PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

PORT ROBINSON ROAD UNDERPASS
HIGHWAY 406 FOUR-LANING
GWP 280-99-00
CITY OF THOROLD, ONTARIO

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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
Port Robinson Road Underpass
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1. INTRODUCTION

This report summarizes the results of the preliminary foundation investigation carried out for the proposed Port Robinson Road Underpass at Highway 406 in the City of Thorold. Peto MacCallum Ltd. (PML) conducted the preliminary foundation investigation for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The bridge is part of the twinning of the Highway 406 section that extends from Port Robinson Road in the City of Thorold southerly 5.6 km to East Main Street in the City of Welland, Ontario. The proposed new underpass will carry the realigned Port Robinson Road traffic over the proposed Highway 406 northbound and southbound lanes at approximate Sta. 16+600 (proposed Highway 406 chainage). The location of the structure was changed after the subsurface investigation was carried out, however the obtained data was considered to be adequate for preliminary design purposes.

This preliminary report pertains to the bridge structure and approach embankments within about 20 m of the abutments and is considered to be suitable for planning and preliminary design purposes and should not be used for detail design. As specified by MTO, the preparation of this report follows the terms of reference (TOR) outlined in the original request for proposal (April 19, 2000).

2. SITE DESCRIPTION AND GEOLOGY

The contemplated structure is proposed immediately south of the existing Port Robinson Road and Highway 406 at-grade crossing. The site is about 878 m north of the existing Merritt Road intersection at Highway 406.

Land use in the vicinity of the site comprises the existing transportation corridors of the Port Robinson Road and Highway 406 at-grade intersection. The proposed Port Robinson Road Underpass will run roughly east to west. The local topography of the structure site is generally flat to gently rolling. The ground cover beyond the paved highway comprises grasses and isolated stands of trees. A small creek crosses Highway 406 about 100 m north of the existing intersection.

The site is located in the Haldimand Clay Plain physiographic region. The soil cover in the region typically comprises lacustrine silts and clays. Dolostone bedrock of the Salina Formation is anticipated at an approximate depth of 35 m.

3. INVESTIGATION PROCEDURES

The field work was carried out on September 29 and October 14, 2008. Two sampled boreholes were put down at the site. The boreholes were drilled to practical refusal at depths of 33.6 and 34.2 m at the locations shown on Drawing PR-1. Borehole PR1 was extended by coring 3.0 m into bedrock to a total depth of 37.2 m, elevation 145.0.

The locations of boreholes were established by PML and the ground surface elevations at the boreholes in the field were surveyed by Callon Dietz Inc.

The boreholes were advanced using continuous flight hollow stem augers and rotary diamond drilling, powered by a truck-mounted CME-75 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member off our engineering staff.

Representative soil samples were recovered in the boreholes at depth intervals of 0.75, 1.5 and 3.0 m in accordance with the TOR. The soil samples were obtained using a split spoon sampler in conjunction with standard penetration tests. In-situ vane shear strength and penetrometer testing was also performed to further assess the undrained shear strength of the cohesive soils. It is noted that the results of penetrometer tests may be lower than the actual values due to sample disturbance.

In borehole PR1, an approximate 3.0 m length of rock core was recovered using NXL rock coring equipment. The PML senior geologist examined and classified the recovered rock core samples. Detailed descriptions of the recovered rock core are provided in Table A.

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

Upon completion of augering, the boreholes were backfilled with auger cuttings and hole plug bentonite to the ground surface in accordance with current Regulation 903 and MTO guidelines.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, soil classification and laboratory testing. The visual examination indicated that the soils are typical of the Haldimand Clay Plain. The laboratory testing program comprised the following tests:

- Natural moisture content determinations (36)
- Grain size analyses (10)
- Atterberg limits (8)

The results of the laboratory natural moisture content determinations, grain size analyses and Atterberg limits are shown on the Record of Borehole sheets. The grain size distribution charts are presented on Figures GS-PR-1 to GS-PR-6. The Atterberg limits results are presented on Figures PC-PR-1 to PC-PR-4.

4. SUMMARIZED SUBSURFACE CONDITIONS

4.1 General

Refer to the attached Record of Borehole Sheets for the details of the subsurface conditions including soil classifications, inferred stratigraphy, soil and rock boundary levels and groundwater observations.

The borehole locations and the preliminary layout of the Port Robinson Road underpass are presented on the attached Foundation Drawing PR-1.

The subsurface stratigraphy revealed in the boreholes generally consisted of a 0.7 to 1.4 m thick surficial fill unit made up of compact gravelly sand and very stiff clayey silt covering a localized 0.7 m thick compact sand layer in borehole PR2 and 16.2 and 14.7 m thick firm to very stiff deposit of cohesive soils (upper layers). These upper cohesive deposits overlay an about 3.0 m thick loose to compact silt layer underlain by 9.0 and 9.5 m thick firm to very stiff reddish brown cohesive soils (lower layers).

A 5.7 and 6.7 m thick, hard to very stiff layer of reddish brown clayey silt/silty clay till was contacted at about 29 m depth, elevation 153.0 and mantles dolostone bedrock at depths of 33.6 and 34.2 m. The strata encountered are summarized below.

4.2 <u>Fill</u>

Fill units were found surficially in both boreholes extending to 0.7 and 1.4 m depths, elevations 181.5 and 180.5. The fill units consist of very stiff clayey silt and silty clay. A surficial 450 mm thick gravelly sand layer was encountered in borehole PR2. N values were 10 and 20. The laboratory moisture content of the fill units were 8 and 22%.

4.3 <u>Sand</u>

Beneath the fill units at 1.4 m depth (elevation 180.5), a 0.7 m thick compact sand layer with silt and some clay was encountered in borehole PR2.

The sand layer extended to an underlying cohesive clayey silt/silty clay/clay deposit at a depth of 2.1 m (elevation 179.8).

The sand layer was compact with N value of 18. The grain size distribution chart of representative sample of the deposit is shown on Figure GS-PR-4. The sand sample was non-plastic according to Atterberg limits determination. A water content of 18% was measured on one sample.

4.4 Silty Clay/Clayey Silt/Clay (Upper Layers)

Beneath the fill units at depth of 0.7 m (elevation 181.5) in borehole PR1 and below the 0.7 m thick sand layer at 2.1 m depth (elevation 179.8) in borehole PR2, 9.5 and 9.0 m thick cohesive clayey silt/silty clay/clay deposits were encountered in both boreholes, respectively. The units extended to an underlying silt layer at uniform depths of 16.9 and 16.8 m (elevations 165.3 and 165.1). The deposits contain sand and scattered layers of silt.

The cohesive soils were typically firm to stiff with very stiff layers at depths of 3.0 and 2.1 m in boreholes PR1 and PR2, respectively. In both boreholes, the cohesive soils were firm to stiff from depths of 5.4 and 5.5 m to depths of 16.9 and 16.8 m (elevations 176.4 and 176.8 to 165.3 and 165.1). N values varied from 2 to 25 typically in the range of 3 to 5. Field vane tests indicated shear strength of 50 to 190 kPa and sensitivity of 2 and 4. Penetrometer tests indicated undrained shear strengths of 25 to 78 kPa.

The grain size distribution charts of representative samples of the deposits are shown on Figures GS-PR-1 to GS-PR-3 and the Atterberg plasticity limits on the Plasticity Chart Figures PC-PR-1 to PC-PR-3. The liquid limits of the deposits varied between 32 and 55 and the plastic limits between 18 and 24, giving the plasticity index values of 14 and 31. The water content of the clayey silt varied from 21 to 43% typically in the range of 21 to 29%, indicating that the materials were typically at natural water content lower than the liquid limits for relatively low compressibility characteristics. The Atterberg limits for the upper cohesive soils are listed in the Table below:

MATERIAL	BOREHOLE	SAMPLE	DEPTH (m)	WATER CONTENT (%)	LIQUID LIMIT (W_L)	PLASTIC LIMIT (W_P)	PLASTICITY INDEX (I_P)
Clay	PR2	9	10.7 – 11.3	43	55	24	31
Silty Clay	PR1	4	3.0 – 3.6	22	49	22	27
	PR1	6	6.1 – 6.7	29	35	18	17
	PR1	9	10.7 – 11.3	40	48	23	25
Clayey Silt	PR2	5	4.6 – 5.2	22	32	18	14

4.5 Silt

A deposit of cohesionless loose to compact silt was encountered in both boreholes below the upper cohesive soils at depths of 16.9 and 16.8 m (elevations 165.3 and 165.1). The silt deposit contains trace sand and gravel and silty clay/clayey silt layers. The thickness of the deposit was about 3.0 m overlying a lower layer of cohesive soils. The N values in the silt typically ranged from 2 to 8. One N value of zero (sampler sunk under the weight of the rods and hammer) was likely caused by sample disturbance because the material exhibits high dilatancy characteristics.

The water content of the silt ranged between 20 and 28%, indicating wet conditions. The grain size distribution chart of representative sample of the silt is shown on Figure GS-PR-5. The silt sample was non-plastic according to an Atterberg limits determination.

4.6 Clay Silt/Silty Clay (Lower Layers)

Cohesive deposits of clayey silt/silty clay (lower layers) with a characteristic reddish brown colour were present below the silt in both boreholes at a uniform depth of 19.8 m (elevations 162.4 and 162.1). The deposits were 9.0 and 9.5 m thick extending to the underlying clayey silt till at depths of 28.8 and 29.3 m (elevations 152.9 and 153.1), respectively.

The deposit exhibited firm to very stiff consistency. N values ranged from 2 to 19. One field vane test carried out in the deposit indicated that the undisturbed shear strength value was 58 kPa (soil sensitivity of 1, indicating high silt content).

The grain size distribution charts of representative samples of the deposit are shown on Figure GS-PR-2, and the Atterberg plasticity limits on the Plasticity Chart Figure PC-PR-2. The liquid limit for the clayey silt was 31 and the plastic limit 18, giving the plasticity index value of 13. The water content of the deposit varied from 19 to 33%. Details of the Atterberg limits determined on a single sample of these deposits are listed below:

MATERIAL	BOREHOLE	SAMPLE	DEPTH (m)	WATER CONTENT (%)	LIQUID LIMIT (W_L)	PLASTIC LIMIT (W_P)	PLASTICITY INDEX (I_P)
Clayey Silt	PR2	16	24.4 – 25.0	30	31	18	13

4.7 Clayey Silt Till/Silty Clay Till

A deposit of hard very stiff clayey silt till/silty clay till was encountered in both boreholes below the lower cohesive soils at depths of 28.8 and 29.3 m (elevations 152.9 and 153.1), respectively. The thickness of the till was 4.8 and 4.9 m. In borehole PR1, the till was underlain by a thin 0.3 m thick layer of sandy silt. The till and localized thin sandy silt layer overlay bedrock at 33.6 and 34.2 m depths, elevations 148.3 and 148.0, respectively. The cohesive till deposit contains variable sand content from trace to sandy, trace to with gravel content and scattered cobbles.

In borehole PR1, N values in the till were 24 and 39, while in borehole PR2, N values ranged from 96 blows per 250 mm penetration to 50 blows per 100 mm penetration. The higher N values in borehole PR2 are likely due to gravel and cobbles presence in the deposit.

The grain size distribution chart of representative samples of the deposits is shown on Figure GS-PR-6 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-PR-4. The liquid limits of the clayey silt deposits were 17 and 25 and the plastic limits were 11 and 17, giving relatively low plasticity index values of 6 and 8. The water content of the clayey silt till was about 12% and in the localized silty clay till 32%. The underlying sandy silt was at a moisture content of 12%. These water contents indicated that the soils were in a moist condition and relatively low compressibility characteristics. Details of the Atterberg limits are listed in the Table below.

MATERIAL	BOREHOLE	SAMPLE	DEPTH (m)	WATER CONTENT (%)	LIQUID LIMIT (W_L)	PLASTIC LIMIT (W_P)	PLASTICITY INDEX (I_P)
Clayey Silt Till	PR1	18	30.5 – 31.1	12	25	17	8
	PR2	18	30.5 – 31.0	N/A	17	11	6

4.8 Bedrock

Dolostone bedrock of the Salina Formation was encountered in both boreholes below the native soils at the levels listed in the following table. In borehole PR2 the bedrock was inferred by refusal to auger penetration and sampling.

BOREHOLE No.	DEPTH (m)	ELEVATION	ROCK CORE LENGTH (m) (*)
PR1	34.2	148.0	3.0
PR2	33.6	148.3	N/A

(*) NXL rock cores obtained.

The core recovery was 98 and 100% for both core runs from borehole PR1. The rock exhibited medium strength and was found to be slightly weathered to unweathered. The rock typically is of very poor to poor quality (RQD values were 16 and 31%). Loss of drilling water was not experienced during drilling. The detailed rock core descriptions are provided in Table A.

4.9 Groundwater

Upon completion of drilling groundwater was measured in borehole PR2 at a depth of 9.8 m (elevation 172.1). The groundwater table was not determined upon completion of drilling in borehole PR1 because the borehole was charged with water for rock coring purposes but was observed at 16.9 m during drilling.

The groundwater levels are subjected to fluctuations due to seasonal and rainfall patterns.

5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. C.M. Nascimento, P.Eng., Project Manager. The drilling equipment was supplied by Elite Drilling. Laboratory testing was carried out in the PML Toronto laboratories.

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
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City of Thorold, Ontario

6. ENGINEERING RECOMMENDATIONS

6.1 General

Part B of this report provides the preliminary foundation engineering recommendations regarding design and comments for construction of the Port Robinson Road Underpass at the Highway 406 in the City of Thorold, Ontario. The recommendations are preliminary and based on the results of the limited subsurface investigation that was outlined in the Part A of this report.

The proposed new underpass will carry the Port Robinson Road traffic over the Highway 406 northbound and southbound lanes at approximate Sta. 16+600 (proposed Highway 406 chainage). As indicated by MRC, the span of the proposed two-span underpass structure is approximately 84 m. It is estimated that the approach embankments will be about 8 m high at the abutments and the ground surface at the abutments and pier is at approximately elevation 181.2.

In summary, the subsurface stratigraphy revealed in the boreholes generally consisted of 0.7 to 1.4 m thick surficial fill units made up of compact gravelly sand and very stiff clayey silt covering a localized 0.7 m thick compact sand layer in borehole PR2 and 16.2 and 14.7 m thick firm to very stiff deposit of cohesive soils (upper layers). These upper cohesive deposits overlay an about 3.0 m thick loose to compact silt layer underlain by 9.0 and 9.5 m thick firm to very stiff reddish brown cohesive soils (lower layers).

At about 29 m depth, elevation 153.0, a 5.7 and 6.7 m thick, hard to very stiff layer reddish brown clayey silt/silty clay till was contacted and mantles dolostone bedrock at depths of 33.6 and 34.2 m.

Use of conventional procedures to design and construct the underpass on deep or shallow type foundations is considered to be feasible.

The pile length will be about 36 m for piles driven to refusal on the bedrock underlying the site. It is also possible that some of the piles will find refusal in the hard cohesive till soils which contains cobbles found in borehole PR2 (west abutment). The geotechnical resistance of the deep foundations should be designed for bearing on the hard layer or on the bedrock at both abutments for preliminary design purposes. The piles should be provided with rock points due to potentially heavy driving through the glacial till which contains cobbles and (possibly) boulders.

The detail design of the structure should verify the added stresses on the underlying stiff clayey soils from the possible use of RSS walls.

The recommendations in this report are preliminary and based on PML's interpretation of the factual information obtained from a limited number of boreholes. A detailed foundation investigation will be required at the structure location during the Detail Design phase of the project. The foregoing “red-flag” issues and the interpretation and recommendations in this report are only provided for planning purposes and feasibility studies.

A list of the standard specifications referenced in the report is enclosed in Table 1.

6.2 Foundations

6.2.1 General

Based on the preliminary data, founding the proposed underpass structure on pile foundations driven to practical refusal on the hard cohesive soils or bedrock is considered feasible. Lightly loaded footings placed on the native stiff soils or on engineered fill may be used for semi-integral or conventional abutment design.

Drilled caissons bearing on the native soils or on the bedrock to support the underpass structure are not considered to be practical due to the presence of cobbles and (likely) boulders in the native soils, low carrying capacity of the cohesive soils, as well as groundwater presence relatively above the expected founding levels for caissons.

The seismic site coefficient for the stratigraphic conditions at this site is 1.0 [soil profile Type I, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6].

6.2.2 Deep Foundations

6.2.2.1 General

As indicated previously, conventional or integral/semi-integral abutment designs using driven piles are considered feasible at the site.

The preliminary pile foundation design recommendations for conventional and semi-integral abutments are provided on the following section together with additional recommendations for integral abutment foundations.

6.2.2.2 Conventional Abutment Considerations

Piles for the east and west abutments should be driven to refusal into the hard glacial till at the west abutment or to the bedrock at the east abutment. The preliminary estimated founding reference levels for the east and west abutments are provided in the following table:

FOUNDATION ELEMENT	FOUNDING MATERIAL	BOREHOLE No.	FOUNDING DEPTH (m)	STRATUM FOUNDING ELEVATION	ESTIMATED PILE TIP ELEVATION
East Abutment	Bedrock	PR1	34.2	148.0	148.0
West Abutment	Glacial Till	PR2	28.8	153.1	151.1
	Bedrock	PR2	33.6	148.3	148.3

Subject to verification during detail design, piles for the west abutment could be driven through the glacial till and to refusal into the bedrock.

The reference depths and elevations are taken from the existing ground surface at the borehole locations to the top of the founding stratum. About 1.5 to 2.0 m for pile embedment at refusal in the glacial till deposit should be allowed.

The piles will have to be driven through native soils containing firm to stiff clayey soils at the abutment locations. The existing grade at both abutments will be raised about 8 m. Consequently, the development of negative skin friction on the piles should be considered to the axial resistance at ultimate limit states (ULS) for the abutment piles.

Alternatively, these firm to stiff clayey soils can be preloaded with the approach embankment fill at the abutments, as discussed in Section 6.4. Should the approach embankments be preloaded as recommended, the negative skin friction could be neglected.

Based on hard glacial till soils at the pile tips at the west abutment and medium strength bedrock anticipated at the east abutment, the preliminary factored axial resistance at ultimate limit states (ULS) for a steel HP 310x110 pile is 1,600 kN.

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the maximum 36 m pile length required, the design is not expected to be governed by settlement since the required load causing that magnitude of deformation of the pile is larger than the recommended ULS factored capacity.

The capacity of the HP 310x110 piles should be reduced to allow for negative skin friction of 260 kN if the area is not preloaded and/or surcharged as recommended in Section 6.4 of this report.

The piles will set on a relatively level bedrock surface or on the hard glacial till at the abutments and should be equipped with rock points according to OPSD-3000.201 or with Titus H-Bearing Pile Standard Model according to SP 903S01. The rock points should be used to minimize the potential for damage when driving through the hard glacial till containing cobbles and likely boulders or by the driving on the bedrock surface.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

6.2.2.3 Integral Abutment Considerations

For the integral abutment design, the H-piles should be driven to the hard glacial till or bedrock anticipated at the depths/elevations and axial resistance are indicated in the previous section. The minimum 5.0 m long pile length below the abutment stem which should be incorporated in the design will not be a concern at this site.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

6.2.2.4 Lateral Resistances

The soil adjacent to the upper section of the piles is expected to comprise the compacted approach fill. Typically, cohesive very stiff to firm native cohesive soils will be locally present at depth below the embankment fill.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. For integral abutment piles, only the length below the annular space referred to previously should be considered. The assessed lateral resistance for the HP 310x110 pile section noted previously is as follows:

	FIRM SILTY CLAY / CLAYEY SILT / CLAY	**GRANULAR BACKFILL 'A' OR 'B' TYPE II**
Factored Lateral Resistance at ULS, kN	120	120
Lateral Resistance at SLS, kN	35	50

The assessed values of lateral resistance assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended. If greater resistance is required, batter piles should be installed.

To evaluate the point of contraflexture, the coefficient of horizontal subgrade reaction, k_s (MN/m^3) should be computed using the following equation:

Cohesionless Soils (Terzaghi, 1955)

	k_s	=	n_h z/b
where	n_h	=	coefficient related to soil density
		=	10.0 MN/m^3 for granular backfill
	z	=	depth, m
	b	=	pile width, m

The cohesionless soil parameter n_h is applicable to all granular fill materials to be provided along the piles.

The coefficient of horizontal subgrade reaction, k_s, for the native clayey soils should be taken as 25,000 kN/m^3 for preliminary design.

6.2.3 Shallow Foundations

6.2.3.1 Spread Footings on Native Soil

As indicated previously, supporting the abutments and pier of the underpass structure on conventional spread footings founded on native soil is considered to be feasible.

Assuming the final ground surfaces at the abutments to be at about elevation 181.2, spread footings should be constructed on the native soils comprising typically very stiff silty clay at elevation 180.0 (borehole PR1) and elevation 179.8 (borehole PR2). The recommended preliminary bearing resistance for spread footings constructed on the native soils is as follows:

Factored Geotechnical Resistance at ULS, kPa	375
Geotechnical Resistance at SLS, kPa	250

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m and groundwater level below founding depth was assumed for computation of the ULS resistance. A maximum footing width of 2.0 m should be used at this site to minimize the stress on the underlying stiff clayey deposits.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

Construction of the spread footings on native soil should be performed and monitored in accordance with OPSS 902 and SP 902S01 to verify the competency of the founding surface.

All footings subject to frost action should be provided with 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

6.2.3.2 Spread Footings On Structural Fill

Construction of the abutment footings on structural fill placed in the approach embankment could also be employed to support the foundation loads. The structural fill should comprise Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

A general sketch of the engineered fill geometry is enclosed in Figure 1. Footings should not be constructed on rockfill. However, rockfill may be placed adjacent to the Granular 'A' core. The recommended bearing resistance for 2.5 m wide footings constructed on structural fill is as follows:

Factored Geotechnical Resistance at ULS, kPa	900
Geotechnical Resistance at SLS, kPa	350

A minimum 2.5 m thickness of the structural fill pad was used for the computation. The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.70 is recommended for footings placed on granular fill.

6.3 Lateral Earth Pressures

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. For preliminary design, the lateral earth pressure, p (kPa) may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation.

$$p = K(\gamma h + q) + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/m^3

h = depth below final grade, m

q = surcharge load, kPa, if present

C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where $\varnothing$ = angle of internal friction of retained soil (35º for Granular A or Granular B Type II or Type III)

δ = angle of friction between the soil and wall (23.5º for Granular A or Granular B Type II or Type III)

Free-draining granular material should be used as backfill behind the wall. The following parameters are recommended for preliminary design:

PARAMETERS	GRANULAR A OR GRANULAR B TYPE II OR TYPE III
Internal Friction Angle, Ø (degrees)	35
Unit weight, γ (kN/m^3)	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient of Passive Earth Pressure, K_p	3.69

The assigned geotechnical parameter values are the same for all granular materials in view of their similar physical characteristics.

The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. We refer to Figure C6.16 of the CHBDC for this computation. The subsoil/backfill should be considered as medium dense sand for the project.

A subdrain system (SP 405F03) or weep holes (OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. Subdrains should be used where there is a potential for flooding behind the abutments. The subdrain tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipes should be installed on a positive grade and lead to frost-free outlets.

Where required, a retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 6.4 Approach Embankments 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 bearing resistances and founding levels recommended previously for spread footings constructed on native soils or structural fill should be employed for design of the RSS wall.

The supplier of the RSS should also be responsible for the detail 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.

6.4 Approach Embankments

The scope of work for this preliminary study did not require that boreholes be carried out for the approach embankments to the Port Robinson Road Underpass. Based on the acquired data, the approach embankments are likely to be founded on the upper layer of very stiff to firm cohesive soils underlain by loose to compact silt deposit followed by cohesive soils (lower layers) at the east and west abutments.

The approach embankments should be designed and constructed in accordance with OPSD-200.010, 201.010, 202.010, 3101.200 and SP 206S03. The side slopes of the approach embankments will be stable where they are inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rockfill.

It is noted that where the embankment fill height exceeds 8 or 10 m for earth and rockfill, respectively a 2 m wide mid-height berm will be required. The earth fill slopes, if employed, should be protected against surface erosion by sodding (OPSS 571) and suitable vegetation.

Based on the limited laboratory test data, it is estimated that some 90 mm of consolidation settlement of the clayey subgrade soils will occur at the approach embankments. This estimated settlement of cohesive soils is likely to take up to 24 months to occur to 80% completion. It is recommended that both approach embankment fills be placed and a 2 m high surcharge be applied for period of at least 18 months prior to driving the abutments piles. This surcharge period would eliminate or reduce the negative skin friction on the abutment piles. Further subsurface investigation and laboratory tests should be carried out during detail design for this analysis.

The backfill to the structure should be made of granular materials. The magnitude of the "consolidation" of these fills depends on the workmanship employed by the contractor and, if placed in 200 mm thick lifts compacted to 100% of standard Proctor maximum dry density in accordance with the requirements of SP 206S03 and OPSS 501 (Method A), should be in the order of 30 to 40 mm. These estimated total settlements of the approach fill surface near the abutments should be essentially complete within 3 to 6 months after placement of the fill.

6.5 Construction Considerations

6.5.1 Excavation

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations. For this purpose, the upper cohesive very stiff to firm silty clay encountered in the boreholes is considered Type 3 soil according to OHSA (Ontario Regulation 213/91) criteria.

6.5.2 Groundwater Control

Groundwater was observed during the course of the field work at a depth of 9.8 m (elevation 172.1) at the west abutment and will not affect the construction excavations. It is considered that seepage from soil and surface water run-off that enters the excavation should be readily handled by conventional sump pumping techniques.

Groundwater conditions should be further assessed during detail design.

7. ADDITIONAL STUDIES

The recommendations in this report are preliminary and are based on PML's interpretation of the factual information obtained from a limited number of boreholes and a visual site assessment. Detailed foundation investigations will be required at the structure location during the Detail Design phase of the project. The interpretation and recommendations are provided for planning and feasibility studies purposes only.

The following items should be considered for the detailed design studies.

1. Carry out the complete scope of detailed field investigations and laboratory testing at the structure site. Incorporate the data from the previously drilled boreholes included in this report for the Detail Design.

2. Determine/evaluate the slope of the bedrock founding surface to evaluate the need for steel pile rock points and the reduction of axial bearing resistance related to the extent of cobbles and likely extent of boulders in the hard glacial till mantling the dolostone bedrock.

3. Evaluate the characteristics of the native soils relevant to the consolidation under the added loading of the approach embankment fills and/or RSS walls to assess the pre-loading/ surcharging requirements.

8. CLOSURE

This Preliminary Foundation Investigation and Design Report was prepared by Mr. Idib (Adeeb) Sadoun, MSc, P.Eng., Project Engineer and by Mr. C.M.P. Nascimento, P.Eng., Project Manager. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, carried out an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

NOTE: Hard copies signed and stamped

Idib (Adeeb) Sadoun, MSc, P.Eng.
Project Engineer

NOTE: Hard copies signed and stamped

C. M. P. Nascimento, P.Eng.
Project Manager

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

CN/BRG:mi

8. CLOSURE

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Yours very truly,

Peto MacCallum Ltd.

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TABLE A

ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
1	15 16 17 18	25.0 – 26.5 26.5 – 28.1 28.1 – 29.6 29.6 – 31.1	85 98 100 83	0 0 0 0	25.0 – 31.1	DOLOSTONE: Buff to grey, fine grained, low to medium strength; unweathered; with occ. layers of grey to black shale (up to 700 mm thick), with black shale partings and occ. seams of gypsum and calcite, very close to close spaced flat bedding layers, smooth planar, tight; very poor quality. (Salina Formation)

RQD: Rock Quality Designation

Originated: MR
Compiled: JFW
Checked: MRA

TABLE 1

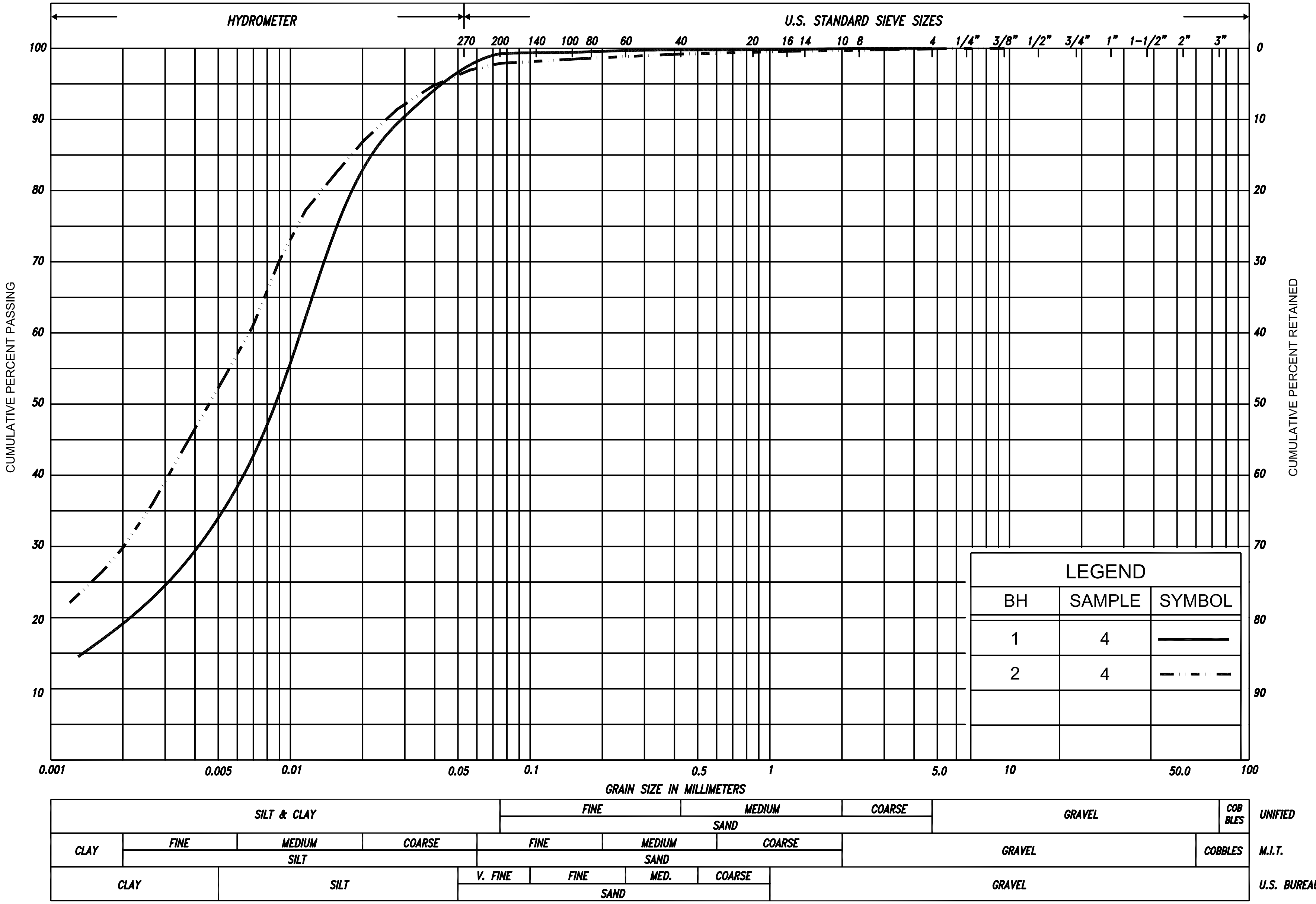
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 501	Construction Specification for Compacting
OPSS 571	Construction Specification for Sodding
OPSS 902	Excavation and Backfilling of Structures
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)
SP 902S01	Excavation and Backfilling of Structures
SP 903S01	Construction Specification for Piling
OPSD-200.010	Earth/Shale Grading – Undivided Rural
OPSD-201.010	Rock Grading-Undivided Rural
OPSD-202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD-3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD-3101.200	Rock Backfill - Walls Abutment
OPSD-3190.100	Retaining Wall and Abutment Wall Drain Detail

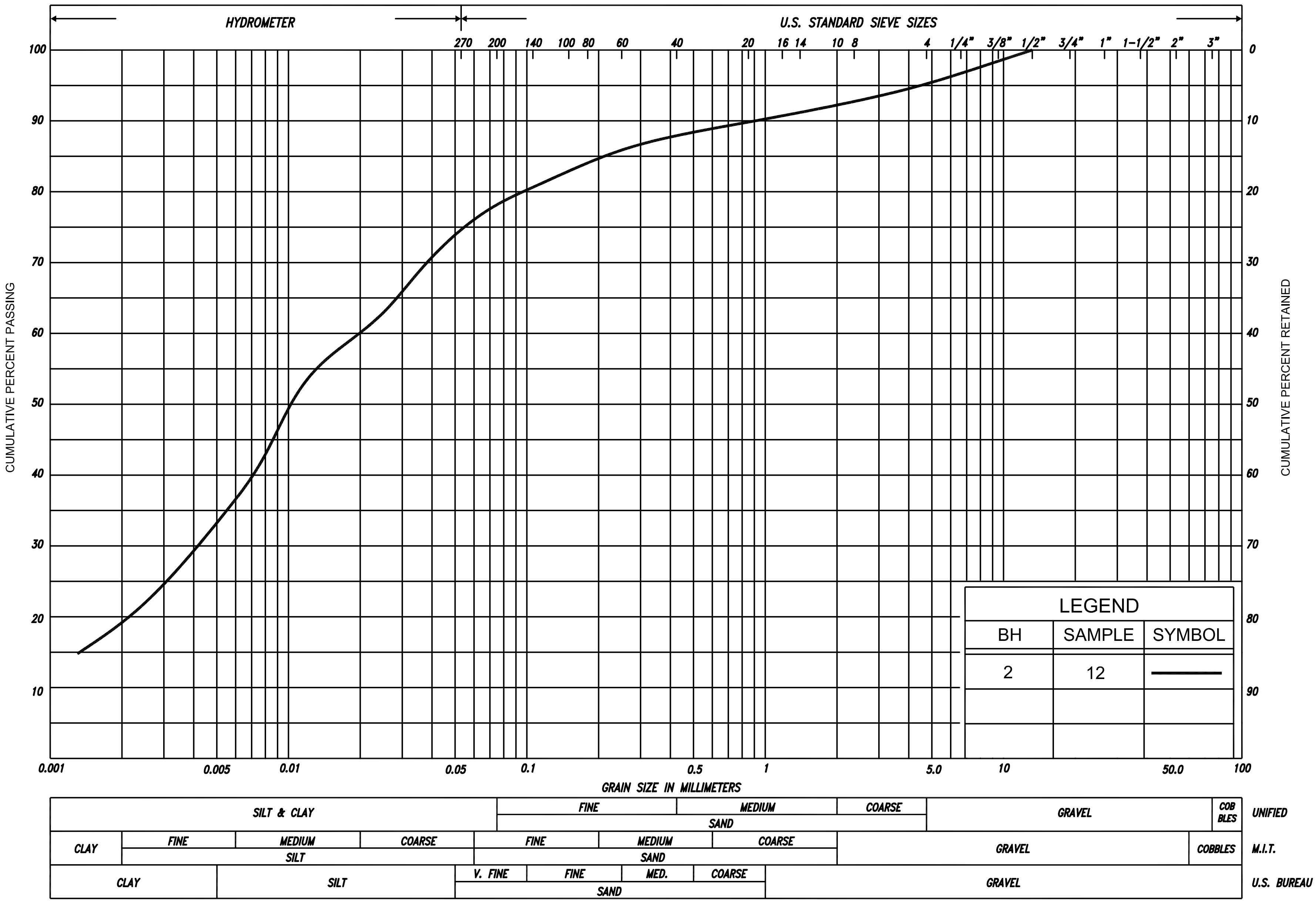
TABLE 2
GRADATION SPECIFICATION FOR SAND FILL IN PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

MTO Sieve Designation	Percentage Passing by Mass
2 mm (#10)	100
600 μm (#30)	80 – 100
425 μm (#40)	40 – 80
250 μm (#60)	5 – 25
150 μm (#100)	0 – 6

Note: From MTO Report S0-96-01, Revision 1 – July, 1996.



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION	FIG No. GS-WL-1
	CLAYEY SILT, trace sand	HWY: 406
		G.W.P. No. 280-99-00



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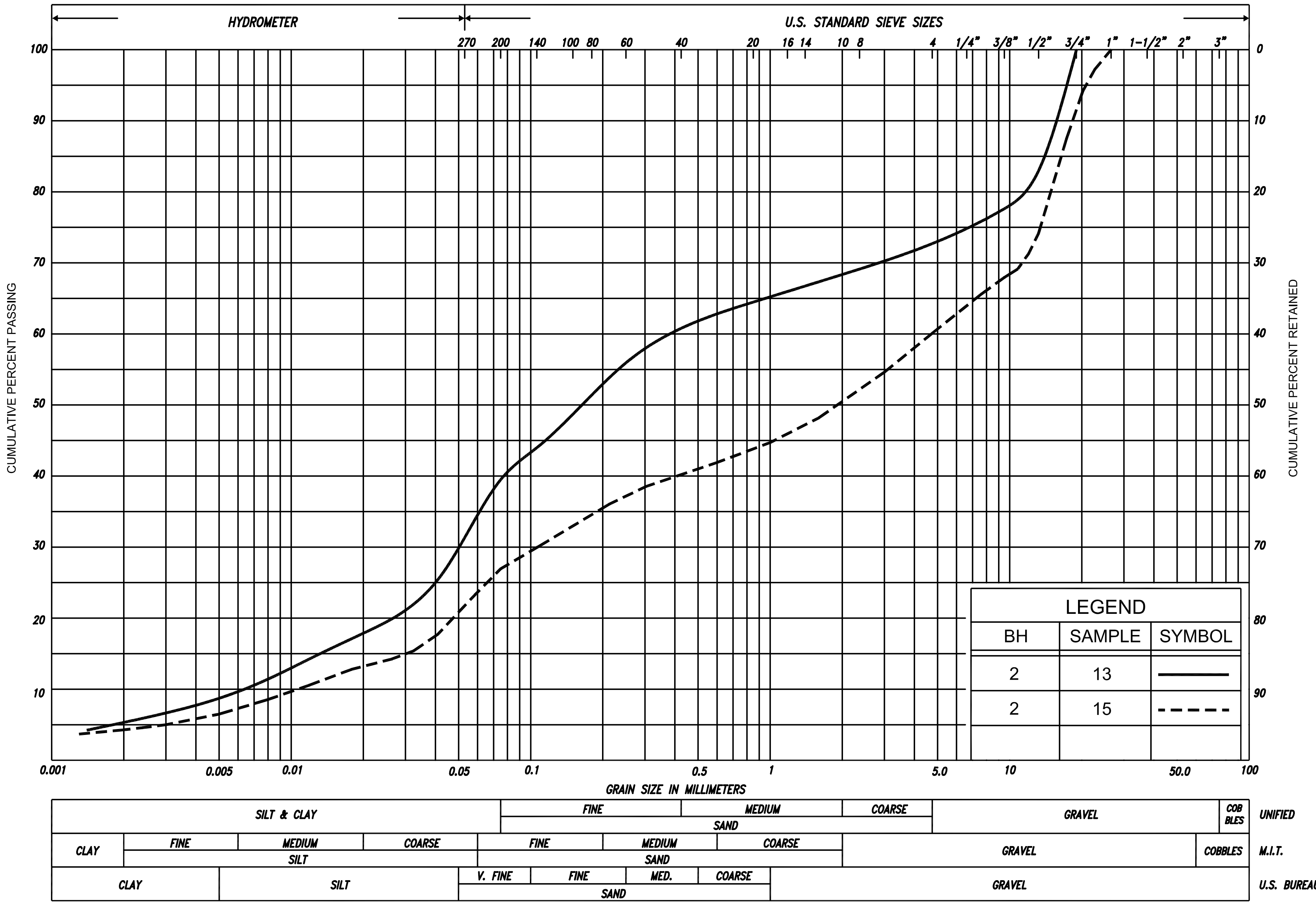
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand, trace gravel (TILL)

FIG No.	GS-WL-2
HWY:	406
G.W.P. No.	280-99-00



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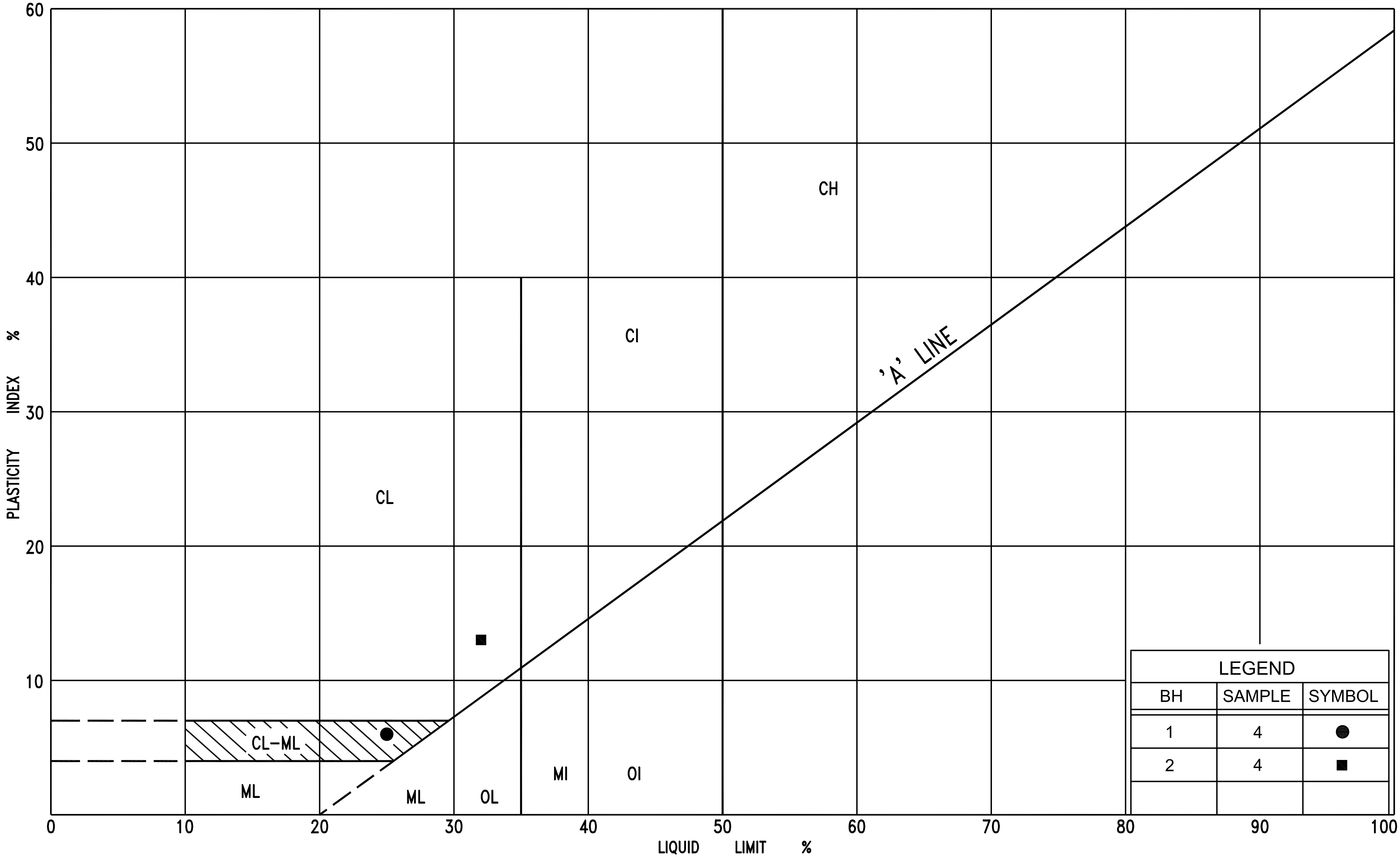
Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT with gravel to gravelly, trace clay (TILL)

FIG No.	GS-WL-3
HWY:	406
G.W.P. No.	280-99-00



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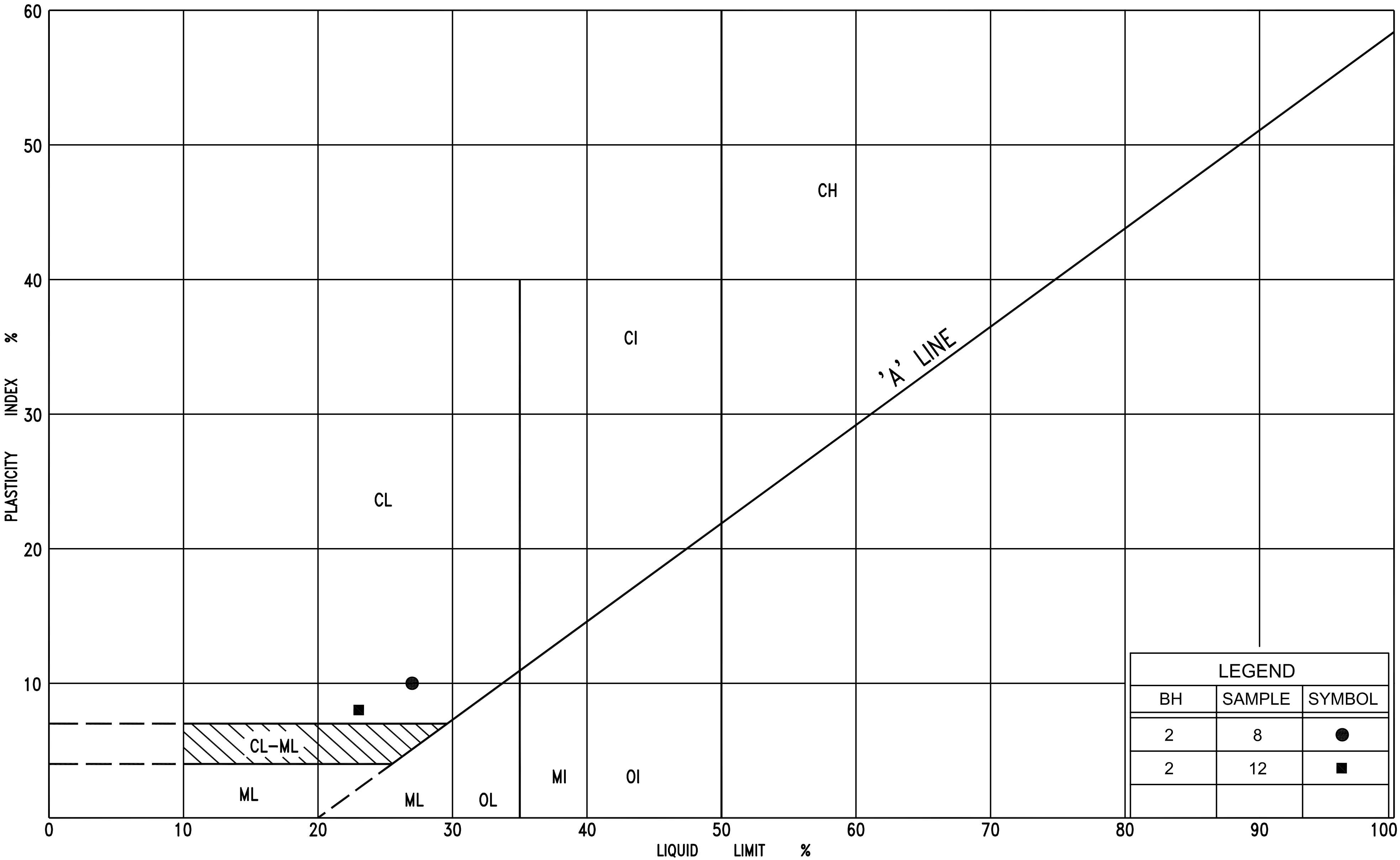
PLASTICITY CHART

CLAYEY SILT, trace sand

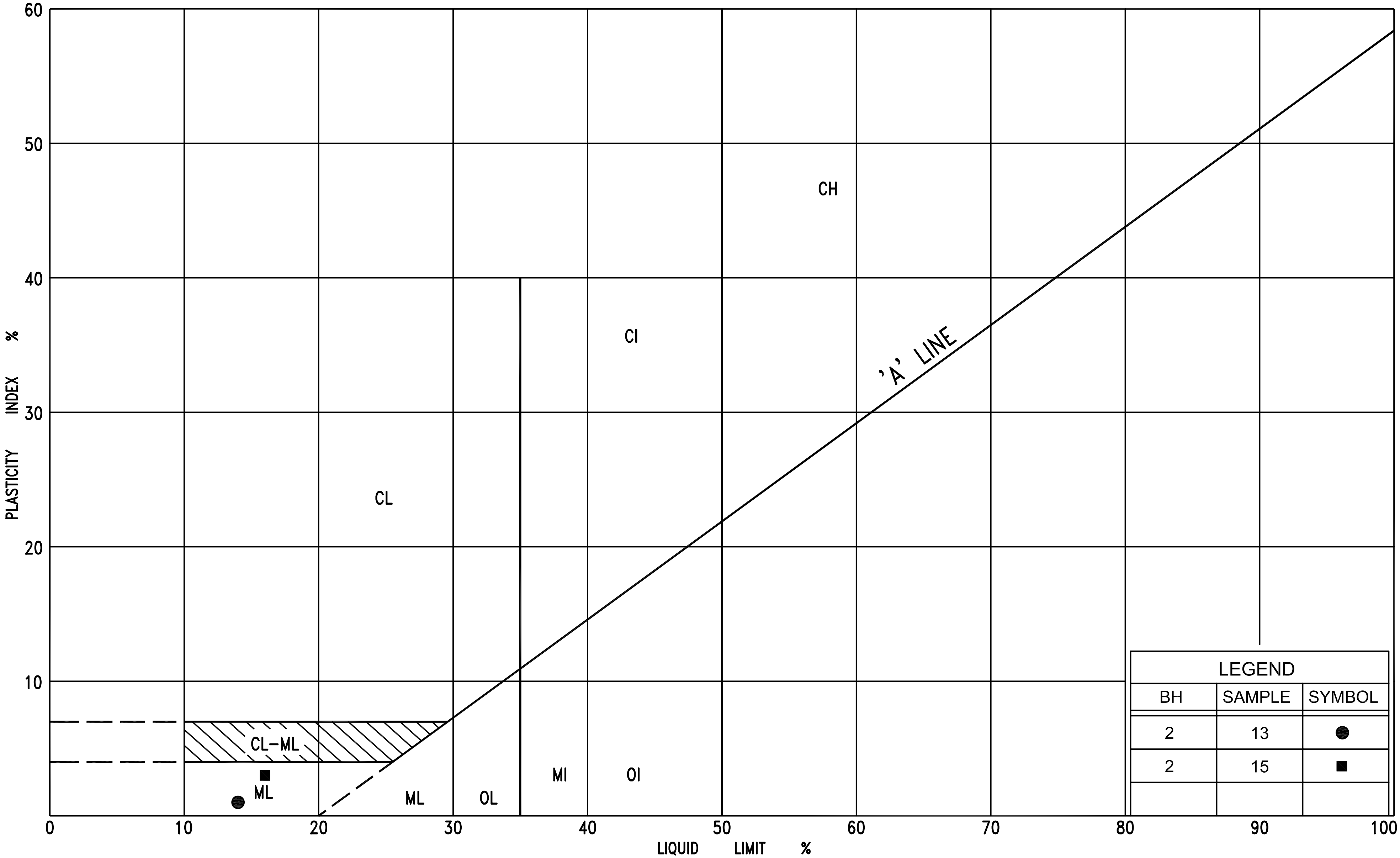
FIG No.	PC-WL-1
HWY:	406
G.W.P. No.	280-99-00



LEGEND

BH	SAMPLE	SYMBOL
2	8	●
2	12	■

Ministry of Transportation Ontario	PLASTICITY CHART	FIG No.	PC-WL-2
	CLAYEY SILT, trace sand, trace gravel (TILL)	HWY:	406
		G.W.P. No.	280-99-00



LEGEND		
BH	SAMPLE	SYMBOL
2	13	●
2	15	■

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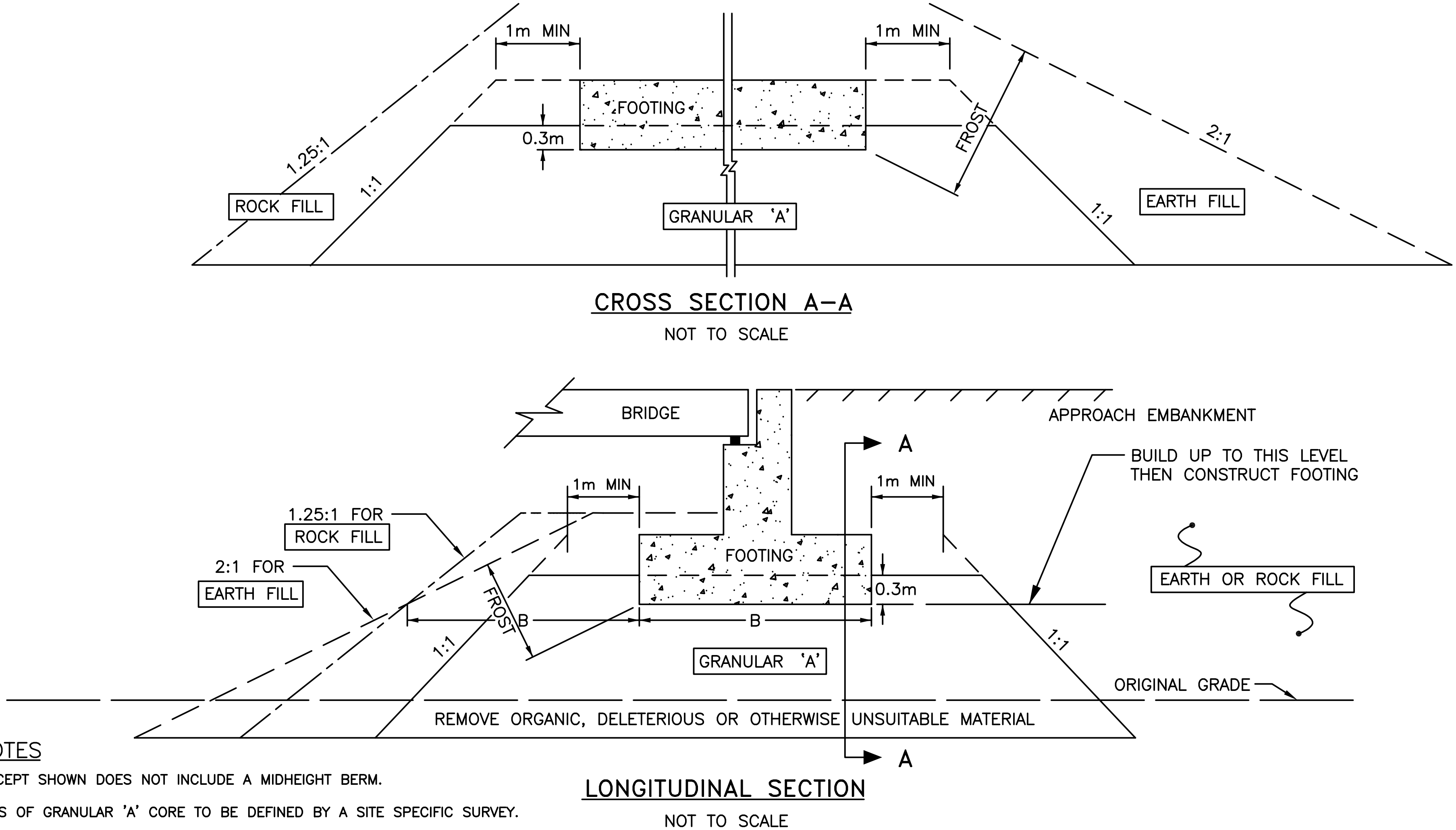
PLASTICITY CHART

SANDY SILT with gravel to gravelly, trace clay (TILL)

FIG No.	PC-WL-3
HWY:	406
G.W.P. No.	280-99-00



NOTES

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE

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

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 280-99-00
LOCATION Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road
ORIGINATED BY M.R.

DIST CR HWY 406
BOREHOLE TYPE C.F.S.S.A. + NW Wash Boring + NXL Rock Coring
COMPILED BY G.D.

DATUM Geodetic
DATE November 07, 2001
CHECKED BY M.R.A.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	Ground Surface							
0.0 / 181.6 / 0.3	Shoulder structure, 25mm of tar and chip over 250mm of granular 'A' crushed limestone (FILL)							
	Clayey silt, trace sand. Very stiff Brown Moist	1	SS	26	181		o	
	Hard	2	SS	32	180		o	
	oxidized stains and thin partings of silt. Very stiff	3	SS	21	179		o	
	lenses of silty clay	4	SS	29			o	0 1 80 19
177.9 / 4.0	Clayey silt trace sand, trace gravel. Stiff Reddish brown Moist (TILL)	5	SS	11	178, 177		o	
		6	SS	10	176, 175		o	
	lenses of silt. Firm to stiff Brown	7	SS	6	174		o	
			FV			+ 2		
	Stiff to hard Reddish brown	8	SS	8	173		o	
			FV		172	2 +		
		9	SS	8	171		o	
			FV			158 +		
		10	SS	15	170		o	
166.9	Cont'd				169, 168, 167			

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; * GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa 40 80 120 160 200; 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.

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 1

METRIC

Co-ords: 4 764 229 N; 327 334 E

G.W.P. 280-99-00	LOCATION Highway 406 at Woodlawn Road		ORIGINATED BY M.R.
DIST CR HWY 406	BOREHOLE TYPE C.F.S.S.A. + NW Wash Boring + NXL Rock Coring		COMPILED BY G.D.
DATUM Geodetic	DATE November 07, 2001		CHECKED BY M.R.A.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
166.9 / 15.0		11	SS	43	166	~20
164.8 / 17.1	Sandy silt, trace clay some gravel, cobbles Dense to very dense Reddish brown Moist to wet 10cm layer of sand (TILL)	12	SS	42	163	~19
		13	SS	52	160	~18
		14	SS	50/5cm	157	~13
156.9 / 25.0	Dolostone Bedrock 23 cm deep void at 25.4m Buff to grey Unweathered Shale layers Low to medium strength Very poor quality	15	RC NXL	REC 85%	156	
		16	RC NXL	REC 98%	155–154	
		17	RC NXL	REC 100%	153	
151.9	Cont'd				152	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); * GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa (40, 80, 120, 160, 200); 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

Elevation scale: 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152

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

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

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 1

METRIC

Co-ords: 4 764 229 N; 327 334 E

G.W.P. 280-99-00 LOCATION Highway 406 at Woodlawn Road ORIGINATED BY M.R.

DIST CR HWY 406 BOREHOLE TYPE C.F.S.S.A. + NW Wash Boring + NXL Rock Coring COMPILED BY G.D.

DATUM Geodetic DATE November 07, 2001 CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (kPa) 40 80 120 160 200 / 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
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	* GROUND WATER CONDITIONS	ELEVATION SCALE				
151.9 / 30.0			18	RC NXL	REC 83%		151				
150.8 / 31.1	End of borehole										

* Borehole charged with drilling water

C.F.S.S.A denotes: Continuous Flight Solid Stem Augers

+7, ×5: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 280-99-00 LOCATION Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road ORIGINATED BY M.R.

DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.

DATUM Geodetic DATE November 15, 2001 CHECKED BY M.R.A.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.1	Ground Surface				
0.0	Clayey silt, trace sand fissured; Very stiff Brown Moist				
		1	SS	23	
	thin layers of silt	2	SS	29	
		3	SS	21	
		4	SS	20	0 2 68 30
179.4 / 3.7	Silt, trace gravel layers of grey silty clay; Compact Reddish brown Moist	5	SS	20	
		6	SS	27	
177.6 / 5.5	Clayey silt, trace sand specks of shale; Stiff to very stiff Reddish brown Moist (TILL)	7	SS	14	
	trace gravel	8	SS	8	
	faintly layered				
		9	SS	8	
	thin silt layers	10	SS	9	
	layers of silt, trace gravel up to 20mm diameter	11	SS	13	
	some sand				
168.1	Cont'd				

Elevation scale: 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169

Ground water conditions: ▼*

DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa: 40 80 120 160 200
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

+7, ×5: Numbers refer to Sensitivity   20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 280-99-00 LOCATION Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road ORIGINATED BY M.R.

DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.

DATUM Geodetic DATE November 15, 2001 CHECKED BY M.R.A.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
168.1 / 15.0					168	
		12	SS	19		5 17 59 19
166.1 / 17.0	Sandy silt with gravel, trace clay. Very dense, Reddish brown, Moist (TILL)				167, 166, 165	
		13	SS	61	164	27 33 35 5
163.1 / 20.0	Clayey silt some sand, trace gravel. Very stiff to hard, Reddish brown, Moist (TILL)				163, 162	
		14	SS	30	161	
160.1 / 23.0	Sandy gravelly silt trace clay, cobbles. Very dense, Reddish brown, Moist (TILL)				160, 159	
		15	SS	67/10cm	158	40 32 24 4
157.2 / 25.9	Bedrock - Possible Dolostone shale layers. Augered with difficulty. Shattered / Very weak possible void was encountered at 26.8m				157, 156, 155, 154	
153.1	Cont'd					

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa (40 80 120 160 200); 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

$+^7$, $\times^5$: 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

Co-ords: 4 764 164 N; 327 311 E

G.W.P. 280-99-00	LOCATION	Highway 406 at Woodlawn Road	ORIGINATED BY	M.R.
DIST CR	HWY 406	BOREHOLE TYPE	Continuous Flight Solid Stem Augers	COMPILED BY	G.D.
DATUM Geodetic	DATE	November 15, 2001	CHECKED BY	M.R.A.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa (40 80 120 160 200) / SHEAR STRENGTH kPa (20 40 60 80 100)	WATER CONTENT (%) w_P w w_L (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
153.1 / 30.0							153				
152.6 / 30.5	End of borehole										

SHEAR STRENGTH legend: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

PLASTIC LIMIT; NATURAL MOISTURE CONTENT; LIQUID LIMIT

* 2001 11 15

▼ Water level measured after drilling

■ Penetrometer Test

ON_MOT VER3 08TF005_WL.GPJ ON_MOT.GDT 11/5/2008 11:55:38 AM

+7, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 280-99-00
LOCATION Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road
ORIGINATED BY M.R.
DIST CR HWY 406
BOREHOLE TYPE C.F.S.S.A.+ NW Wash Boring + Rotary Diamond Drilling
COMPILED BY G.D.
DATUM Geodetic
DATE November 07, 2001
CHECKED BY M.R.A.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.9	Ground Surface							
0.0	Shoulder structure, 25mm of tar and chip over 250mm of granular 'A' crushed limestone (FILL)							
0.3	Clayey silt, trace sand Very stiff Brown Moist	1	SS	26	181		o	
	Hard	2	SS	32	180		o	
	oxidized stains and thin partings of silt Very stiff	3	SS	21	179		o	
	lenses of silty clay	4	SS	29			o	0 1 80 19
177.9 / 4.0	Clayey silt trace sand, trace gravel Stiff Reddish brown Moist (TILL)	5	SS	11	178, 177		o	
		6	SS	10	176, 175		o	
	lenses of silt Firm to stiff Brown	7	SS	6	174		o	
			FV			+ 2		
	Stiff to hard Reddish brown	8	SS	8	173		o	
			FV		172	+ 2		
		9	SS	8	171		o	
			FV			+ 158		
		10	SS	15	170		o	
166.9	Cont'd				169, 168, 167			

DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa: 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

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 280-99-00	LOCATION Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road	ORIGINATED BY	M.R.
DIST CR HWY 406	BOREHOLE TYPE C.F.S.S.A.+ NW Wash Boring + Rotary Diamond Drilling	COMPILED BY	G.D.
DATUM Geodetic	DATE November 07, 2001	CHECKED BY	M.R.A.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa (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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS
166.9 15.0		11	SS	43	
164.8 17.1	Sandy silt, trace clay some gravel, cobbles Dense to very dense · Reddish brown · Moist to wet 10cm layer of sand (TILL)	12	SS	42	
		13	SS	52	
		14	SS	50/5cm	
156.9 25.0	Dolostone Bedrock 23 cm deep void at 25.4m Buff to grey Unweathered Shale layers Low to medium strength Very poor quality	15	RC NQ	REC 85%	RQD 0%
		16	RC NQ	REC 98%	RQD 0%
		17	RC NQ	REC 100%	RQD 0%
151.9	Cont'd				

Elevation scale: 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152

+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 1

METRIC

G.W.P. 280-99-00 LOCATION Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road ORIGINATED BY M.R.

DIST CR HWY 406 BOREHOLE TYPE C.F.S.S.A.+ NW Wash Boring + Rotary Diamond Drilling COMPILED BY G.D.

DATUM Geodetic DATE November 07, 2001 CHECKED BY M.R.A.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
151.9											
30.0			18	RC NQ	REC 83%		151				RQD 0%
150.8 / 31.1	End of borehole Sample 14: Sampler bouncing * Borehole charged with drilling water C.F.S.S.A denotes: Continuous Flight Solid Stem Augers										

SOIL PROFILE / SAMPLES

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

WATER CONTENT (%): 20 40 60

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

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

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No 2

METRIC

G.W.P. 280-99-00	LOCATION Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road		ORIGINATED BY M.R.
DIST CR HWY 406	BOREHOLE TYPE Continuous Flight Solid Stem Augers		COMPILED BY G.D.
DATUM Geodetic	DATE November 15, 2001		CHECKED BY M.R.A.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
183.1	Ground Surface
0.0	Clayey silt, trace sand fissured Very stiff Brown Moist thin layers of silt
179.4 / 3.7	Silt, trace gravel layers of grey silty clay Compact Reddish brown Moist
177.6 / 5.5	Clayey silt, trace sand specks of shale Stiff to very stiff Reddish brown Moist (TILL) trace gravel faintly layered thin silt layers layers of silt, trace gravel up to 20mm diameter some sand
168.1	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	23	
2	SS	29	
3	SS	21	
4	SS	20	0 2 68 30
5	SS	20	
6	SS	27	
7	SS	14	
8	SS	8	
9	SS	8	
10	SS	9	
11	SS	13	

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (183 to 169); 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.

150

+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

Co-ords: 4 764 164 N; 327 311 E

G.W.P. 280-99-00	LOCATION Highway 406 at Woodlawn Road		ORIGINATED BY M.R.
DIST CR HWY 406	BOREHOLE TYPE Continuous Flight Solid Stem Augers		COMPILED BY G.D.
DATUM Geodetic	DATE November 15, 2001		CHECKED BY M.R.A.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
168.1 / 15.0	
166.1 / 17.0	Sandy silt with gravel, trace clay Very dense, Reddish brown, Moist (TILL)
163.1 / 20.0	Clayey silt some sand, trace gravel Very stiff, Reddish brown, Moist (TILL)
160.1 / 23.0	Gravelly sandy silt trace clay, cobbles Very dense, Reddish brown, Moist (TILL)
157.2 / 25.9	Bedrock - Possible Dolostone shale layers Augered with difficulty Shattered / Very weak possible void was encountered at 26.8m
153.1	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
12	SS	19	5	17	59	19
13	SS	61	27	33	35	5
14	SS	30				
15	SS	67/10cm	40	32	24	4

GROUND WATER CONDITIONS

ELEVATION SCALE: 168, 167, 166, 165, 164, 163, 162, 161, 160, 159, 158, 157, 156, 155, 154

DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa: 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/m^3

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

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

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 280-99-00 LOCATION Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road ORIGINATED BY M.R.

DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.

DATUM Geodetic DATE November 15, 2001 CHECKED BY M.R.A.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT kPa (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
153.1											
30.0							153				
152.6 / 30.5	End of borehole										

Sample 15: Sampler bouncing

* 2001 11 15

▼ Water level measured after drilling

■ Penetrometer Test

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

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

METRIC

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

CONT No

GWP No 280-99-00

WOODLAWN ROAD UNDERPASS

HIGHWAY 406

BOREHOLE LOCATIONS

SHEET

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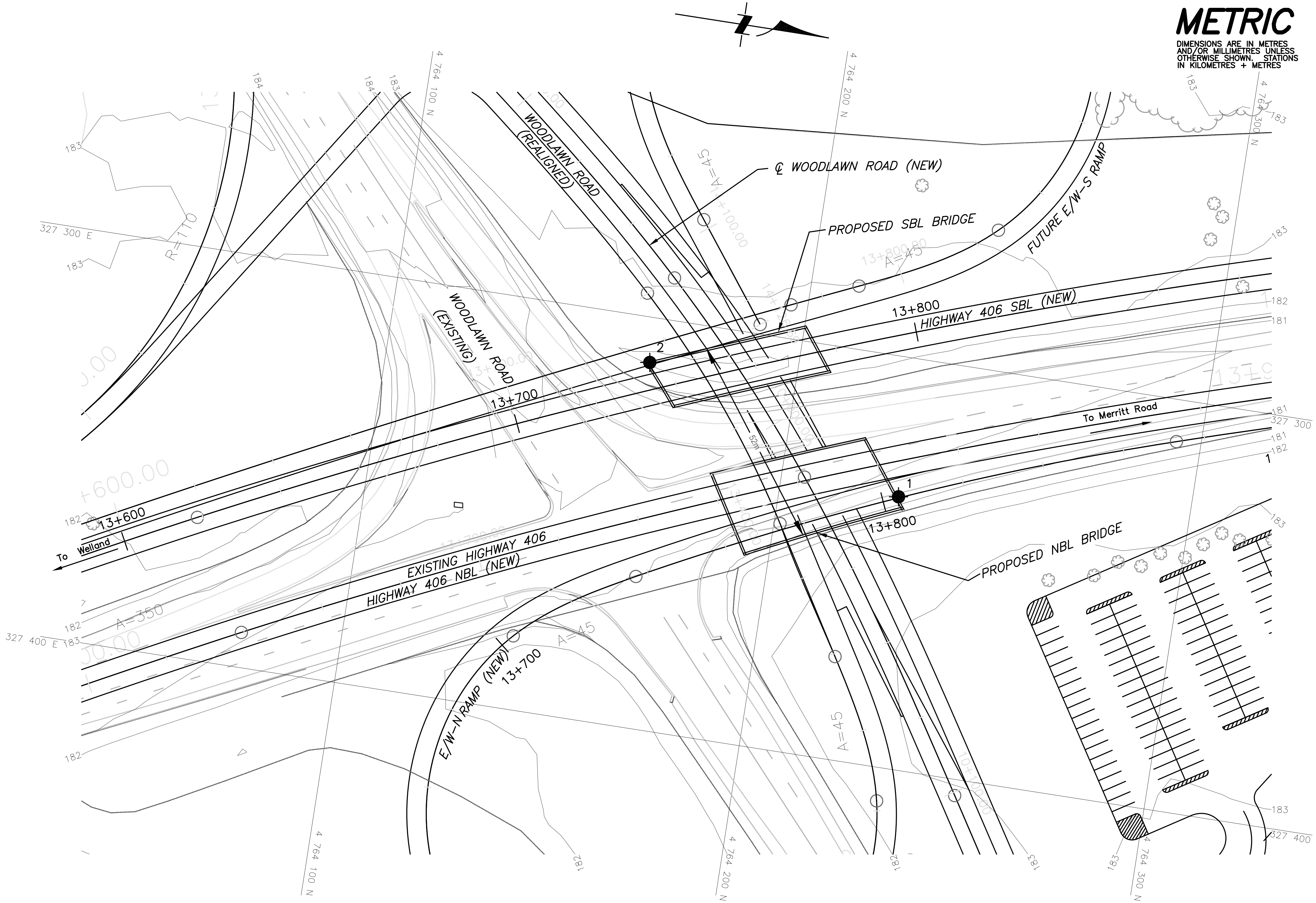
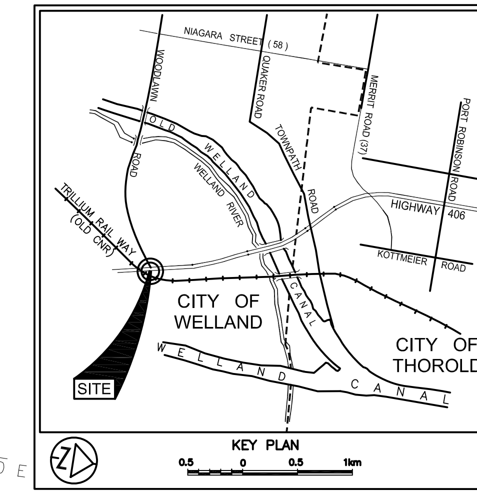
CONSULTING ENGINEERS



PLAN

SCALE



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 Nov 2001
	Head
	ARTESIAN WATER Encountered
	PIEZOMETER

BH No	ELEVATION	CO-ORDINATES NORTHINGS	EASTINGS
1	181.9	4 764 229	327 334
2	183.1	4 764 164	327 311

– 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

Geocres No. 30M03-236

HWY No 406				DIST CENTRAL	
SUBM'D NSB	CHECKED NSB	DATE DEC. 05, 2008		SITE --	
DRAWN NA	CHECKED CN	APPROVED BRG		DWG WL-1	

NOTE:

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

REF No. MRC DRAWINGS: OLD-BASE MAP-ONE COLOUR.dwg
PREFERED-DPTION-22.5M-CL.dwg
RECEIVED ON SEPTEMBER 25, 2008