PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

CROSSING ROAD INTERCHANGE UNDERPASS
HIGHWAY 11
TOWN OF HUNTSVILLE
TOWNSHIP OF STEPHENSON
GWP 320-00-00
DISTRICT 52, HUNTSVILLE

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

3 cc: Stantec Consulting Ltd. for distribution to MTO, Project Manager + one digital copy
1 cc: Stantec Consulting Ltd. for distribution to MTO, Pavements and Foundations Section + one digital copy
1 cc: Stantec Consulting Ltd. + one digital copy
1 cc: PML Toronto

PML Ref.: 08TF008
Index No.: 025FIDR
GEOCRES No.: 31E-295
June 17, 2009

TABLE OF CONTENTS

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Crossing Road Interchange Underpass
Highway 11
Town of Huntsville
Township of Stephenson
GWP 320-00-00
District 52, Huntsville

---

1. INTRODUCTION

This report provides the factual data from the preliminary foundation investigation carried out at a proposed crossing road interchange underpass in the Town of Huntsville. The underpass is part of the proposed upgrading to a fully access controlled freeway standard of the existing four-lane Highway 11 section that extends from 1.0 km north of Highway 141 northerly for 5.5 km within the Geographic Township of Stephenson and the Town of Huntsville. Peto MacCallum Ltd. (PML) carried out the study for the Ministry of Transportation of Ontario (MTO) on behalf of Stantec Consulting Ltd. (Stantec).

The preliminary foundation investigation was conducted for a proposed site located about 1.1 km north of the Stephenson Road 8 Intersection

2. SITE DESCRIPTION

The study area of the proposed crossing road underpass is generally located about 11 km south of Huntsville within the Geographic Township of Stephenson and the Town of Huntsville. Representative site photographs are included in Appendix A.

At the site location, a bedrock outcrop about 8 to 9 m high is visible to the east of Highway 11, east of right-of-way (R.O.W), dipping down east to west at an angle about 15 to 30^{o}. A wet and swampy area covers the west side of the Highway 11. The Lancelot (Bullen) Creek crosses the Highway 11 at approximate Sta. 20+760, 560 m north of the site location.

Land uses in the vicinity of the underpass include industrial/commercial activity from Muskoka Concrete and Ontario Stone facilities and a number of residences along the local side roads.

3. SITE PHYSIOGRAPHY AND GEOLOGY

This site is located within the physiographic region known as the Number 11 Strip. This area includes a narrow strip of land that follows Highway 11 from Gravenhurst to North Bay. The local topography is undulating as the highway traverses areas which alternate between steep rock ridges and low lying, swampy areas. The native overburden soils consist mainly of fine sands and silts, classified as Berriedale fine sand and Magnetawan silt.

The depth of soil cover in the local swamp areas is variable and may extend to 30 m or greater.

Generally, surface water run off in the area of the proposed underpass drains into the Lancelot (Bullen) Creek and its tributaries. Groundwater is inferred typically near the ground surface (1 to 5 m deep) west of Highway 11 at the proposed structure site.

The study area is located within the Central Gneiss Belt. The bedrock in this area includes Precambrian rock of Mesoproterozoic age. The predominant bedrock types in the area are migmatites, gneisses (biotite and quartzofeldspathic gneisses) and felsic igneous rocks (granodiorites and granites). The local bedrock along this section of highway undulated from the ground surface to over 30 m below the ground surface; scattered bedrock outcroppings are present along the highway north and south of the underpass site.

4. FIELD INVESTIGATION

The field investigation for the structure was carried out during the period from April 7 to 9 and 13, 2009. A total of one probe hole and three boreholes were put down at the underpass location as shown on the attached Drawing AR-1. One dynamic core penetration test (cone test) was advanced from the end of one of the boreholes. The probe hole, boreholes and core test extended to depths ranging from 1.1 to 38.1 m below ground surface, elevations 249.6 to 291.0 m (a difference of 41.4 m).

The originally proposed location of borehole 4 was moved to a swampy area about 80 m south of the proposed underpass alignment and within the right-of-way due to lack of permission to enter at the time of the investigation.

Probe hole 1 was advanced manually within 5 m of a massive rock outcrop due to access constraints to a drilling. The remaining boreholes were advanced using continuous hollow stem augers, in conjunction with dynamic cone penetration test (DCPT) (borehole 2) and wash boring (borehole 3), powered by a track mounted D-50 turbo rig, supplied and operated by a specialist drilling contractor. The drilling crews worked under the full-time supervision of a member of our engineering staff. The supervisor was in frequent contact with the project engineer to provide status reports and obtain direction when adjustments to the field program warranted.

Representative samples of the soils encountered in the boreholes were recovered at 0.75, 1.5 and 3.0 m depth intervals. In the boreholes advanced with conventional drill rigs, soil samples were obtained using a split spoon sampler in conjunction with standard penetration tests. Field vane testing and penetrometer tests were carried out to estimate the consistency of the encountered soils. The penetrometer tests results typically provide lower shear strength values than the actual values due to sample disturbance. Where standard penetration tests were not carried out, the consistency/relatively density of the encountered soils was estimated from manual examination or the rate (ease) of advances of the augers.

The boreholes were backfilled in accordance with the MTO guidelines and MOE regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes.

EJWilliams Surveying Limited staked and surveyed out the locations of the probe hole 1 and boreholes 2 and 3. Borehole 4 was staked and surveyed by PML staff. The horizontal northing and easting co-ordinates and the geodetic ground surface elevations at the boreholes are shown on the Record of Boreholes and are listed on the Borehole Location Plan, Drawing AR-1.

Soils were identified in the field in accordance with the MTO Soil Classification procedures.

Recovered soil samples were returned to PML Toronto laboratory for detailed visual examination, soil classification and laboratory testing. The current laboratory test program comprised the following tests:

- Natural moisture content determinations (51)
- Grain size analyses (11)
- Atterberg limits tests (5)
- Unconfined compression test (1)

The results of the laboratory testing are shown on the Record of Borehole sheets, on the attached grain size distribution charts AR-GS-1 to AR-GS-3 and plasticity charts AR-PC-1 and AR-PC-2.

5. SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets for moisture content determinations, soil classification, inferred stratigraphy, standard penetration test N values, penetrometer test values and field vane test values, together with groundwater observations in the open boreholes. The boundaries between soil strata have been established at the boreholes locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The results of laboratory natural water content determinations are also shown on the Record of Borehole sheets. The locations of the boreholes drilled at the site are shown on Drawing AR-1.

The summarized subsurface conditions revealed in the probe hole and boreholes generally comprised surficial topsoil/peat/fill overlying an upper deposit of silt unit underlain by cohesive clayey soils, which in turn underlain by a lower deposit of cohesionless silt matrix. Probable bedrock surface was encountered below the silt unit in the probe hole 1 and borehole 3.

5.1 Fill

A 600 mm thick layer of surficial sandy silt fill was encountered in borehole 3, extending from elevations 287.7 to 287.1 m. One N value of 2 was obtained. The moisture content determination of the obtained fill sample was 21%.

5.2 Topsoil / Peat

A 300 and 500 mm thick surficial topsoil unit was encountered in boreholes 1 and 2, respectively, extending to elevations 291.8 and 289.4 m. Further, a 500 mm thick deposit of topsoil was encountered in borehole 3 below the fill unit extending to 1.1 m, elevation 287.1 m. One N value of 3 was obtained in the topsoil. Two moisture content determinations were 18 and 64%.

A 500 mm thick surficial peat was encountered in borehole 4 and extended to elevation 286.6 m.

5.3 Silt

A continuous cohesionless silt deposit was encountered at 0.3 to 1.1 m, elevations 286.6 to 291.8 m, below topsoil in boreholes 1 to 3 and below peat in borehole 4, and extended to 1.1 to 10.2 m depths below ground surface, elevations 279.7 to 291.0 m. Lower zone of the silt unit were encountered at 8.8 to 13.1 m, elevations 275.8 to 278.3 m, in boreholes 2 to 4 and extended to 14.3 to 38.1 m, elevations 249.6 to 272.8 m. The relative density of the silt was typically compact with local loose and dense zones. N values typically ranged from 10 to 29 blows, with local N values of 4 to 8 in the loose zones and one N value of 34 in the dense zone found in borehole 3.

A cone test was carried out in borehole 2 from 31.1 to 36.0 m depths, elevations 258.8 to 253.9 m. The cone test obtained values ranged from 14 to 141, increasing with depth.

A grain size distribution chart envelope of selected silt samples is presented in Figure AR-GS-1. Moisture contents typically ranged from 16 to 22%, with a local high value of 30%.

5.4 Clayey Silt

Discontinuous cohesive 3.0 and 3.2 m thick deposits of clayey silt were encountered at 5.5 and 5.6 m depths, elevations 282.2 and 281.5 m, in boreholes 3 and 4, respectively, below the silt layer and extended to 8.5 and 8.8 m depths, elevations 279.2 and 278.3 m. A 1.4 m thick lower deposit of clayey silt was encountered at 10.5 m, elevation 277.2 m, below silty clay in borehole 3 and extended to 11.9 m depth, elevation 275.8 m. The consistency of the clayey silt unit is stiff with a local firm zone (borehole 3). One penetrometer test value was 50 kPa. Penetrometer tests provide typically lower than actual shear strength values due to sample disturbance. Field vane test results ranged from 72 to 84 kPa with sensitivity values of 5 and 6. One unconfined compression test obtained 29 kPa shear strength (9% strain at failure) in borehole 3. Three obtained N values were 2, 3 and 4.

A grain size distribution chart for selected clayey silt samples is presented in Figure AR-GS-2. The plasticity chart of the clayey silt samples is presented in Figure AR-PC-1. The Atterberg liquid and plastic limits obtained ranged from 30 to 34 and 18 to 24, respectively, with plasticity index values ranging from 10 to 12. Moisture contents ranged from 23 to 50%.

5.5 Silty Clay

Local deposits of 2.9 and 2.0 m thick cohesive silty clay were encountered below silt in borehole 2 at 10.2 m, elevation 279.7 m, and at 8.5 m depth, elevation 279.2 m, below clayey silt in borehole 3, extending to 13.1 and 10.5 m depths, respectively, elevations 276.8 and 277.2 m. The consistency of the silty clay is stiff. Field vane test results ranged from 72 to 92 kPa with sensitivity values of 4 and 5. Three N values were 3, 4 and 7.

A grain size distribution chart for selected silty clay samples is presented in Figure AR-GS-3. The plasticity chart of the silty clay samples is presented in Figure AR-PC-2. The Atterberg liquid and plastic limits obtained are 49 and 50, and 25 and 24, respectively, with plasticity index of 24 and 26. Moisture contents ranged from about 38 to 49%.

5.6 Bedrock

Probable bedrock was encountered in probe hole 1 at 1.1 m, elevation 291.0 m and in borehole 3 below silt at 38.1 m depth, elevation 249.6 m. The table below summarizes the depth to bedrock / probable bedrock at the borehole locations:

BOREHOLE /PROBE HOLE NO.	DEPTH TO PROBABLE BEDROCK (m)	PROBABLE BEDROCK ELEVATION (m)
1	1.1	291.0
2	>36.0	<253.9
3	38.1	249.6
4	>14.3	<272.8

The bedrock surface was not encountered in boreholes 2 and 4.

5.7 Groundwater

Groundwater was observed at 0.3 to 1.4 m depths, elevations 286.5 to 288.5 m, in boreholes 2 to 4 during augering. Upon completion of augering the water level was observed at surface in boreholes 3 and 4, elevations 287.7 and 287.1 m, respectively.

Groundwater was not encountered in probe hole 1 during and after completion of augering.

6. MISCELLANEOUS

The field work was carried out under the supervision of Mr. F. Portela, Field Supervisor, and direction of Mr. M. Narduzzi, BEng. and Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Walker Drilling Inc. supplied the drilling equipment.

.

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
Crossing Road Interchange Underpass
Highway 11
Town of Huntsville
Township of Stephenson
GWP 320-00-00
District 52, Huntsville

---

7. PRELIMINARY ENGINEERING DISCUSSION AND RECOMMENDATIONS

7.1 General

Based on the preliminary drawings, it is inferred that the bridge deck will be at an approximate elevation of 299.0 m. Consequently, the height of the approach embankments will be in the order of 8 m (east approach) to 12 m (west approach) at the proposed underpass site.

A summary of the subsurface conditions revealed in the probe hole and boreholes at the structure location is provided in the attached Table 1.

All footings and/or pile caps subject to frost action should be provided with 2.0 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover. Footings bearing directly on bedrock do not require protection from frost.

Preliminary comments and recommendations for design of the structure foundation and approach embankments are summarized in Table 2 – Advantages, Disadvantages, Relative Costs and Risks/Consequences.

In general, the subsurface conditions are suitable for the design and construction of foundations to support the structure at the underpass site in accordance with standard MTO procedures.

7.2 Spread Footings

Use of conventional spread footings founded on the native silty material or bedrock, or a pad of structural fill is considered to be feasible at the east abutment location in view of the relatively shallow soil cover over the probable bedrock surface revealed in probe hole 1.

The following table summarizes the depth to probable bedrock encountered at the east side of the underpass:

LOCATION	DEPTH TO PROBABLE BEDROCK (m)	PROBABLE BEDROCK ELEVATION (m)	RELEVANT BOREHOLE/ PROBE HOLE
East Abutment	1.1	291.0	1 *

Note: * - Noted bedrock outcrop exposed 5.0 m east of probe hole 1.

The recommended preliminary geotechnical resistances for the footings founded on bedrock or on a structural fill pad are as follows:

FOUNDATION ALTERNATIVE	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kN)	GEOTECHNICAL RESISTANCE AT SLS (kN)
Bedrock	8,000	N/A
Structural Fill	900	350

The construction of the east abutment footing should be straightforward and without groundwater problems. Based on the findings of the investigation, rock excavation may be required to establish the footing subgrade on a level founding surface.

7.3 Deep Foundations

Deep foundation is the recommended method for the centre pier and west abutment locations in view of the subsurface conditions at each foundation location. Steel H-piles driven to bedrock is considered to be the preferred deep foundation design method for the centre pier and west abutment locations because of high water level and low geotechnical resistance of the native soils, which precludes the use of drilled cast-in-place caissons.

To found the east abutment on piles, a trench would need to be excavated/blasted into the existing bedrock. The depth of excavation of a trench into rock to accommodate the use of integral abutments will be dictated by structural design details. The excavation width should be at least 1 m wider than the plan area of the piles or as required to allow for compaction of the trench backfill.

The design and construction of the foundations using driven steel H-piles will be subject to the final road grade at the bridge site.

The estimated reference founding levels for piles driven to refusal on bedrock are provided on the following table:

LOCATION	DEPTH TO PROBABLE BEDROCK* (m)	PILE FOUNDING ELEVATION (m)	RELEVANT BOREHOLE/ PROBE HOLE
East Abutment	1.1	291.0 **	1
Centre Pier	>36.0	<253.9	2
West Abutment	38.1	249.6	3

Note: * - A +1.0 m variation of the average depth to rock should be allowed for preliminary cost estimate preparation purposes.
** - A trench may need to be excavated in the bedrock at the east abutment location to allow for a structurally adequate pile length.

The piles will be driven through native soils containing compressible clayey soils at the centre pier and west abutment locations. The existing grade at the west abutment will be raised about 10.0 m above the existing grade. Consequently, the development of negative skin friction on the piles should be considered to affect the structural axial capacity for the west abutment piles only.

Based on the anticipated sloping bedrock surface for the east abutment, centre pier and west abutment location at the pile tips, the preliminary factored axial resistance at ultimate limit states (ULS) for a steel HP 310x110 pile is 1,800 kN. The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the maximum 38.1 m pile length (borehole 3) or greater than 36.0 m (borehole 2) required, the design is not expected to be governed by settlement since the required load causing that magnitude of deformation of the pile (1,900 kN) is larger than the ULS factored geotechnical resistance.

However, differential settlements between the pier and west abutment in relation to the east abutment due to elastic compression of the piles should be considered in the structural design. For the preliminary design purposes, 15 to 20 mm of differential settlement should be expected.

The capacity of the HP 310x110 piles for the west abutment should be reduced to allow for negative skin friction of 260 kN if the area is not preloaded and/or surcharged.

Cobbles and boulders were not encountered during the investigation, however, their presence could damage piles and/or will be difficult to excavate, if encountered. Consequently, the piles should be provided with driving shoes to minimize this potential concern.

Where the piles will set on or into sloping bedrock as determined during the investigation for detail design the pile tips should be equipped with "Rock Points". SP 903S01 calls for the use of Oslo Point (OPSD 3000.201) or Titus H Bearing Pile Points Rock Injector Model on piles driven to sloping bedrock.

A retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 8 Approach Embankments settlements are accommodated. A high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to SP 599S22.

The founding level of the RSS footing is not defined however it is considered that a level above the existing ground surface would be adequate.

The RSS walls at the abutments could be placed on a 1.0 m thick layer structural fill of Granular A core material placed on compacted fill layer over the native soil (west abutment) and/or bedrock (east abutment). The structural fill supporting the RSS wall fill pad and footing should be compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

The RSS supplier should be responsible for specifying the type of backfill material employed, taking into consideration the engineering properties of the proprietary product, the design life of the structure, the pullout resistance required and drainage requirements.

The supplier of the RSS should also be responsible for the design of the structure (reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance, etc.

8. APPROACH EMBANKMENTS

The existing topsoil and other organic or deleterious soils within the backfill zone of the abutments should be excavated prior to placement of the backfill.

The level of the approach embankments will be typically raised about up to 12.0 m at the west approach and a maximum of 8.0 m at the east approach above the existing surface. In addition, an approximate 4.0 m rock cut is anticipated along the east side to continue the proposed underpass crossing alignment to the east ramps and the proposed East Service Road.

The new embankments may be constructed with earth fill or rock fill. The side slopes of earth fill and rock fill embankments should be inclined no steeper than 2H:1V and 1.25H:1V, respectively. Where the height of the embankment fill is greater than 8 m earth fill or 10 m for rock fill, a 2 m wide mid-height bench will be required.

The construction of the approach embankments may require pre-loading or surcharging of the existing native soils to reduce the long-term settlements induced by the 6.4 m thick cohesive and compressible native clayey soils encountered at the west abutment (borehole 3). Settlements of the underlying cohesionless silts and sands will occur rapidly and likely during the embankment construction.

Slope stability analyses were carried out for the west embankment for short-term and long-term conditions. Based on the soil data and limited laboratory tests conducted on selected samples, the table below summarizes the soil parameters applied to the analyses.

LOCATION	MATERIAL DESCRIPTION	UNIT WEIGHT (kN/m^3)	EFFECTIVE COHESION (kPa)	EFFECTIVE FRICTION ANGLE (Degree)
West Embankment	Rock Fill	18.0	0	42
	Earth Fill	21.0	50	0
			5*	28*
	Silty Clay	18.0	60	0
			5*	28*
	Clayey Silt	18.0	50	0
			5*	28*
	Silt	19.0	0	30 (Upper layer)
				32 (Lower layer)

Notes: * - Refers to soil parameters used for long-term slope stability analyses.

The table below summarized the results of preliminary slope stability analyses carried out under for the short-term and long-term scenarios. The analyses were carried out using the Slope/W software prepared by Geo-Slope International Inc. and the graphs are attached in Appendix B.

LOCATION	APPROXIMATE FILL HEIGHT (m)	FILL TYPE	SHORT-TERM / LONG-TERM DURATION	FACTOR OF SAFETY (FOS)
Adjacent to the West abutment (Borehole 3)	10	Rock Fill (1.25H:1V)	Short-term	1.7
			Long-term	2.2
		Earth Fill (2H:1V)	Short-term	1.6
			Long-term	2.2

The FOS values of 1.7 (rock fill) and 1.6 (earth fill) for short-term duration, and 2.2 for long-term duration are considered adequate for stability considerations.

Settlement, during and following completion of construction, will result from the consolidation of the existing native soils below the embankment fill and “self-weight” consolidation of the embankment fill.

At the east approach, only the “self-weight” consolidation of the embankment rock fill of about 40 mm is expected. Settlement of the embankment fill due to consolidation of the underlying bedrock will be negligible. The settlement at the east abutment is expected to be essentially complete within six to nine months following placement of the new fill.

At the west approach, the estimated magnitude of settlement of new rock fill will be in the order of 20 to 60 mm. Consolidation of the underlying soil is expected to be in the order of 20 to 35 mm. Hence, the total consolidation settlement is estimated up to 95 mm. This estimated settlement of cohesive soils at the west abutment is likely to take up to 18 months to occur to 90% completion.

Construction of the west abutment and approach may require special construction procedures such as preloading/surcharging, advance construction or the use of wick drains to accelerate the rate of consolidation.

9. ADDITIONAL STUDIES

The investigations for this report are considered adequate for preliminary design purposes. The recommendations provided in this report are preliminary only and are based on our interpretation of the factual information obtained from a limited number of boreholes. Detailed foundation investigations will be required at the underpass structure location during the Detail Design phase of the project.

For the Detail Design phase, the following is the recommended list of the boreholes to be carried out at the underpass location:

- One borehole at the east abutment, center pier and west abutment locations should be drilled to bedrock and rock cores of at least 3.0 m length should be obtained.
- At each foundation element, additional boreholes should be drilled to determine the bedrock slope transverse to the bridge alignment. Two boreholes with 3.0 m long cores should be advanced at each foundation element with additional auger probes at the east abutment for this purpose.
- Boreholes should be drilled for approach embankments 20 m away from the abutments.
- Minimum two additional boreholes should be programmed at the west approach location to characterize the soil stratigraphy in the swampy area.
- Three boreholes for the design of the anticipated culvert for the Lancelot (Bullen) Creek tributary.

Where the boreholes in this report were drilled within the final footprint of the foundation elements and approach embankments, the data should be used for the Detail Design Foundations Engineering.

10. CLOSURE

This report was prepared by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer with the assistance of Mr. N. Rahman, BASc, and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Contact.

Yours very truly,

Peto MacCallum Ltd.

REGISTERED PROFESSIONAL ENGINEER
C. M. P. Nascimento
PROVINCE OF ONTARIO

C. M. P. Nascimento, P.Eng.
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
June 17/09
PROVINCE OF ONTARIO

Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

CN/BRG:nr-mi/lnr

TABLE 1

SUMMARY OF SUBSURFACE CONDITIONS

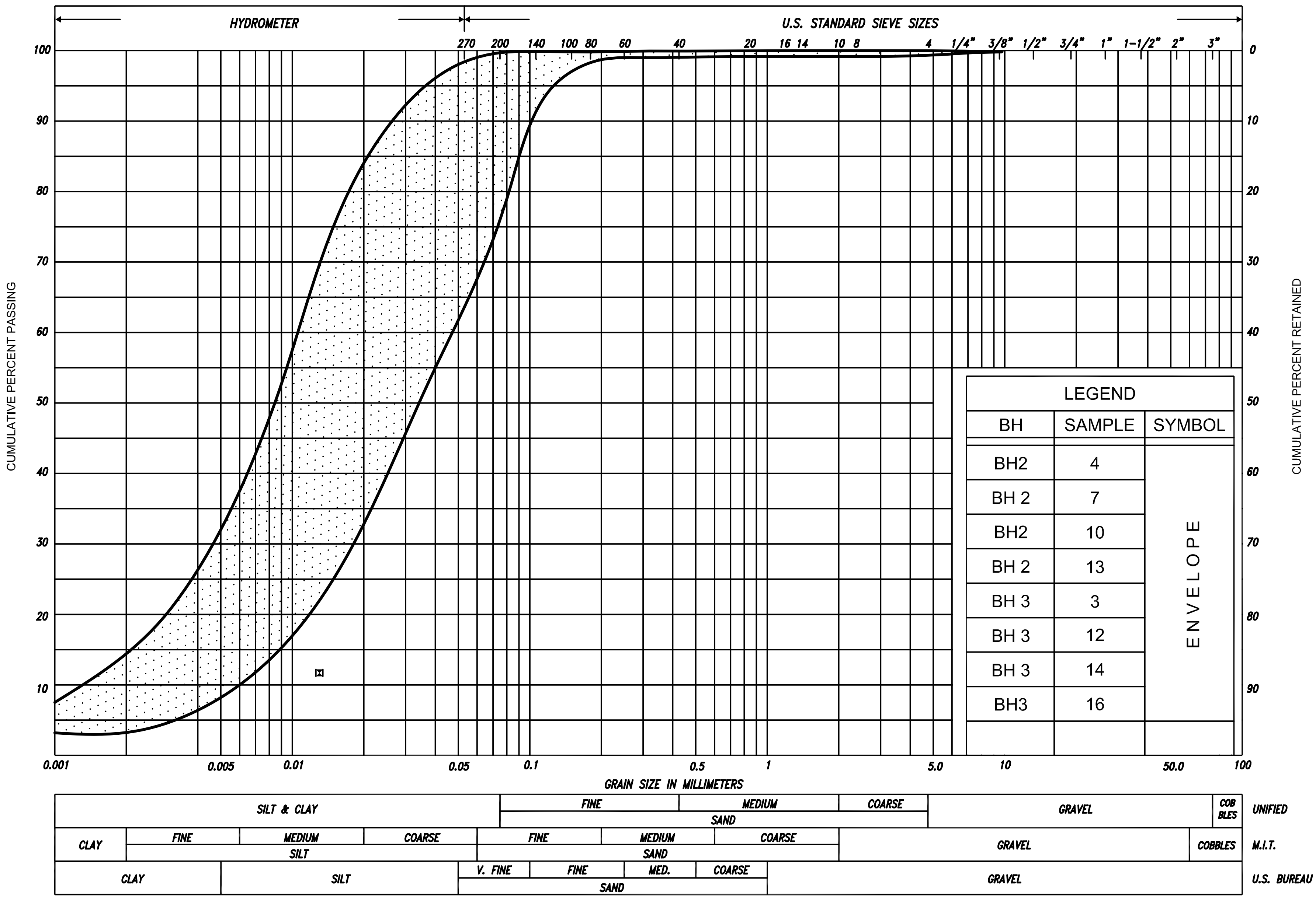
LOCATION AT THE BRIDGE SITE	SUMMARY OF SUBSURFACE CONDITIONS
East Abutment (Probe Hole 1)	Soil stratigraphy included a 300 mm thick topsoil over 800 mm cohesionless deposits of silt. Probable bedrock surface encountered at 1.1 m depth, elevation 291.0 m.
Centre Pier (Borehole 2)	Soil stratigraphy included 500 mm topsoil overlying 9.7 m thick silt unit which in turn underlain by 2.9 m thick cohesive silty clay matrix. A 22.9 m thick cohesionless silt unit was encountered below the silty clay, which mantled the probable bedrock at 36.0 m depth, elevation 253.9 m.
West Abutment (Borehole 3)	Soil stratigraphy encountered at the west abutment location included 600 mm fill overlying 500 mm thick topsoil unit which in turn overlying a 4.4 m thick silt matrix. A 6.4 m thick cohesive clayey unit is underlying the silt unit, which in turn is overlying 26.2 m thick cohesionless silt unit, which mantled the probable bedrock at 38.1 m depth, elevation 249.6 m.

TABLE 2

ADVANTAGES AND DISADVANTAGES, RELATIVE COSTS AND RISKS/CONSEQUENCES

ALLENSVILLE UNDERPASS

STRUCTURE FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/CONSEQUENCES	RANK
East Abutment Shallow Foundations - Spread footings	• Conventional construction methods • Spread footings on engineered fill may use higher bearing resistances	• Low geotechnical resistances requires large footings	• Less costly than deep foundations	• Low risk	1
	• Semi-integral abutment design is possible	• Requires bedrock trench to provide adequate pile length	• Costly than shallow foundations	• Work with piling equipment near existing highway requires special care	2
Centre Pier and West Abutment Deep Foundations - Steel H-Piles	• High load carrying capacities are obtained on piles to the bedrock • Integral abutment design is possible with pile foundations	• Requires heavy pile driving equipment • Higher cost than shallow foundations • Requires surcharging of site to reduce negative skin friction	• More costly than shallow foundations	• Work with piling equipment near existing highway requires special care	1
Deep Foundations - Caissons	• High load bearing capacity	• Low soil resistances require deep installations below water table (not practical)	• More costly than shallow foundations	• Unwatering of caisson holes may not be feasible	2 (not practical)
APPROACH EMBANKMENTS	**ADVANTAGES**	**DISADVANTAGES**	**RELATIVE COSTS**	**RISKS/CONSEQUENCES**	**RANK**
Surcharging without Soil Removal	• Excavation near existing embankment are not required • Post-construction settlements are mitigated	• Requires preloading/surcharging to mitigate long-term settlement of approach embankment	• Lower cost than soil removal option	• Possible post-construction settlements of new roadway may need repair or maintenance	1
Removal of Compressible Soils	• Reduced long-term settlements	• Excavation of cohesive soil not practical	• Higher cost than surcharge option	• Excavation may cause instability to existing highway embankment	2 (not practical)



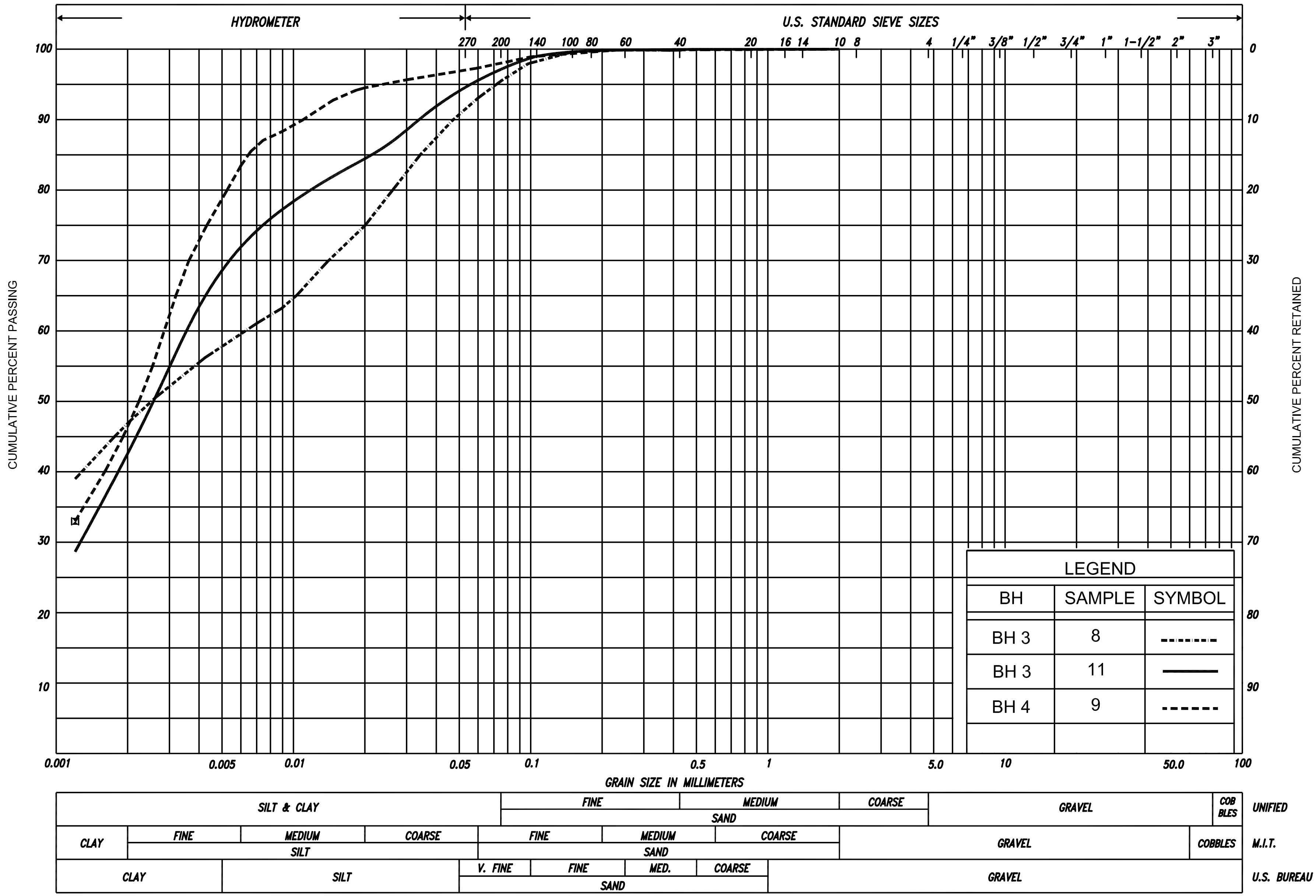
LEGEND		
BH	SAMPLE	SYMBOL
BH2	4	ENVELOPE
BH 2	7	
BH2	10	
BH 2	13	
BH 3	3	
BH 3	12	
BH 3	14	
BH3	16	

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILT, some to with sand, trace clay, trace gravel

FIG No.	AR-GS-1
HWY	11
W.P. No.	320-00-00

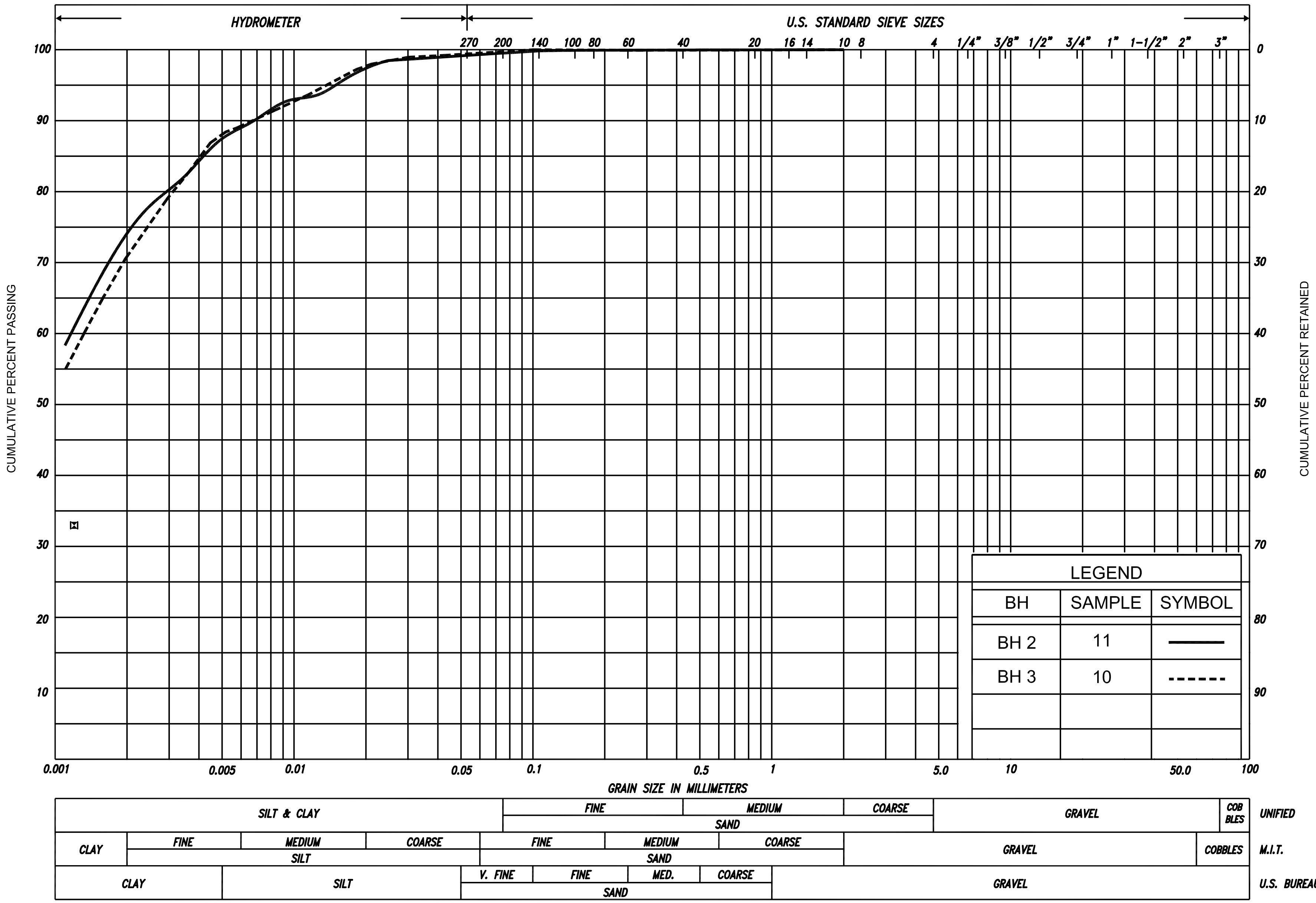


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

FIG No.	AR-GS-2
HWY	11
W.P. No.	320-00-00

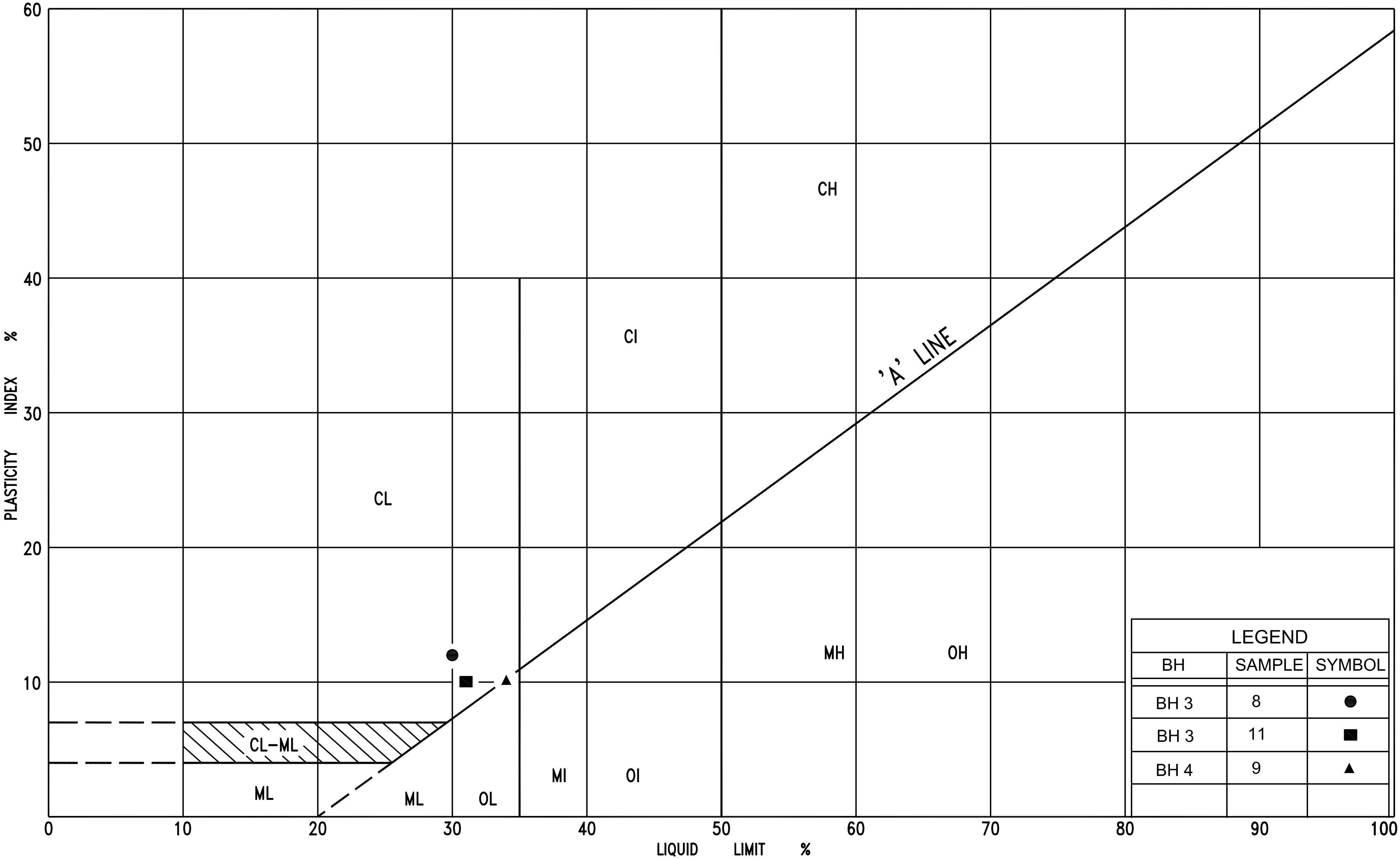


Ministry of Transportation Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand

FIG No.	AR-GS-3
HWY	11
W.P. No.	320-00-00

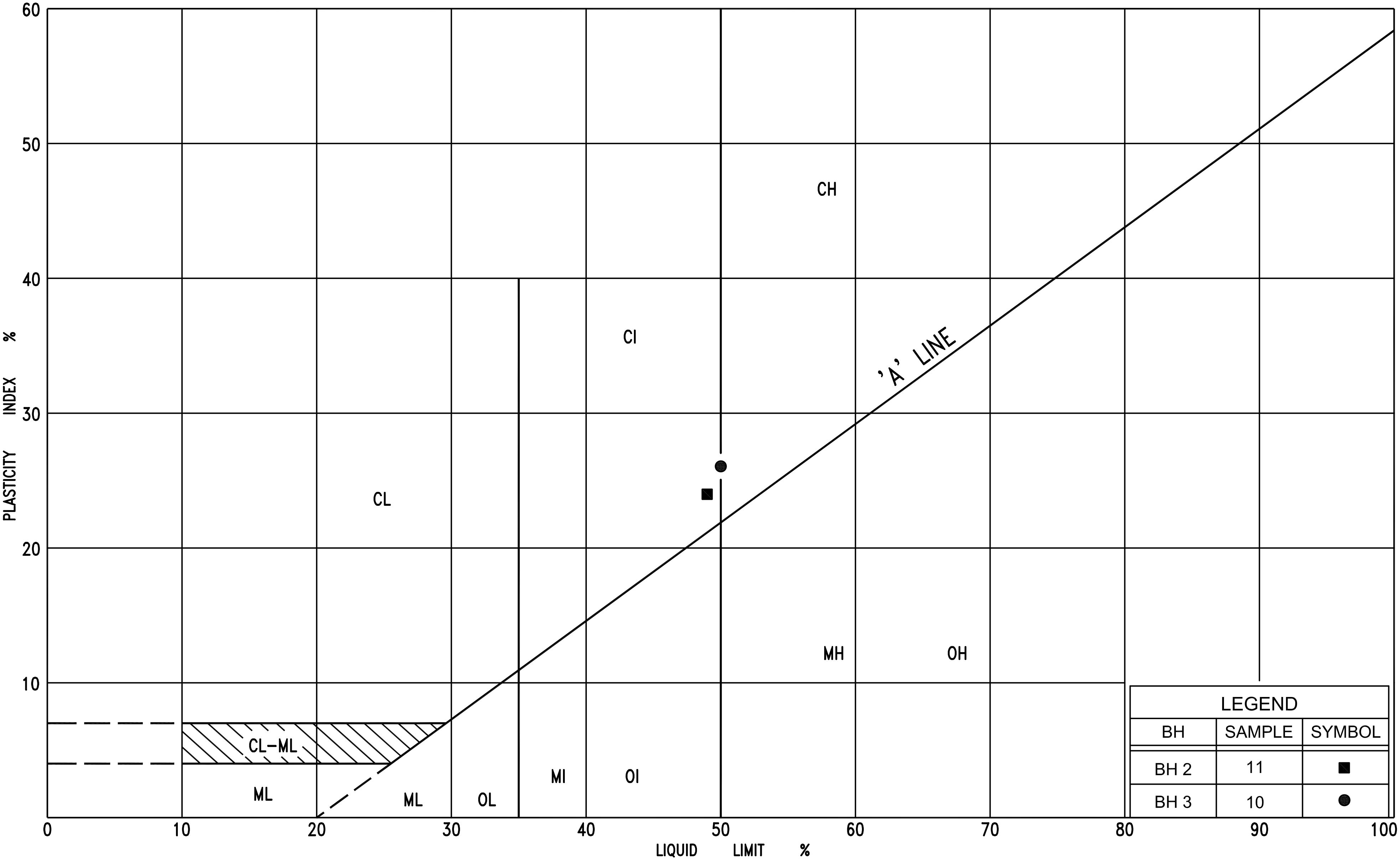


Ministry of Transportation
Ontario

PLASTICITY CHART

CLAYEY SILT, trace sand

FIG No.	AR-PC-1
HWY:	11
W.P. No.	320-00-00



LEGEND		
BH	SAMPLE	SYMBOL
BH 2	11	■
BH 3	10	●

Ministry of Transportation
Ontario

PLASTICITY CHART

SILTY CLAY, trace sand

FIG No.	AR-PC-2
HWY:	11
W.P. No.	320-00-00

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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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
W S	WASH SAMPLE
S T	SLOTTED TUBE SAMPLE
B S	BLOCK SAMPLE
C S	CHUNK SAMPLE
T W	THINWALL OPEN
F V	FIELD VANE
T P	THINWALL PISTON
O S	OSTERBERG SAMPLE
R C	ROCK CORE
P H	T W ADVANCED HYDRAULICALLY
P M	T W ADVANCED MANUALLY
F S	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_α	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_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 320-00-00 LOCATION Highway 11 Coords: 5 011 811.6 N; 320 660.4 E ORIGINATED BY F.P.

DIST 54 HWY 11 BOREHOLE TYPE Manual Probe COMPILED BY N.R.

DATUM Geodetic DATE April 09, 2009 CHECKED BY C.N.

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
292.1						*					
0.0	Topsoil						292				
291.8 / 0.3	Silt, some sand, trace gravel cobbles — Brown Moist										
291.0 / 1.1	End of borehole Refusal on probable bedrock						291				

* Borehole dry

Bedrock exposed 5m east of probe hole location.

+7, ×5: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 320-00-00	LOCATION	Highway 11 Coords: 5 011 823.5 N; 320 621.7 E	ORIGINATED BY F.P.
DIST 54 HWY 11	BOREHOLE TYPE	C.F.H.S.A. and Dynamic Cone Penetration Test	COMPILED BY N.R.
DATUM Geodetic	DATE	April 13, 2009	CHECKED BY C.N.

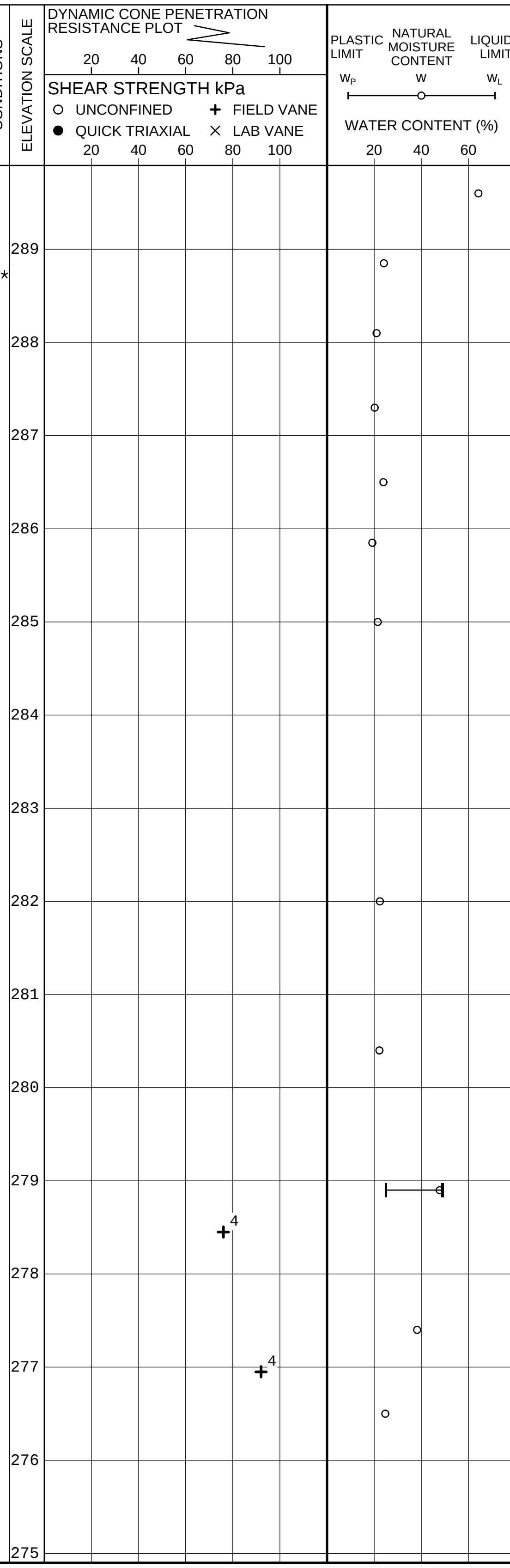
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
289.9	
0.0	Topsoil
289.4 / 0.5	Silt, trace to with sand, trace clay, organic inclusions Loose Brown Moist oxidized pockets Compact clayey silt layers Loose Grey some clay
279.7 / 10.2	Silty clay, trace sand, clayey silt layers Stiff Grey Wet
276.8 / 13.1	Silt, some clay, trace sand Compact Grey Moist
274.9	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3	
2	SS	6	
3	SS	11	
4	SS	10	0 14 78 8
5	SS	11	
6	SS	12	
7	SS	12	0 24 72 4
8	SS	6	
9	SS	4	
10	SS	6	0 7 79 14
11	SS	7	0 1 25 74
	FV		
12	SS	3	
	FV		
13	SS	23	0 1 85 14

GROUND WATER CONDITIONS: ∇*



UNIT WEIGHT γ kN/m³

ON_MOT VER3 08TF008.GPJ ON_MOT.GDT 6/16/2009 11:01:04 AM

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 320-00-00 LOCATION Highway 11 Coords: 5 011 823.5 N; 320 621.7 E ORIGINATED BY F.P.

DIST 54 HWY 11 BOREHOLE TYPE C.F.H.S.A. and Dynamic Cone Penetration Test COMPILED BY N.R.

DATUM Geodetic DATE April 13, 2009 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
274.9 15.0	Silt trace to some clay, trace sand Compact Grey Moist	14	SS	17		~25
					274	
					273	
					272	
		15	SS	20		~20
					271	
					270	
					269	
		16	SS	20		~21
					268	
	Loose				267	
					266	
		17	SS	7		~19
					265	
	Compact				264	
					263	
		18	SS	18		~22
					262	
					261	
259.9	Cont'd				260	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+7, ×5 : Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 320-00-00	LOCATION Highway 11 Coords: 5 011 823.5 N; 320 621.7 E		ORIGINATED BY F.P.
DIST 54 HWY 11	BOREHOLE TYPE C.F.H.S.A. and Dynamic Cone Penetration Test		COMPILED BY N.R.
DATUM Geodetic	DATE April 13, 2009		CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
259.9											
30.0	Silt, trace to some clay trace sand Compact Grey Moist		19	SS	18		259		o		
258.8 31.1	End of borehole Probable silt Compact to dense						258 257 256 255				
253.9 36.0	End of dynamic cone penetration test						254	141			

* 2009 04 13

∇ Water level observed during drilling

■ Penetrometer test

C.F.H.S.A. Denotes Continuous Flight Hollow Stem Augers

+7 , ×5 : Numbers refer to Sensitivity

20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3

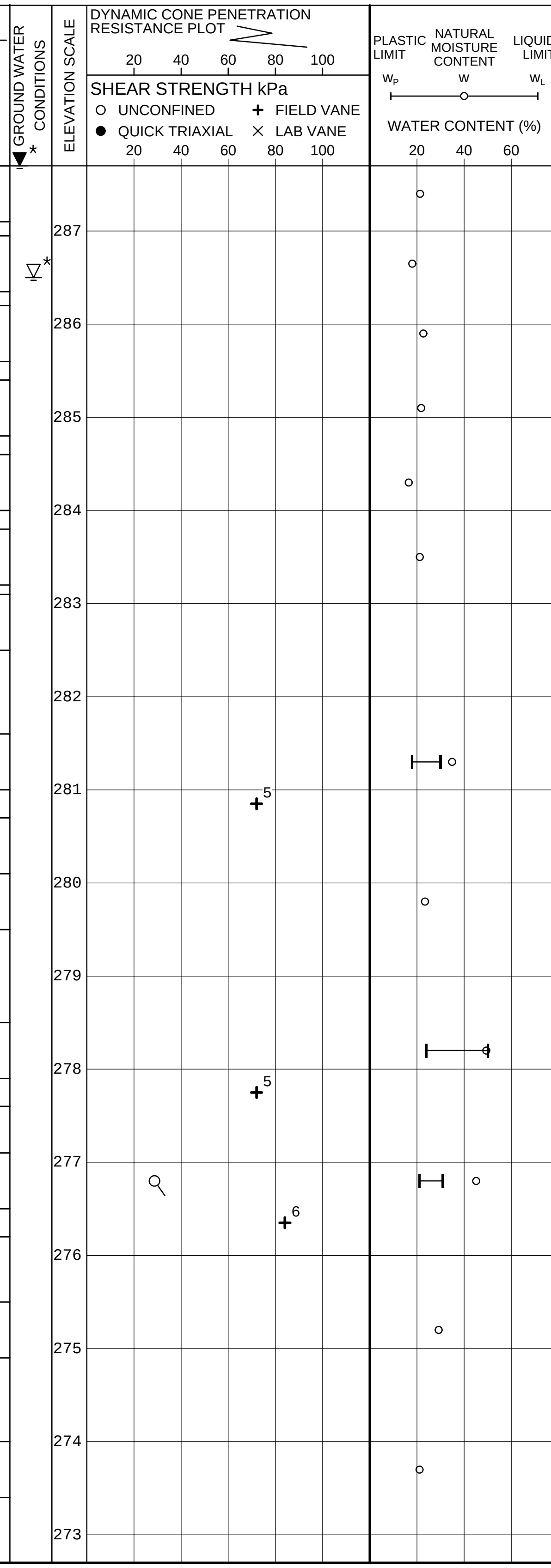
METRIC

G.W.P. 320-00-00 | LOCATION Highway 11 Coords: 5 011 834.8 N; 320 586.9 E | ORIGINATED BY F.P.

DIST 54 HWY 11 | BOREHOLE TYPE C.F.H.S.A. + Wash Boring | COMPILED BY N.R.

DATUM Geodetic | DATE April 07 & 08, 2009 | CHECKED BY C.N.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
287.7									
0.0	Sandy silt, topsoil inclusions, Brown, Moist (FILL)	1	SS	2					
287.1 / 0.6	Topsoil								
286.6 / 1.1	Silt, trace to some sand, trace clay, oxidized layers; Compact, Grey / Brown, Wet	2	SS	16					
		3	SS	16		1	15	77	7
	trace sand; Loose, Grey	4	SS	10					
		5	SS	7					
		6	SS	8					
		7	SS	6					
282.2 / 5.5	Clayey silt, trace sand; Stiff, Grey, Wet	8	SS	3		0	5	48	47
			FV						
		9	SS	2					
279.2 / 8.5	Silty clay, trace sand, silt layers; Stiff, Grey, Wet	10	SS	4		0	1	28	71
			FV						
277.2 / 10.5	Clayey silt, trace sand; Firm to stiff, Grey, Wet	11	TW	PM	17.3	0	3	54	43
			FV						
275.8 / 11.9	Silt, some clay, trace sand; Compact, Grey, Moist	12	SS	20		0	1	85	14
	sandy silt layers	13	SS	14					
272.7	Cont'd								



ON_MOT VER3 08TF008.GPJ ON_MOT.GDT 6/16/2009 11:01:06 AM

+7, ×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

G.W.P.	320-00-00
LOCATION	Highway 11, Coords: 5 011 834.8 N; 320 586.9 E
ORIGINATED BY	F.P.
DIST	54
HWY	11
BOREHOLE TYPE	C.F.H.S.A. + Wash Boring
COMPILED BY	N.R.
DATUM	Geodetic
DATE	April 07 & 08, 2009
CHECKED BY	C.N.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
272.7 / 15.0	Silt, trace clay, trace sand. Compact, Grey, Moist
	dense
	compact
257.7	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	Natural moisture content (approx. %)	GR	SA	SI	CL
14	SS	11	~17	0	6	90	4
15	SS	26	~20				
16	SS	34	~20	0	6	91	3
17	SS	11	~20				
18	SS	18	~20				

Elevation scale: 272, 271, 270, 269, 268, 267, 266, 265, 264, 263, 262, 261, 260, 259, 258

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

+7, ×5 : Numbers refer to Sensitivity

20 / 15 —○— 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd. Foundation Design

RECORD OF BOREHOLE No 3

 METRIC

G.W.P. 320-00-00 LOCATION Highway 11 Coords: 5 011 834.8 N; 320 586.9 E ORIGINATED BY F.P.

DIST 54 HWY 11 BOREHOLE TYPE C.F.H.S.A. + Wash Boring COMPILED BY N.R.

DATUM Geodetic DATE April 07 & 08, 2009 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.7									
30.0	Silt trace clay, trace sand Compact Grey Moist					257			
		19	SS	29			~20		
						256			
						255			
		20	SS	18		254	~21		
						253			
						252			
		21	SS	17		251	~22		
						250			
249.6 / 38.1	End of borehole Refusal on probable bedrock								

* 2009 04 07, 08

∇ Water level observed during drilling

▼ Water level measured after drilling

C.F.H.S.A. Denotes Continuous Flight Hollow Stem Augers

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

$+^7$, $\times^5$: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 4

METRIC

G.W.P. 320-00-00 LOCATION Highway 11 Coords: 5 011 744.9 N; 320 575.2 E ORIGINATED BY F.P.

DIST 54 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.R.

DATUM Geodetic DATE April 09, 2009 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
287.1					287			
0.0	Peat	1	SS	1				
286.6 / 0.5	Silt, trace to some sand, trace clay, organic inclusions; Loose, Grey / brown, Wet	2	SS	9	286		o	
	Compact, Reddish brown, Moist	3	SS	15	285		o	
		4	SS	13	284		o	
	Loose	5	SS	8			o	
	clayey silt layers	6	SS	6	283		o	
		7	SS	6	282		o	
281.5 / 5.6	Clayey silt, trace sand; Stiff, Grey, Wet				281			
		8	SS	4		■	o	
			FV		280	+ 6		
		9	TW		279		o	0 2 52 46
			FV			+ 5	o	
278.3 / 8.8	Silt, trace clay, trace sand; Compact, Grey, Moist	10	SS	25	278		o	
		11	SS	17	277 / 276		o	
	Loose	12	SS	9	275		o	
	Compact	13	SS	16	274 / 273		o	
272.8 / 14.3	End of borehole							

Cont'd

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT; REMARKS & GRAIN SIZE DISTRIBUTION (%)

+7, ×5 : Numbers refer to Sensitivity

20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 4

METRIC

G.W.P. 320-00-00 LOCATION Highway 11 Coords: 5 011 744.9 N; 320 575.2 E ORIGINATED BY F.P.

DIST 54 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.R.

DATUM Geodetic DATE April 09, 2009 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	* 2009 04 09										
	▽ Water level observed during drilling										
	▼ Water level measured after drilling										
	■ Penetrometer test										

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS

CONT No

GWP No 320-00-00

CROSSING ROAD I/C UNDERPASS

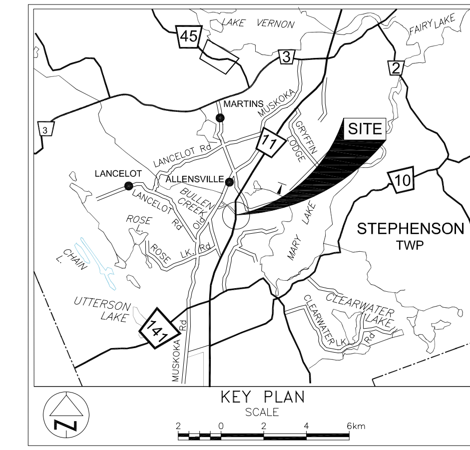
HIGHWAY 11

BOREHOLE LOCATIONS

SHEET

Peto MacCallum Ltd.

CONSULTING ENGINEERS



LEGEND

●	Borehole
⊕	Dynamic Cone Penetration Test (Cone)
	Borehole & Cone
N	Blows/0.3m (Std. Pen Test, 475 J/blow)
CONE	Blows/0.3m (60 Cone, 475 J/blow)
	W L at time of investigation: April, 2009
	Head
	ARTESIAN WATER Encountered
	PIEZOMETER

BH No	ELEVATION	CO-ORDINATES NORTHINGS	EASTINGS
1	292.1	5 011 811.6	320 660.4
2	289.9	5 011 823.5	320 621.7
3	287.7	5 011 834.8	320 586.9
4	287.1	5 011 744.9	320 575.2

PLAN

SCALE

10 0 10 20m

– NOTE –

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

NOTE:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

REGISTERED PROFESSIONAL ENGINEER C M P Nascimento June 17, 2009 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER B. R. GRAY June 17, 2009 PROVINCE OF ONTARIO

REF No. STANTEC Drawing; 674–design_Recommended.dwg; dated February 25, 2009

Geocres No. 31E–295

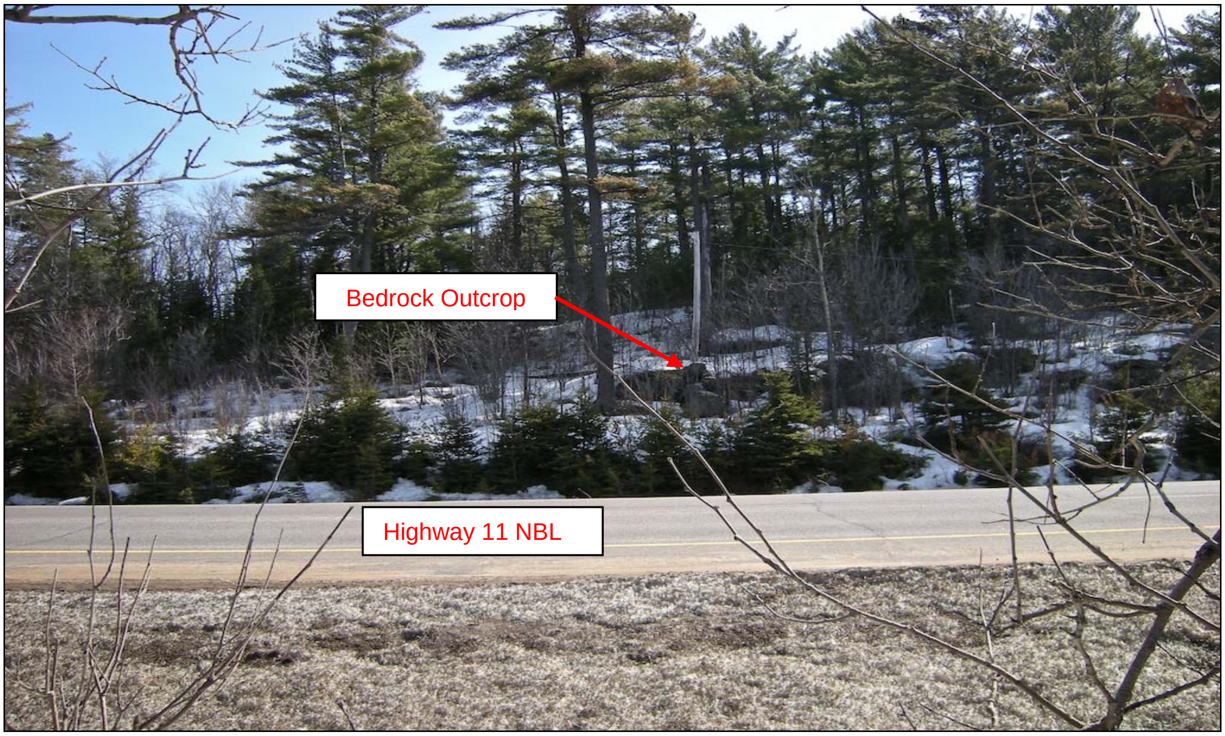
HWY No 11			DIST 52
SUBM'D MN	CHECKED NR	DATE JUNE 17, 2009	SITE --
DRAWN NA	CHECKED CN	APPROVED BRG	DWG AR–1

APPENDIX A

SITE PHOTOGRAPHS

Photograph 1: Looking west from the west shoulder of Highway 11 SBL, approximate Station 20+215 Tip of a plastic culvert visible at the foreground of the photograph.

Photograph 2: Looking east from the median at about Sta. 20+215. A thick underbrush visible in the median of the Highway 11 lanes.



Photograph 3: Viewing east from the median to the east side of Highway 11 NBL at approximate Sta. 20+215. A hill is visible in the background of the photograph with bedrock outcrops exposed.

Photograph 4: Viewing east from the east shoulder of Highway 11 NBL at approximate Sta. 20+215. Bedrock outcrops exposed in the background of the photograph.

APPENDIX B

Slope Stability Analyses Results

SLOPE STABILITY ANALYSIS RESULTS

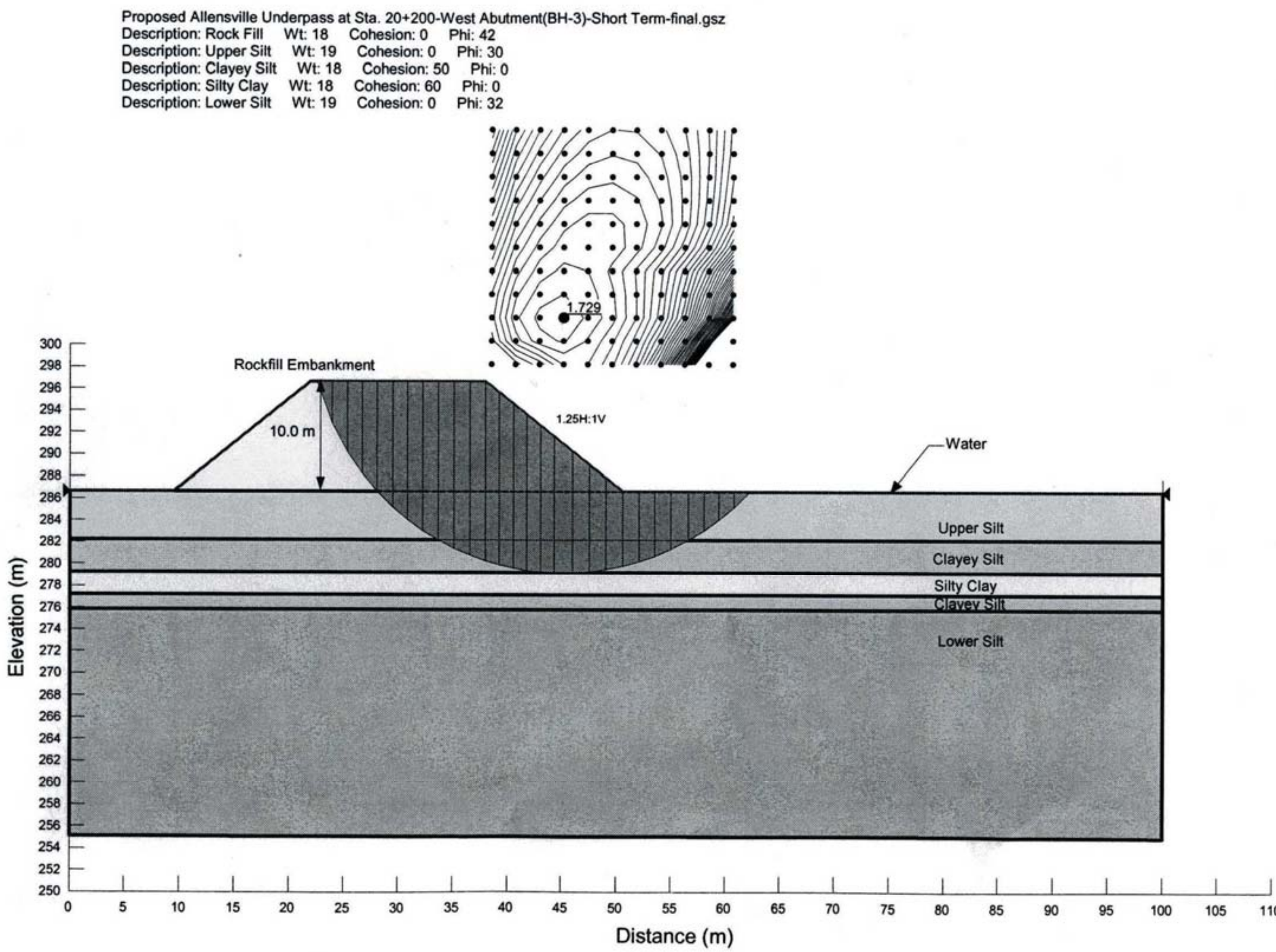


Figure 1

SLOPE STABILITY ANALYSIS RESULTS

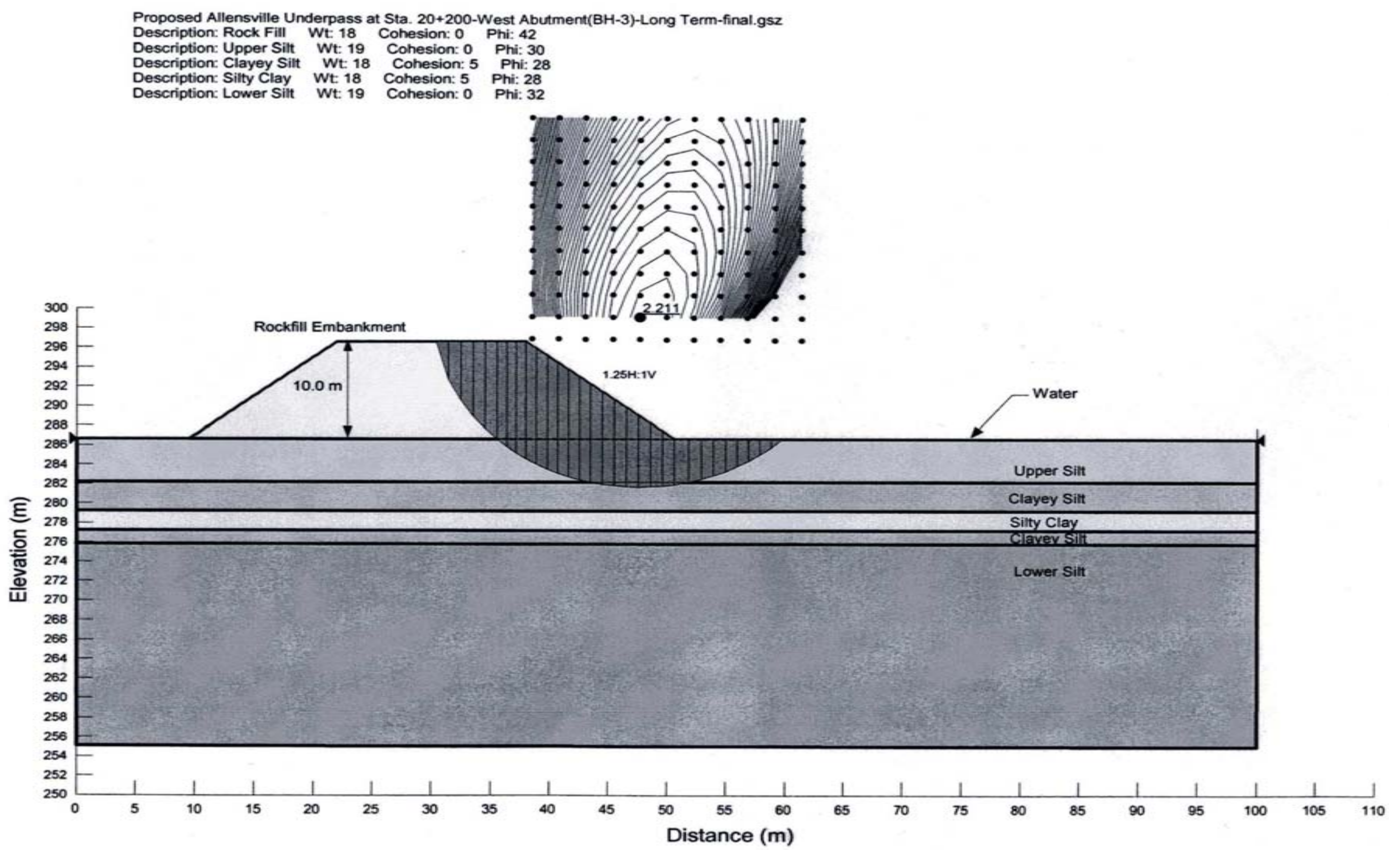


Figure 2

SLOPE STABILITY ANALYSIS RESULTS

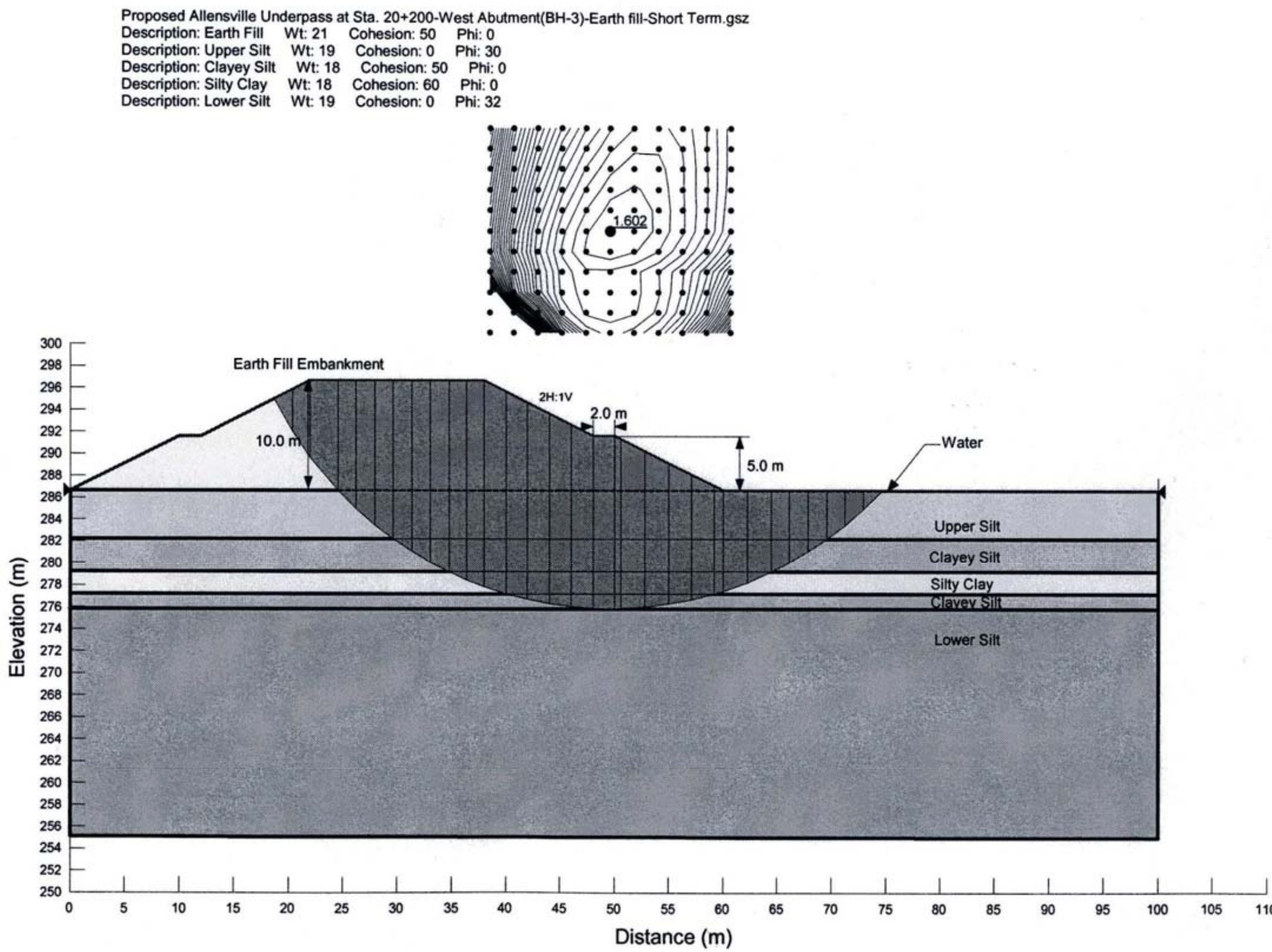


Figure 3

SLOPE STABILITY ANALYSIS RESULTS

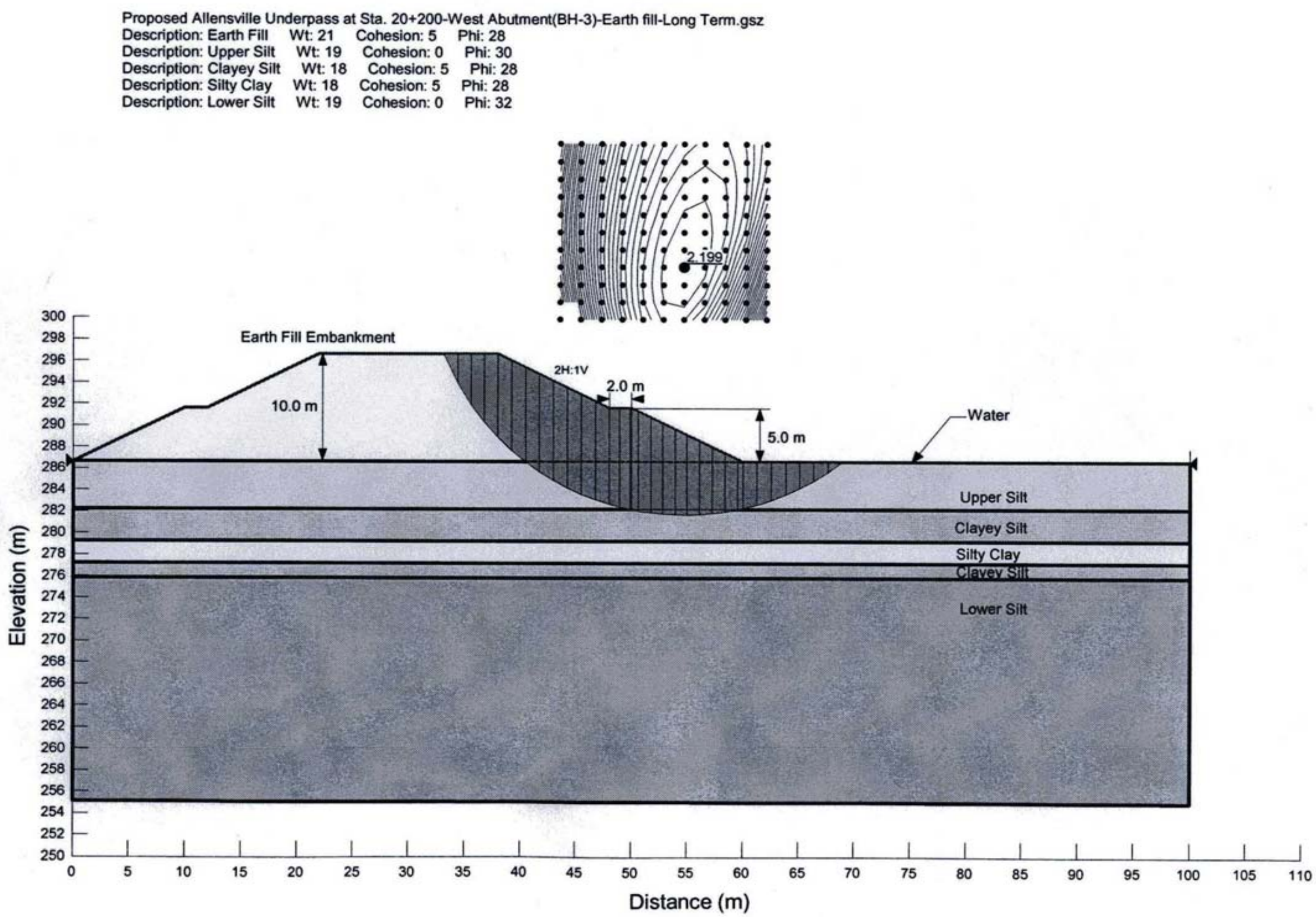


Figure 4