SANDY SILT(SM), brown, wet, very loose to compact, trace fine to coarse grained gravel.		3	SS	9		199											
			4	SS	3		198											
	very loose below ~ 2.57 m depth.		5	SS	2		197											
	trace clay below ~ 3.05 m depth.		6	SS	2		196											
			7	SS	4		195											
	clayey below ~ 4.57 m depth.		8	SS	3		194											
196.5	SILTY CLAY (CL), grey, saturated, low plasticity, firm to stiff		9	TW			193											
5.3			10	SS	0		192											
			11	SS	0		191											
			12	SS	0		190											
			13	SS	0		189											
			14	SS	0		188											
			15	TW			187											
185.7							186											
16.2	BOREHOLE TERMINATED AT ~ 16.15 m DEPTH																	
NOTES: 1. This drawing is to be read with the subject report and project number as presented above. 2. Interpretation assistance by Trow is required before use by others. 3. Date of W.L.=Sept. 11, 2009. 4. Installed PVC standpipes to 12.2 m depth.																		

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

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT.GDT 09/10/21

RECORD OF BOREHOLE No BH-7

1 OF 1

METRIC

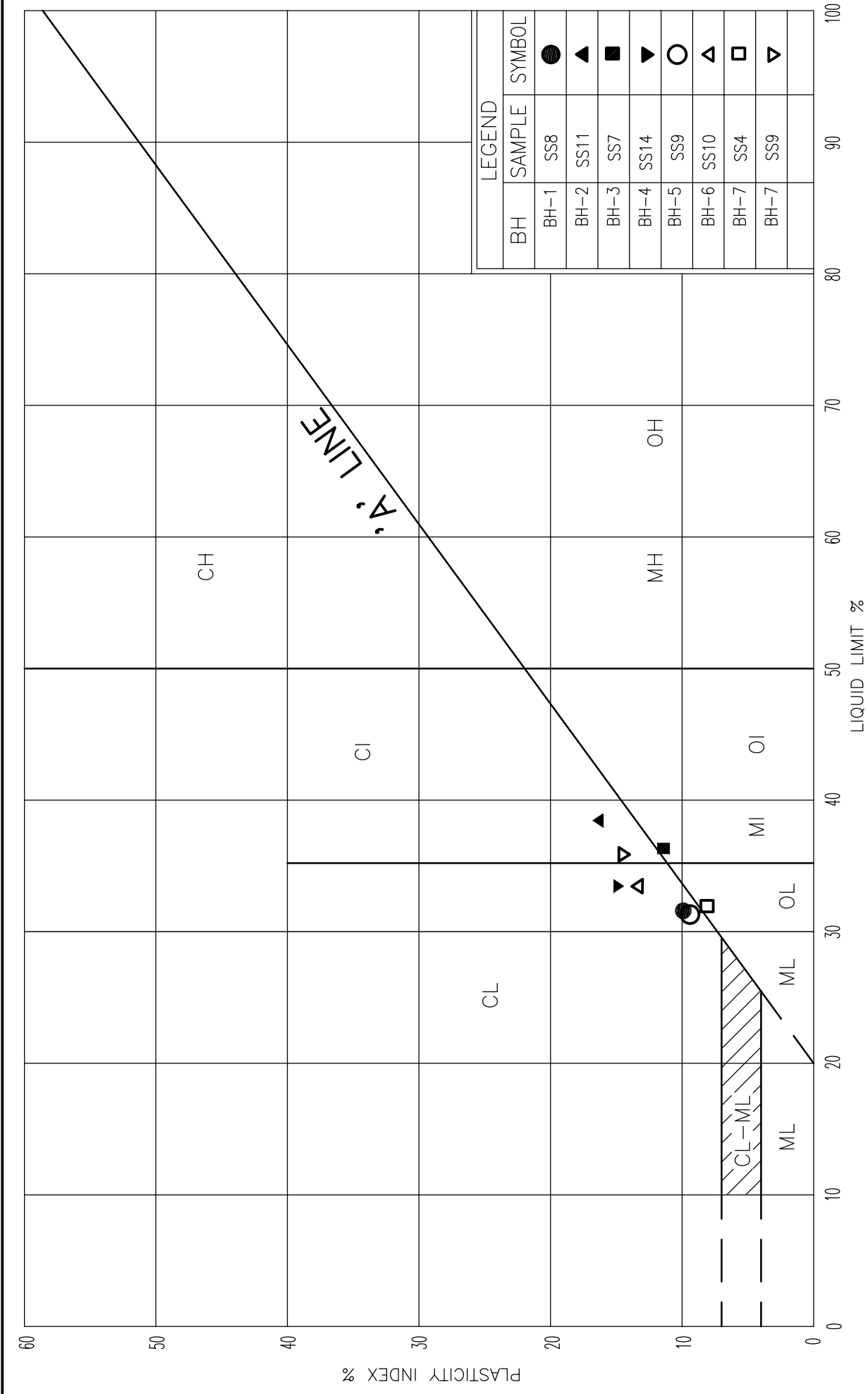
W.P. 5274-08-00 LOCATION Laronde Creek, Nipissing Indian Reserve No. 10 ORIGINATED BY GQ
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Auger (Wash Boring) COMPILED BY GQ
 DATUM Geodetic DATE 09.9.10 CHECKED BY VD



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
198.4 0.0	SAND (FILL) (SW), brown, damp, loose, fine to coarse grained, some silt, trace gravel.		1	AS			198								24 70 (6)			
			2	SS	8		197											
196.8 1.5	SILTY CLAY (ML-CI), grey, saturated, soft to stiff, low to medium plasticity - varved below about 4.57 m		3	SS	10		196									(Gs=2.738) 0 1 44 55		
			4	SS	18		195											
			5	SS	4		194											
							193											
							192											
							191											
							190											
							189											
							188											
							187											
						186												
				185														
				184														
183.0 15.4	BOREHOLE TERMINATED AT ~ 15.4 m DEPTH						183											

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

ON_MOT_SO11878G - LARONDE CREEK BRIDGE BY GREG & GPJ ON_MOT_GDT 09/10/21

Appendix D: LABORATORY DATA



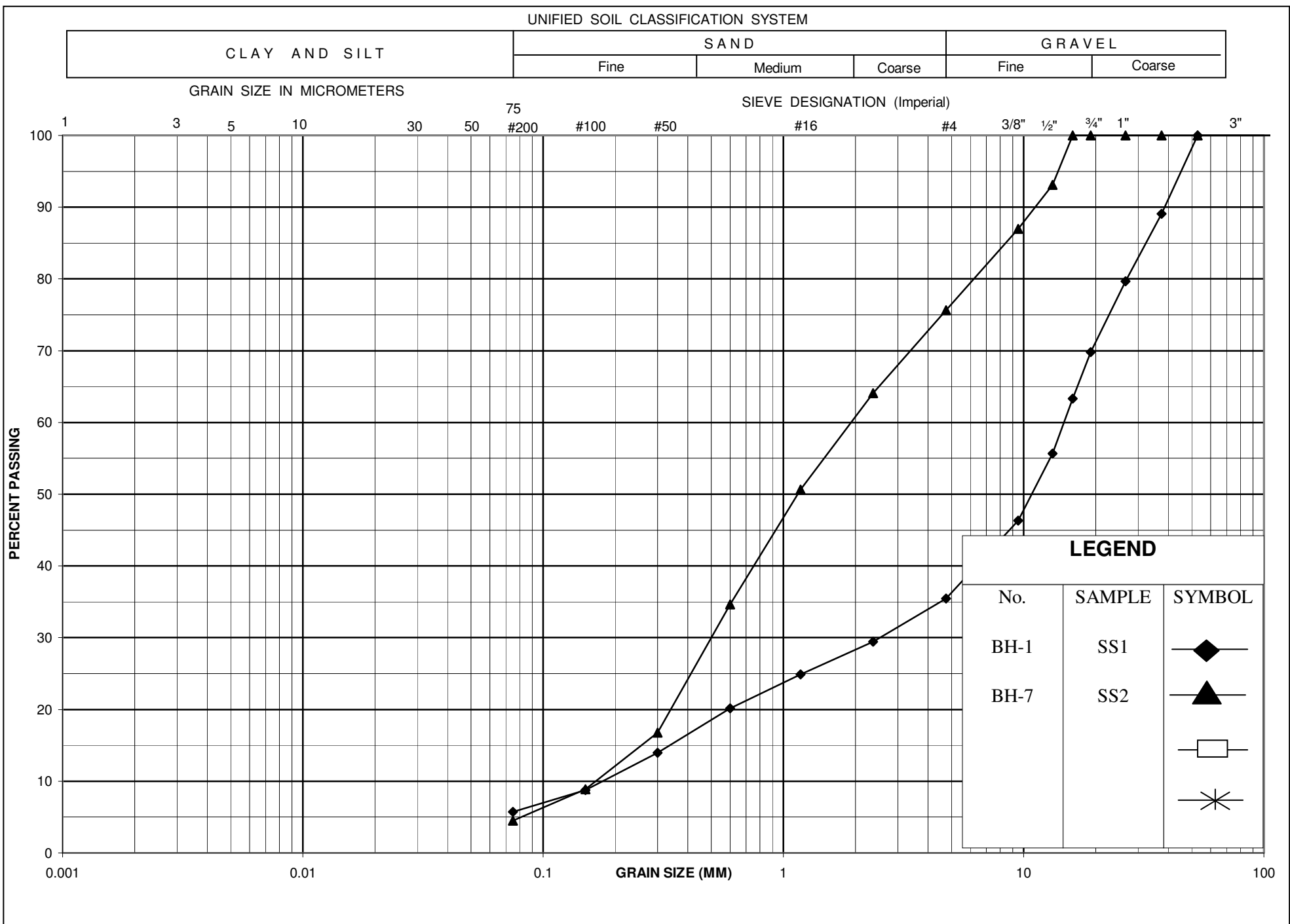


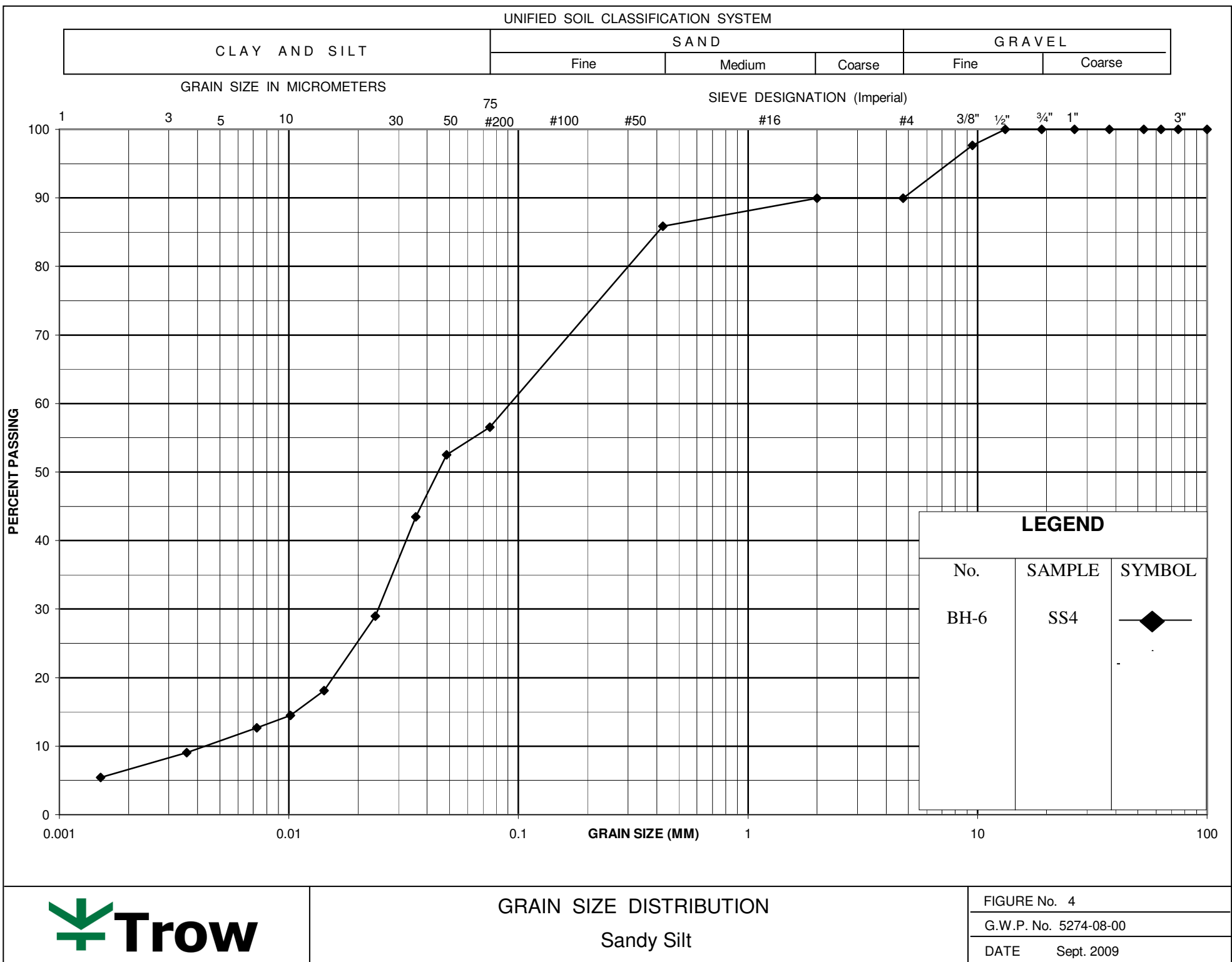
PLASTICITY CHART
SILTY CLAY, (CL,CI,ML,MI)

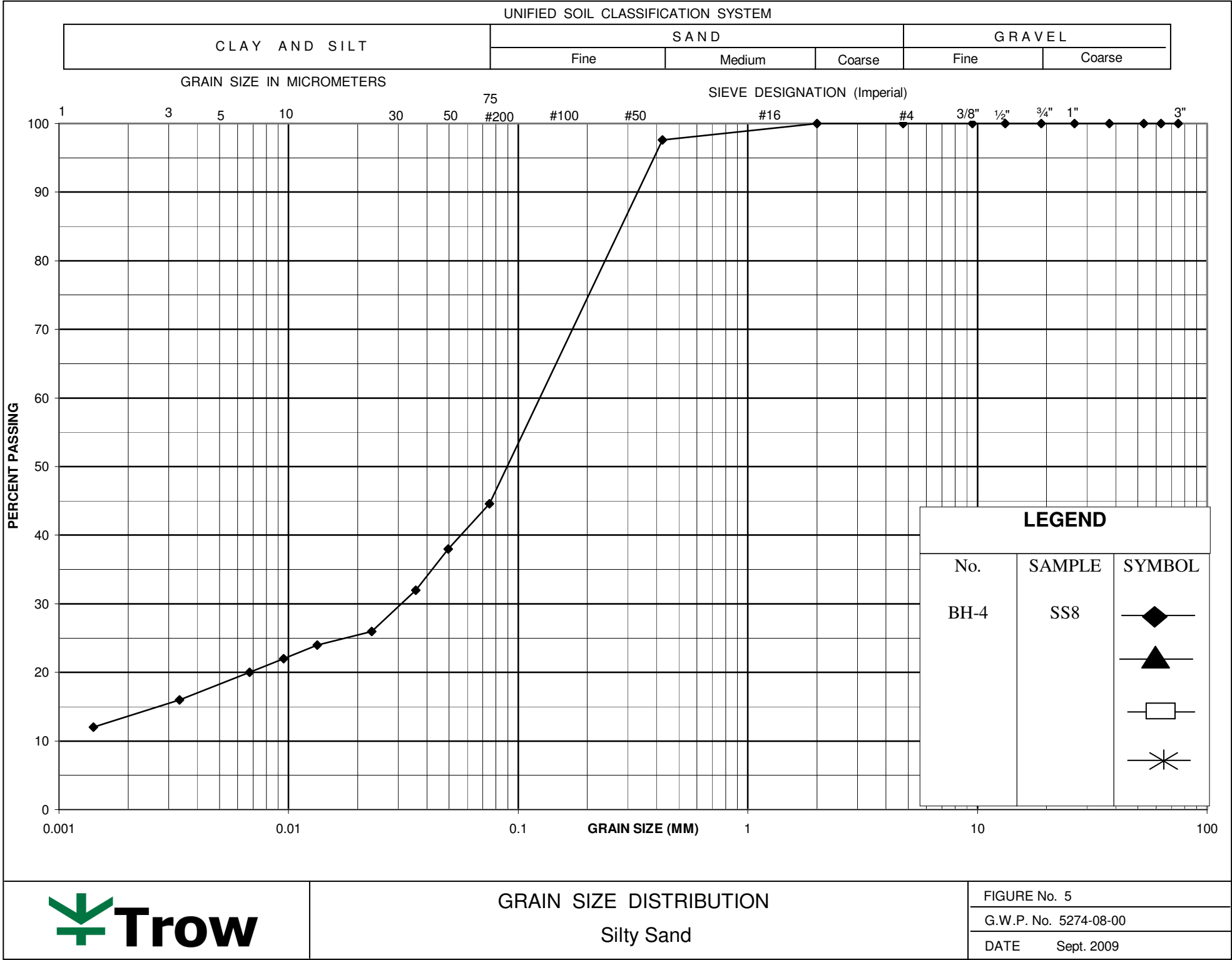
FIGURE No. 1

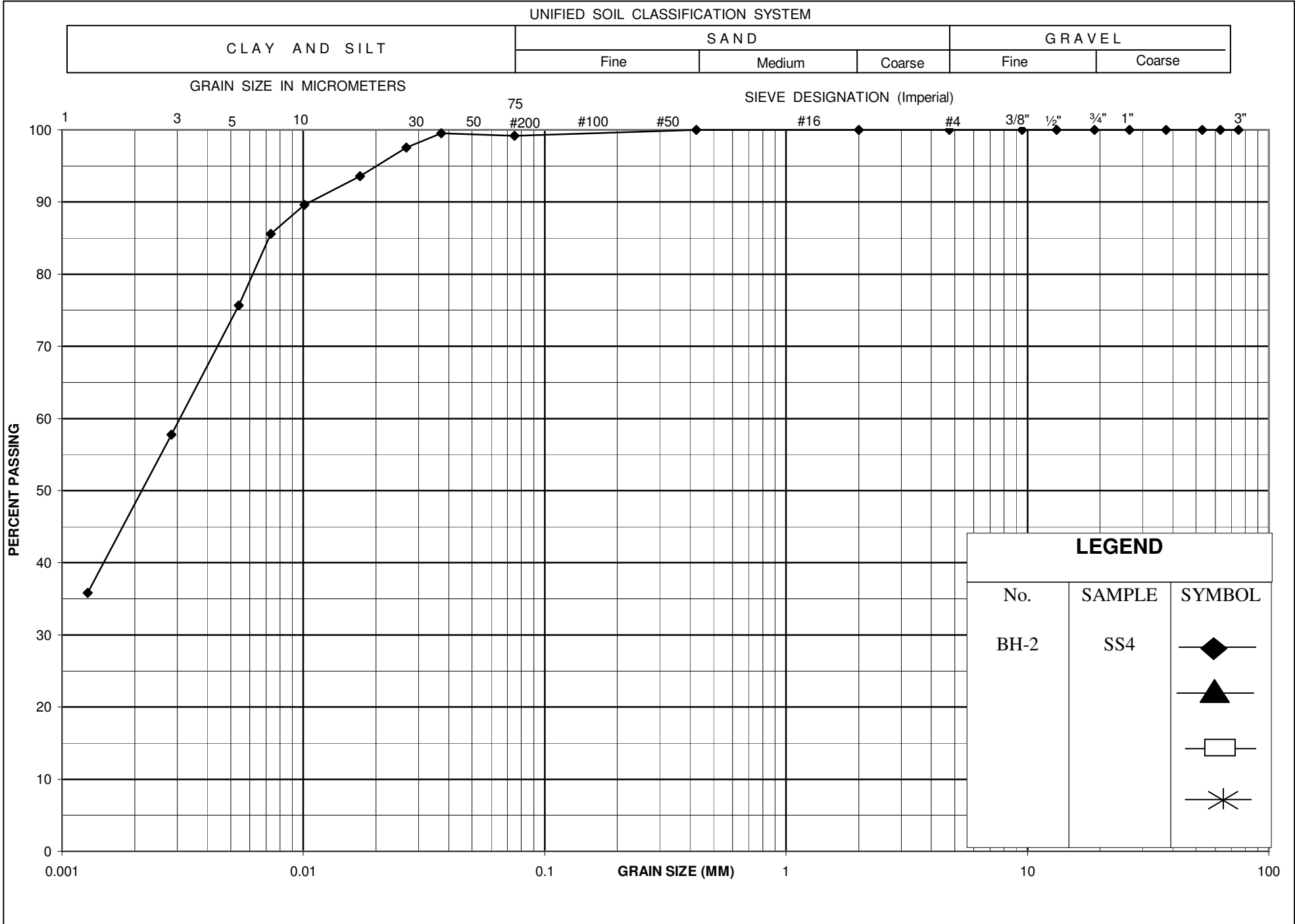
WO: 5274-08-00

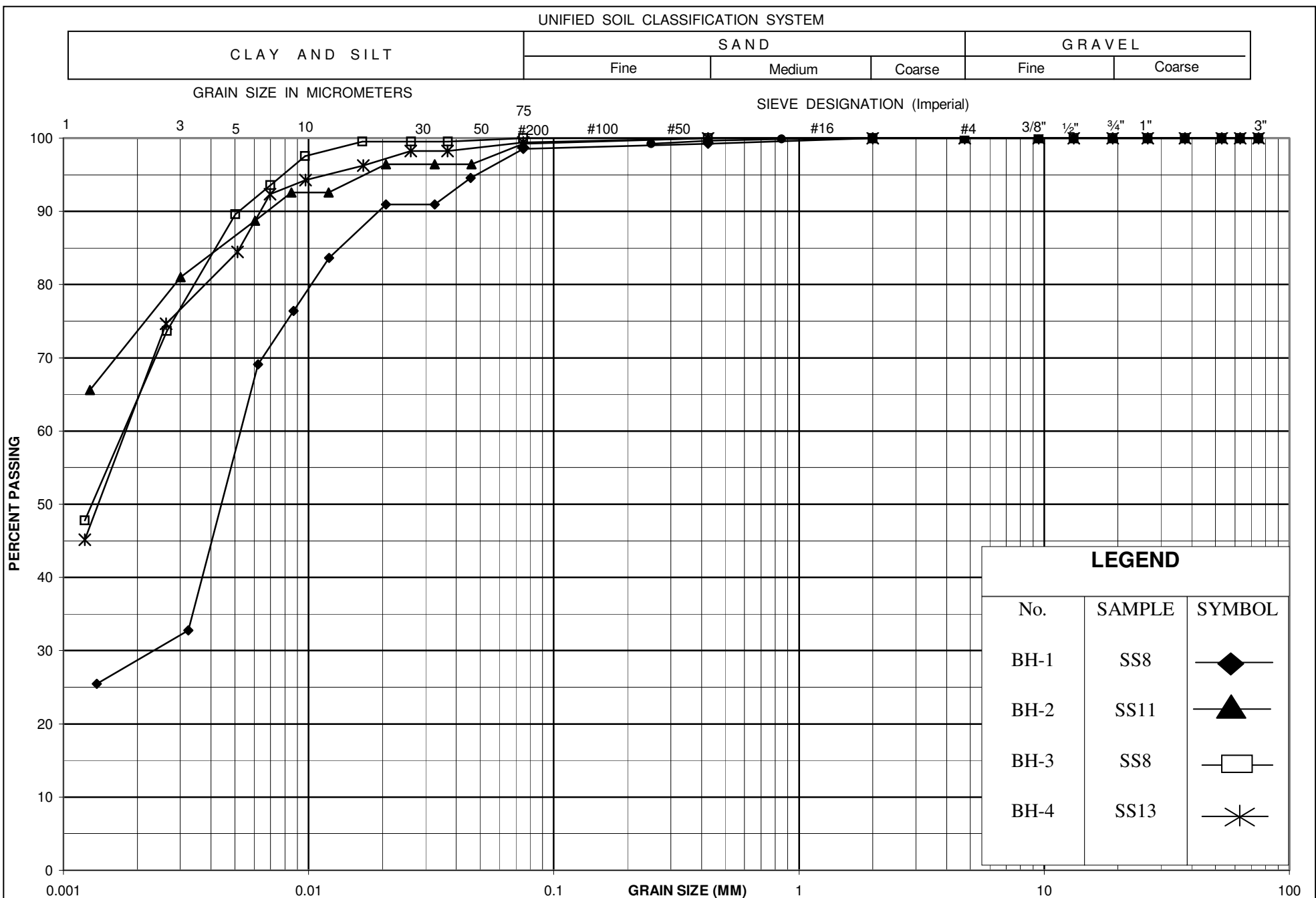
Gabion Wall Construction, Hwy 17 Sudbury

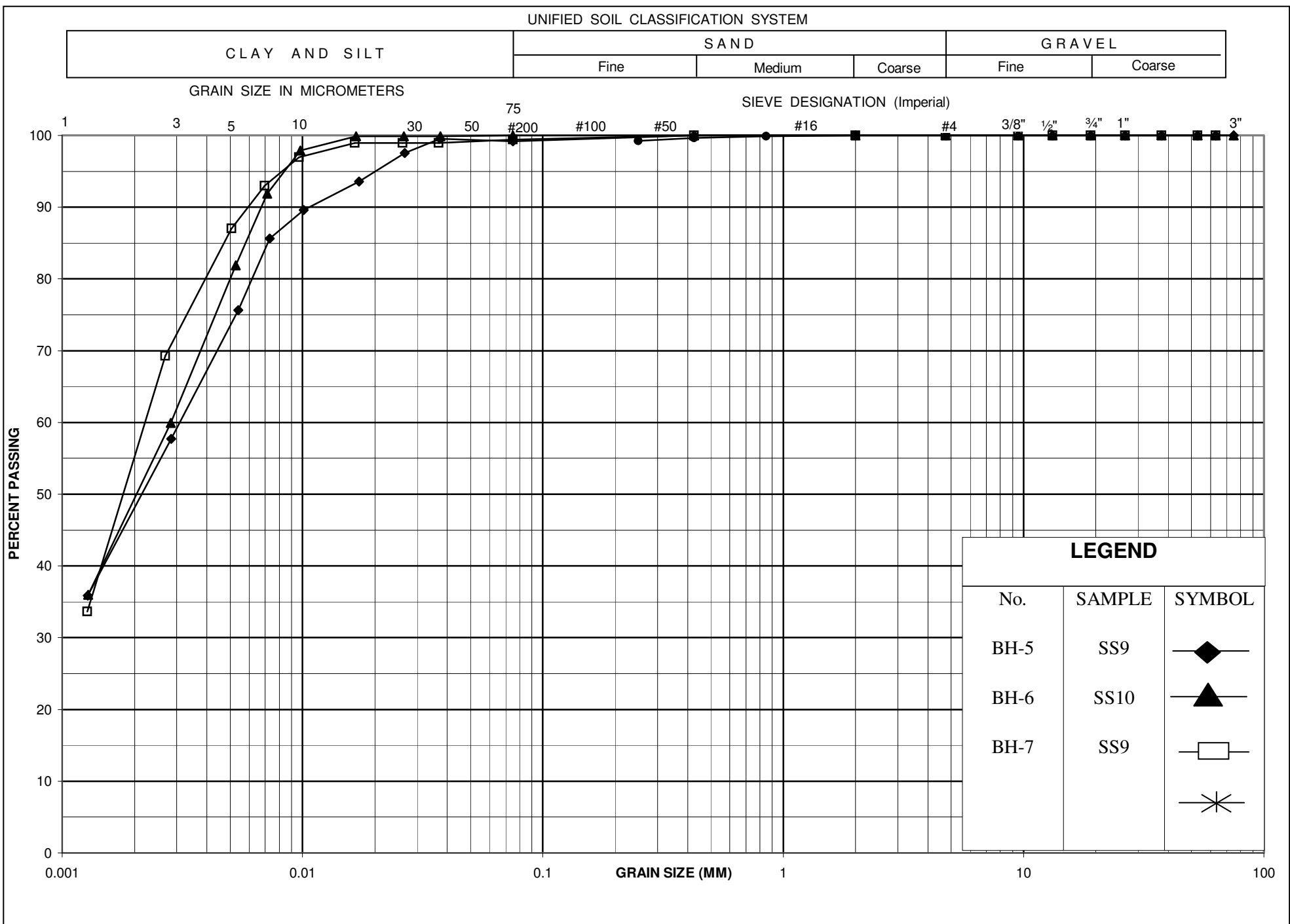












APPENDIX E

GLOBAL STABILITY ANALYSES

Figure 1 Stability Analysis for Case 1
(refer to Drawing 4 in Appendix B for the geometry)

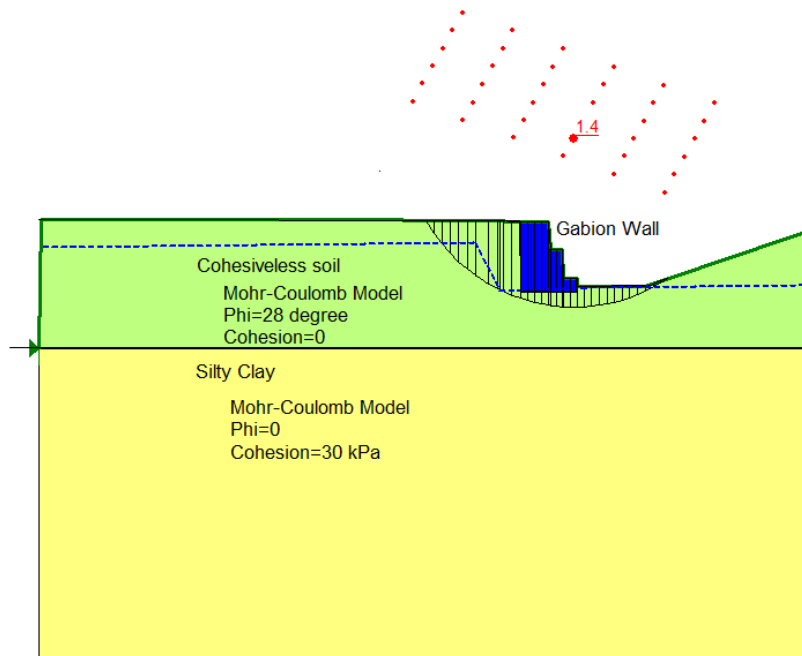


Figure 2 Stability Analysis for Case 2
(Backfill slope = 2H:1V, refer to Drawing 4 in Appendix B for the geometry)

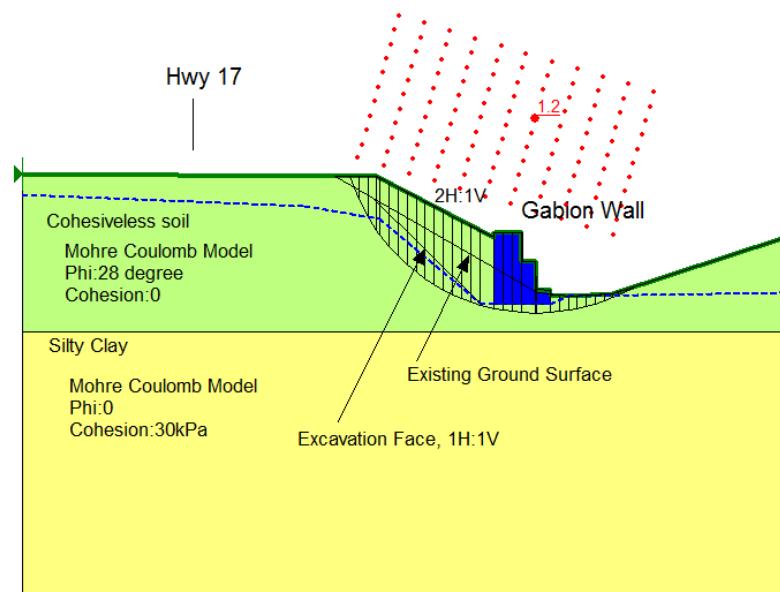


Figure 3 Stability Analysis for Case 2 Assuming a Potential Failure Surface Extended into the Silty Clay.
(Backfill slope = **2H:1V**, refer to Drawing 4 in Appendix B for the geometry)

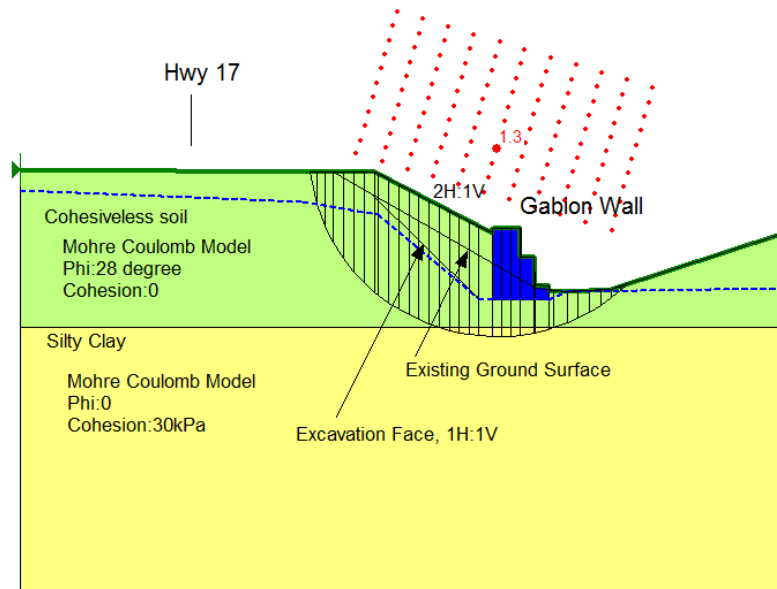


Figure 4 Stability Analysis for Case 2 Assuming the Wall Base Founded on the Silty Clay.
(Backfill slope = **2H:1V**, refer to Drawing 4 in Appendix B for the geometry)

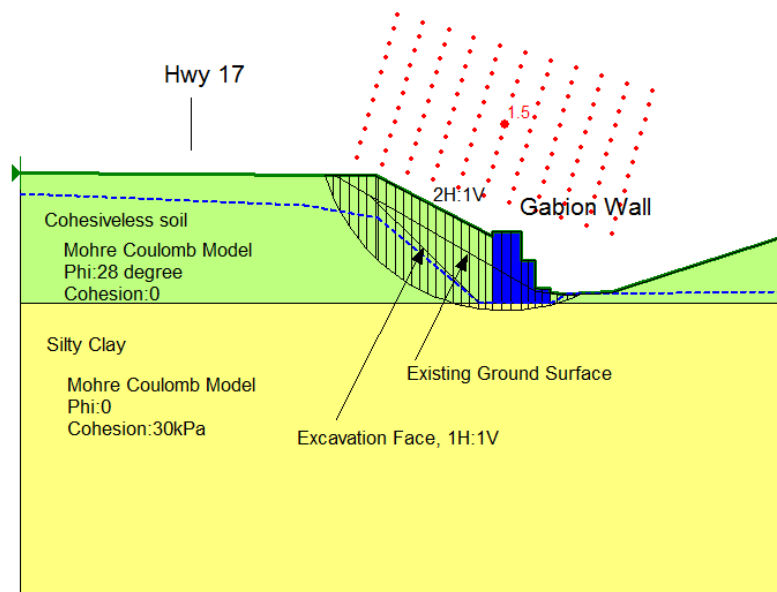


Figure 5 Stability Analysis for Case 2
(Backfill slope = **2.5H:1V**, refer to Drawing 5 in Appendix B for the geometry)

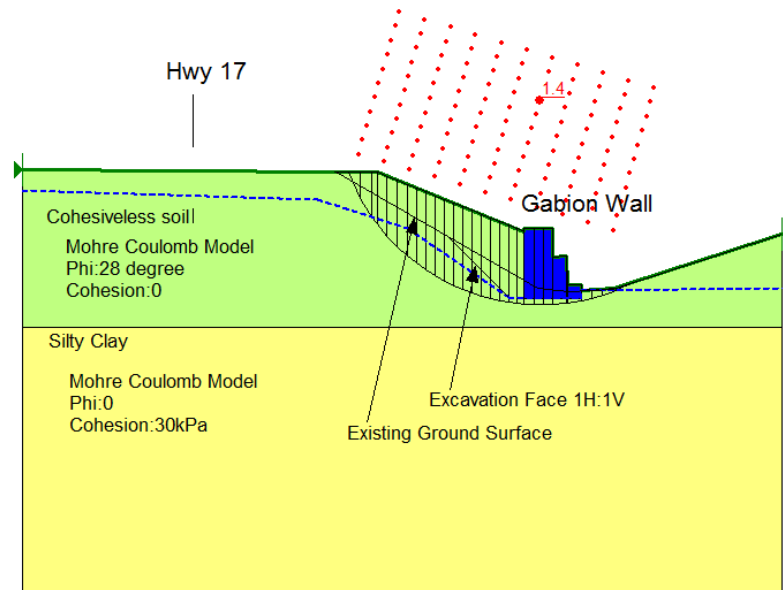
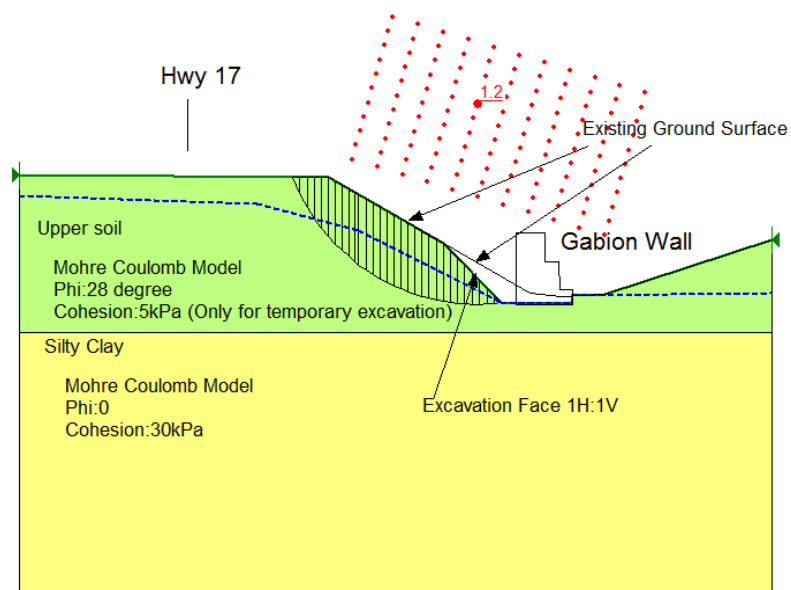


Figure 6 Temporary Excavation Stability Analysis for Case 2
(Backfill slope = **2.5H:1V**, refer to Drawing 5 in Appendix B for the geometry)



APPENDIX F
CROSS-SECTIONS PROVIDED BY MTO

