

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**MERRITT ROAD UNDERPASS
HIGHWAY 406 FOUR-LANING
GWP 280-99-00 (6)
CITY OF THOROLD, ONTARIO**

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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for

Merritt Road Underpass
Highway 406 Four-Laning
GWP 280-99-00
City of Thorold, Ontario

1. INTRODUCTION

This report summarizes the results of the preliminary foundation investigation carried out for the proposed Merritt Road Underpass at the 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 Merritt Road traffic over the proposed Highway 406 northbound and southbound lanes at approximate Sta. 15+845 and Sta. 15+828, respectively (new Highway 406 chainage).

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 report follows the terms of reference (TOR) outlined in the original request for proposal (April 19, 2000). The foundation drawing did not include the preparation of soil sections and this is considered to be adequate for preliminary design purposes.

2. SITE DESCRIPTION AND GEOLOGY

The contemplated structure is proposed about 35 m north of the existing Merritt Road and Highway 406 at-grade crossing. The site is about 900 m south of the existing Port Robinson Road intersection at Highway 406.

Land use in the vicinity of the site comprises the existing Merritt Road and Highway 406 at-grade intersection. The proposed Merritt Road Underpass will run roughly east to west. The local



topography of the structure site is relatively flat. About 6 m high stockpiles were present at the proposed east abutment location and 20 m west of the proposed west abutment location. The areas of the stockpiles are approximately outlined on the attached Drawing MR-1. The eastern stockpile is about 30 m wide and 120 m long while the western stockpile is about 80 m wide and 80 to 100 m long. The ground cover beyond the paved areas of the highway comprises grasses, bushes and stands of trees.

The site is located in the Haldimand Clay Plain physiographic region. The topography is gently flat and undulating. 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 during the period November 1 to 5, 2001. Two sampled boreholes were put down at the site. The boreholes were drilled to refusal at depths of 36.7 and 37.4 m at the locations shown on Drawing MR-1. Borehole 2 located at the proposed west abutment was extended by coring 3.0 m into bedrock to a total depth of 40.4 m.

The locations of and ground surface elevations at the boreholes were established in the field by Peto MacCallum Ltd. The following benchmark (BM) was used for vertical reference:

BENCHMARK	ELEVATION (*)
Top of south east bolt of light standard on south west quadrant of Merritt Road and Highway 406 intersection at Station 16+226.	180.447

(*) Elevations are expressed in meters and referred to the geodetic datum

The boreholes were advanced using continuous flight solid and hollow stem augers and NW wash boring, 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 samples of the soils were recovered in the boreholes at frequent 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 2, casing was extended to the bedrock surface and an approximate 3.0 m length of rock core was recovered using NXL rock coring equipment. The PML geologist examined 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 to the ground surface.

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 (26)
- Grain Size analyses (4)
- Atterberg Limits (6)

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-MR-1 to GS-MR-3. The Atterberg limits results are presented on Figures PC-MR-1 to PC-MR-3.



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 Merritt Road underpass are presented on the attached Foundation Drawing MR-1.

Below a local 0.3 m thick pavement fill unit on the west and 1.4 m thick sandy surficial deposit on the east side the subsurface stratigraphy revealed in the boreholes generally included of a 8.7 to 9.8 m thick deposit of very stiff to firm silty clay overlying a 1.5 to 3.6 m thick loose to very loose silt layer underlain by a 9.9 and 14.3 m thick typically firm clayey silt deposit. These deposits overlay a layer 11.5 to 13.1 m thick compact/dense and very dense silt till. Dolostone bedrock was contacted below the native soils at depths of 36.7 and 37.4 m. The strata encountered are summarized below.

4.2 Fill

A 90 mm thick layer of asphaltic concrete over 190 mm of granular "A" crushed limestone fill was encountered surficially in borehole 2 (west abutment) drilled on existing Highway 406 ramp shoulder. The unit extended to an approximate depth of 0.3 m (elevation 180.1).

4.3 Sandy Silt

A 1.4 m thick deposit of cohesionless sandy silt was found surficially in borehole 1 (east abutment) extending to 1.4 m depth, elevation 179.1. The deposit exhibits loose relative density with one N value of 7. The laboratory moisture content of the sandy silt was 16%.



4.4 Silty Clay

A cohesive deposit of silty clay was present below the sandy silt in borehole 1 (east abutment) at a depth of 1.4 m (elevation 179.1) and below the fill in borehole 2 (west abutment) at a depth of 0.3 m (elevation 180.1). The stratum was 8.7 and 9.8 m thick extending to the underlying cohesionless silt at a uniform depth of 10.1 m (elevations 170.4 and 170.3) in both boreholes.

The silty clay deposit typically exhibited very stiff to firm consistency with some hard local layers. The field vane tests carried out in the silty clay indicated that the undisturbed shear strength values ranged from 48 to 155 kPa (soil sensitivity of 2 and 3). Penetrometer tests results on two samples were 25 and 110 kPa. N values ranged from 3 to 34, typically decreasing with depth.

The grain size distribution chart of a representative sample of the silty clay is shown on Figure GS-MR-1 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-MR-1. The liquid limit of the silty clay was 40 and the plastic limit 20, giving the plasticity index value of 20 indicating a deposit of medium plasticity. The water content of the silty clay varied from 20 to 38%.

4.5 Silt

A 3.6 and 1.5 m thick cohesionless silt deposit containing layers of clayey silt was encountered beneath the silty clay unit at a uniform depth of 10.1 m (elevations 170.4 and 170.3) in both boreholes. The layer extended to the underlying cohesive clayey silt deposit at depths of 13.7 and 11.6 m (elevations 166.8 and 168.8). The silt was loose to very loose with N values ranging from 3 to 6. The moisture content of the layer ranged from 19 to 24%.

4.6 Clayey Silt

Beneath the silt deposit at depths of 13.7 and 11.6 m (elevations 166.8 and 168.8), a 9.9 and 14.3 m thick deposit of cohesive clayey silt was encountered in boreholes 1 and 2, respectively. The unit extended to the underlying silt till at depths of 23.6 and 25.9 m (elevations 156.9 and 154.5).



The clayey silt deposit was typically firm with some local hard layers in borehole 2 below a depth of 22.9 m. N values ranged from 6 to 34, typically in the range of 6 to 10.

The grain size distribution chart of a representative sample of the clayey silt is shown on Figure GS-MR-2 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-MR-2. The liquid limits of the clayey silt were 22 and 32 and the plastic limits 18 and 19, giving the plasticity index values of 4 and 13 indicating a low plasticity deposit. The water content of the clayey silt varied from 19 to 21%.

4.7 Silt Till

A deposit of glacial till comprising dense/compact to very dense silt till was encountered in both boreholes below the clayey silt deposit at depths of 23.6 and 25.9 m (elevations 156.9 and 154.5). The N values in the till typically ranged from 23 to 77. Two N values of 50 and 100 were obtained for 5 and 10 cm sampler penetration in borehole 2.

The composition of the till was variable and included a 1.1 m thick gravel till in borehole 2 (west abutment) at depths between 27.3 and 28.4 m (elevation 153.1 to 152.0). Cobbles and boulders are anticipated in this deposit although not encountered in the boreholes.

The thickness of till deposit was 13.1 and 11.5 m extending to underlying bedrock at depths of 36.7 and 37.4 m (elevations 143.8 and 143.0) in boreholes 1 and 2, respectively.

The grain size distribution charts of representative samples of the silt till are shown on Figure GS-MR-3 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-MR-3. The liquid limit of the silt till was 16 and the plastic limit 12, giving the plasticity index value of 4. The gravel till sample was non-plastic according to Atterberg determination and manual examination. The water content of the silt till was 7%.

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.



LOCATION	BOREHOLE No.	DEPTH (m)	ELEVATION	ROCK CORE LENGTH (m) (*)
East Abutment	1	36.7	143.8	-
West Abutment	2	37.4	143.0	3.0

(*) NXL diamond rock cores obtained.

The core recovery was 93 and 100% for both core samples. The rock exhibited a low to medium strength and was found to be unweathered. The rock typically is of poor quality (RQD values were 32 and 35%). Loss of drilling water was not experienced during drilling. The detailed rock core descriptions are provided on Table A.

4.9 Groundwater

Upon completion of drilling groundwater was measured in borehole 1 (east abutment) at a depth of 12.3 m (elevation 168.2). The groundwater table was not determined upon completion of drilling in borehole 2 (west abutment) because the borehole was charged with water for rock coring purposes.

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

5. MISCELLANEOUS

The field work was carried out in 2001 under the supervision of Mr. M. Rapsey and direction of Mr. P. Cullen, B.Eng. The drilling equipment was supplied by Elite Drilling and Malone's Soil Sampling.

This Preliminary Foundation Investigation Report was prepared by Mr. C.M.P. Nascimento, P.Eng., with the assistance of Ms. N.S. Balakumaran, E.I.T., and independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for

Merritt Road Underpass
Highway 406 Four-Laning
GWP 280-99-00
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 Merritt 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.

Based on the preliminary drawing, the proposed new underpass will carry the realigned Merritt Road traffic over the Highway 406 northbound and southbound lanes at approximate Sta. 15+845 and Sta. 15+828, respectively (new Highway 406 chainage). The proposed underpass structure will be a two span structure with a total length of approximately 70 m between abutments. It is assumed that approach embankments will be about 8 m high at the abutments.

In summary, the inferred subsurface stratigraphy revealed in the boreholes consisted of a surficial deposit of hard/very stiff, becoming firm with depth cohesive silty clay overlying a loose to very loose cohesionless silt layer underlain by a firm cohesive clayey silt deposit overlying a compact/dense to very dense cohesionless silt till about 11.5 to 13.1 m thick. Low to medium strength Dolostone bedrock was contacted below the native soils at depths of 36.7 and 37.4 m (elevation 143.8 and 143.0) at the east and west abutments, respectively.

Use of conventional procedures to construct the underpass foundations is expected to be feasible. The pile lengths will vary in view of the variable relative density of the compact/dense to very dense and over 11.5 m thick silt till stratum which mantles the bedrock. It is likely that some of the piles will be driven through the glacial till and to the bedrock at the east abutment and other piles will find refusal in the denser glacial till at the west abutment. The geotechnical resistance of the deep foundations should be designed for bearing on the glacial till layer at the west abutment



and on the bedrock at the east abutment for preliminary design purposes. The piles should be provided with rock points due to potentially heavy driving into the till deposit which likely contains cobbles and boulders.

Earth stockpiles about 6 m high were placed on the alignments of the east and west approach embankments prior to 2001, as shown on the attached Drawing MR-1. The western stockpile is about 20 m away from the proposed west abutment due to the presence of the N-W exit ramp from Highway 406. The stockpiles have contributed to preload the local cohesive and compressible soils and reduce the estimated 60 mm of consolidation settlement of the clayey soils that would otherwise be expected from the 8 m high approach embankment fill loading. Staged construction at the west approach embankment may be warranted. Further comments including surcharging are included in this report.

The recommendations in this report are preliminary and based on PML's interpretation of the factual information obtained from a limited number of boreholes. 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 dense to very dense silt till or bedrock is considered feasible. Lightly loaded footings placed on the native soils or on engineered fill may be used for semi-integral or conventional abutment design.

Drilled caissons bearing on the glacial till or on the bedrock to support the underpass structure are not considered to be practical due to the presence of cobbles and boulders in the till, loose to very loose cohesionless silt as well as a relatively high groundwater table.

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.

Drilled cast-in-place concrete caissons are not considered to be practical at this site in view of installation difficulties. These would be due to a high groundwater table above the founding levels and potential for cobbles and boulders in the glacial till stratum.

The preliminary pile foundation design recommendations for conventional and semi-integral abutments and for the pier 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 bedrock and very dense silt till, respectively. Encountered bedrock level for the east abutment and estimated reference level of silt till for the west abutment are provided in the following table:

FOUNDATION ELEMENT	BOREHOLE No.	FOUNDING DEPTH (m)	FOUNDING ELEVATION	ESTIMATED PILE TIP ELEVATION
East Abutment	1	36.7	143.8	143.8
West Abutment	2	31.9	148.5	146.5

Note: For preliminary design purposes, the founding levels for driven piles at the proposed pier location should be extrapolated by averaging the founding levels at the abutments. These levels should be confirmed during Detail Design.

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 on till deposit should be allowed.

The piles will have to be driven through native soils containing compressible clayey soils at the abutment locations. The east abutment site has been preloaded as indicated previously and the additional 1.5 m of fill for the approach embankment will only cause minor settlements less than 20 mm. The existing grade at the west abutment will be raised about 7 m above the existing N-W ramp level. Consequently, the development of negative skin friction on the piles should be considered to the axial resistance at ultimate limit states (ULS) for the west abutment piles only.

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

Based on very dense silt till at the pile tips at the west abutment and low to 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 37.4 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 (1,900 kN) is larger than the ULS factored capacity.

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 as recommended in Section 6.4 of this report.



The piles will set on bedrock at the east abutment and should be equipped with rock points (OPSD-3000.201 and SP 903S01). The Titus H Bearing Pile Rock Injector Model (Titus Point) should be used to minimize the potential for damage when driving through dense till containing cobbles and boulders.

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 very dense silt 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 clayey 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:

	NATIVE SILTY CLAY /CLAYEY SILT	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 contraflexure, the coefficient of horizontal subgrade reaction, k_s (MN/m³) should be computed using the following equation:

Cohesionless Soils (Terzaghi, 1955)

$$\begin{aligned} k_s &= n_h z/b \\ \text{where } n_h &= \text{coefficient related to soil density} \\ &= 10.0 \text{ MN/m}^3 \text{ for granular backfill} \\ z &= \text{depth, m} \\ b &= \text{pile width, m} \end{aligned}$$

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 clay/clayey silt units should be taken as 28,000 kN/m³ for preliminary purposes.

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.

Spread footings should be constructed on the native soils comprising typically hard to very stiff silty clay at the proposed elevation 178.5 (2.0 m depth). The recommended bearing resistance for minimum 2.0 m wide footings constructed on the native soils is as follows:

Factored Geotechnical Resistance at ULS, kPa	300
Geotechnical Resistance at SLS, kPa	200

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.

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 Ontario Provincial Standards Specifications (OPSS) Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

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

The thickness of structural fill pad of 2.5 m 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.

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.

6.3 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/m³
 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 \emptyset = angle of internal friction of retained soil (35° for Granular A or Granular B Type II or Type III)
 δ = angle of friction between the soil and wall (23.5° 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, \emptyset (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) and/or weep holes (OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. 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 recommended previously for spread footings constructed on the structural fill should be employed for design of the RSS wall.

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.

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 Merritt Road Underpass. In view of the very stiff to firm cohesive soils underlain by cohesionless soils followed by dense to very dense silt till at the east and west abutments, the approach embankments are likely to be founded on very stiff to stiff cohesive clayey soils. As noted previously, 6 m high fills were stockpiled on the future approaches to Merritt Road underpass, as shown on the site plan provided by MRC in 2001. The fill over the east approach covered the whole extent of the approach embankment, however, the west approach embankment site was about 20 m short of the abutment because this area is occupied by the existing N-W ramp. Subsurface investigation should be carried out at these locations for detail design to assess the condition of the existing fill within the approaches.

The approach embankments should be designed and constructed in accordance with OPSSD-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.

It is inferred that the east approach embankment area has been preloaded by the existing fill stockpile and this area will undergo only minor settlements of 10 to 15 mm resulting from the additional 1.5 to 2.0 m of embankment fill height.



Based on limited laboratory test data, it is estimated that some 60 mm of consolidation settlement of the clayey subgrade soils will occur at the west approach embankment. This estimated settlement of cohesive soils is likely to take up to 19 months to occur to 80% completion. It is recommended that the west approach embankment fill should be placed and a 2 m high surcharge be applied for period of at least 12 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 purpose.

It is considered feasible to backfill the structure using granular materials or rock backfill. 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 hard/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 12.3 m (elevation 168.2) at the east abutment. 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 purposes only and for feasibility studies.

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

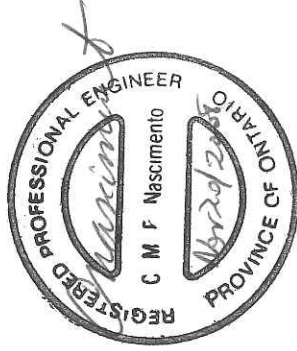
1. Carry out the complete scope of detailed field investigations at the structure site. Incorporate the data from the previously drilled boreholes included in this report for the Detail Design.
2. Determine/evaluate the slope of the bedrock founding surface to evaluate the need for steel pile rock points and the reduction of axial bearing resistance related to the extent of cobbles and boulders in the very dense silt till mantling the dolostone bedrock.
3. Evaluate by means of additional investigations the adequacy of the existing fills placed on the alignment of the approach embankments to be used as part of the road platform.
4. Additional investigation with enhanced laboratory testing should be carried out within the footprint of the approaches for settlement and stability analysis.

8. CLOSURE

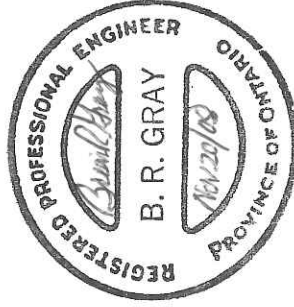
This Preliminary Foundation Design Report was prepared by Mr. C.M.P. Nascimento, P.Eng., with the assistance of Ms. N.S. Balakumaran, E.I.T., and independently reviewed by Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.



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



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact
CN/BRG:nb-mi



TABLE A
 ROCK CORE DESCRIPTION

CORE RECOVERY						CORE DESCRIPTION	
BOREHOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION	
2 (West abutment)	19	37.4 – 38.9	93	32	37.4 – 40.4	DOLOSTONE: Buff to grey, fine grained, low to medium strength; unweathered; with occ. irregular black shale partings and occ. seams of gypsum and calcite, close spaced flat bedding layers, rough planar, tight; poor quality. (Salina Formation)	
	20	38.9 – 40.4	100	35			

RQD: Rock Quality Designation

Originated: MR

Compiled: JFW

Checked: NSB/CN



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-3000.201	Oslo Points for Foundation, Piles, Steel HP310
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 μ m (#30)	80 – 100
425 μ m (#40)	40 – 80
250 μ m (#60)	5 – 25
150 μ m (#100)	0 – 6

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



Ontario
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand

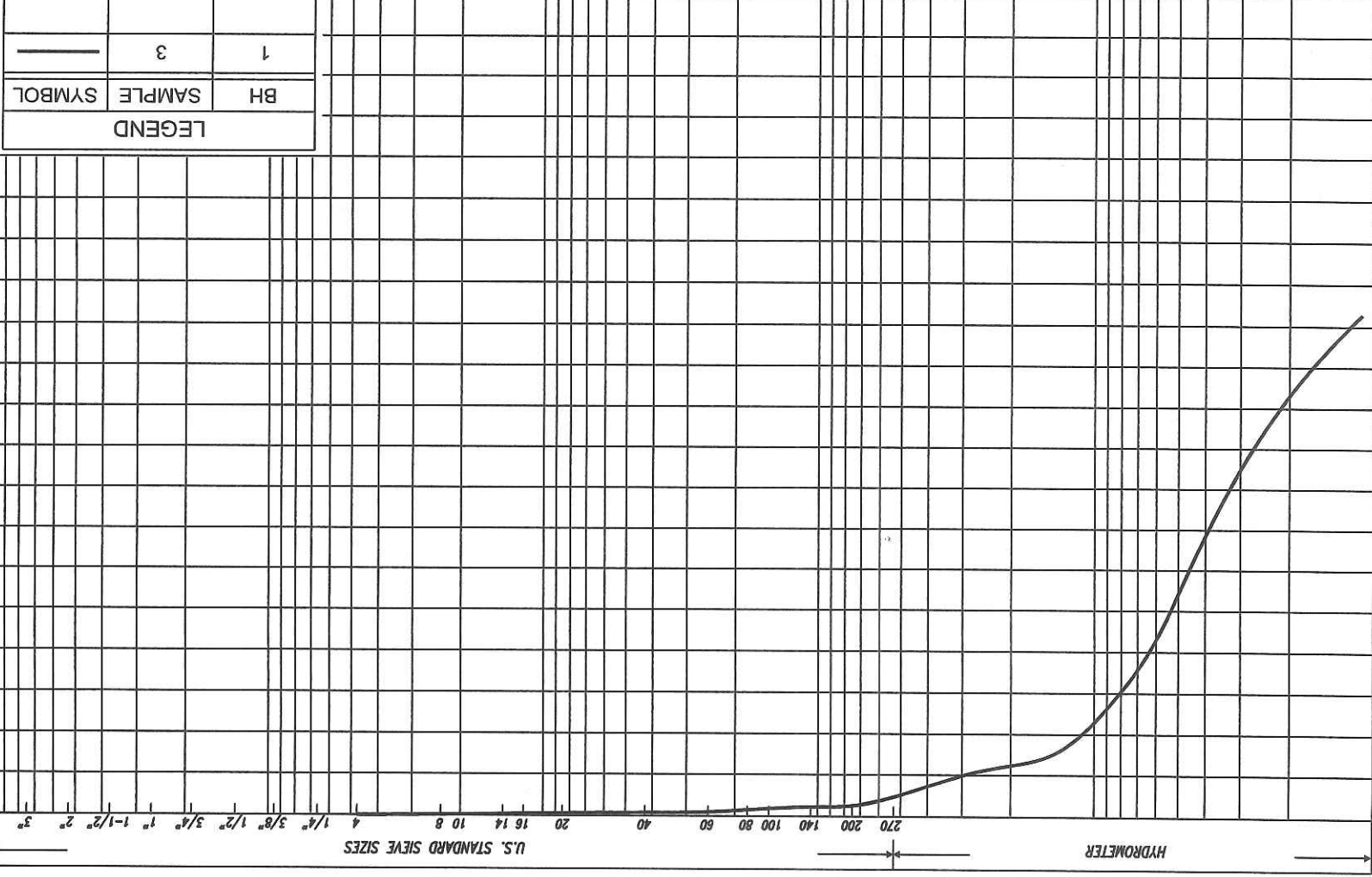
FIG No. GS-MR-1
HWY: 406
G.W.P. No. 280-99-00

U.S. BUREAU
M.I.T.
UNITED

CLAY		SILT		SAND			GRAVEL	
CLAY		SILT		V. FINE	FINE	MED.	GRAVEL	
CLAY		SILT		SAND			GRAVEL	
CLAY		SILT		COARSE	FINE	MEDIUM	COARSE	COBBLES
CLAY & CLAY		SILT		SAND			GRAVEL	
CLAY & CLAY		SILT		COARSE	FINE	MEDIUM	COARSE	COBBLES

GRAIN SIZE IN MILLIMETERS

0.001 0.005 0.01 0.05 0.1 0.5 1 5.0 10 50.0 100





Ontario
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

SILT, with sand, with gravel, some clay (Till)
GRAVEL, with sand, with silt, some clay (Till)

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

U.S. BUREAU

M.I.T.

UNITED

CLAY	SILT			SAND			GRAVEL	
	SILT			SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	MED.	COARSE	COBBLES	
CLAY	SILT & CLAY			SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	COBBLES	

GRAIN SIZE IN MILLIMETERS

0.001 0.005 0.01 0.05 0.1 0.5 1 5.0 10 50.0 100

LEGEND		
BH	SAMPLE	SYMBOL
1	15	—
2	15	- - -

CUMULATIVE PERCENT RETAINED

CUMULATIVE PERCENT PASSING

U.S. STANDARD SIEVE SIZES

HYDROMETER

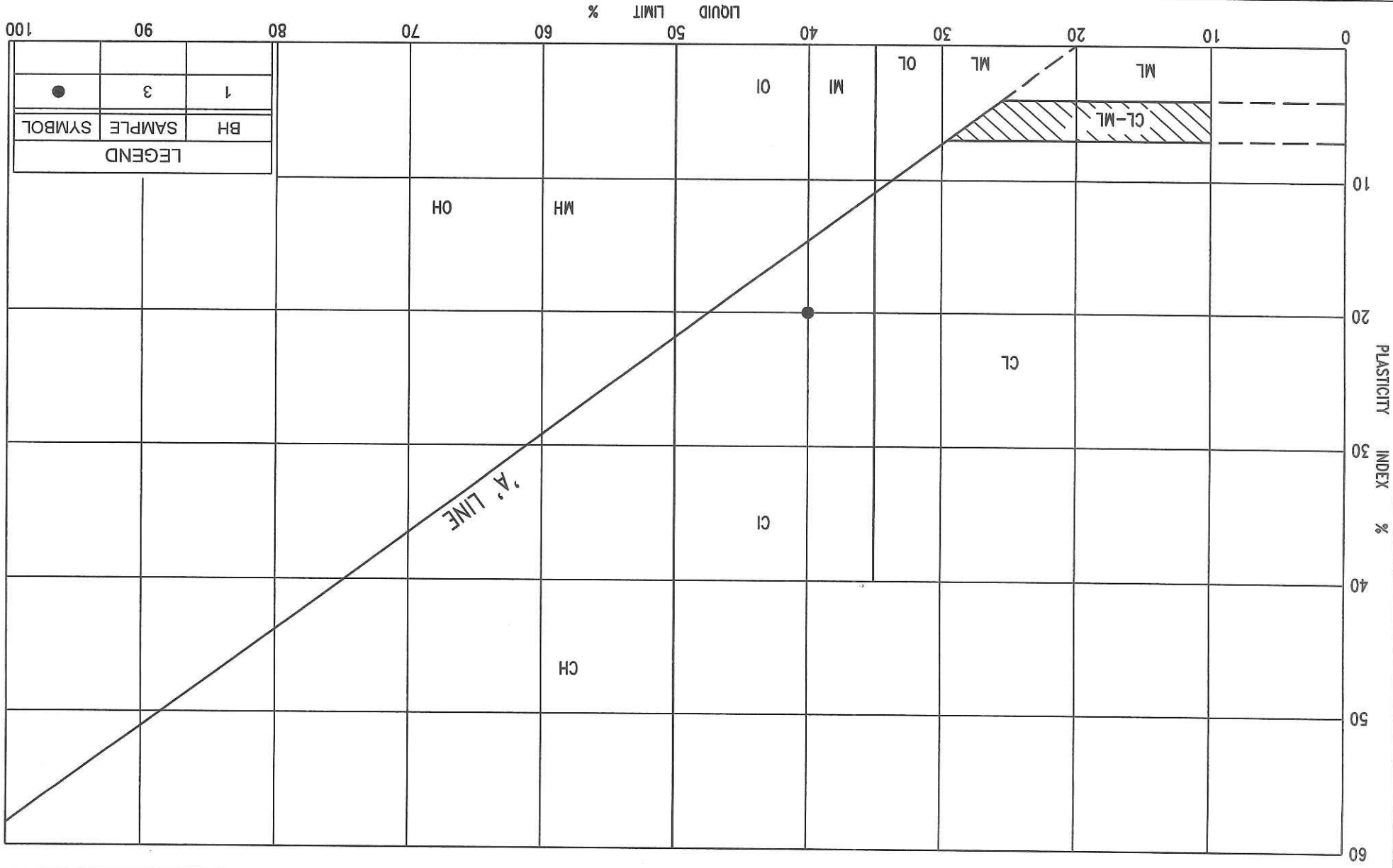


Ontario
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Transportation

PLASTICITY CHART

SILTY CLAY, trace sand

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



LEGEND

BH SAMPLE SYMBOL

1 3

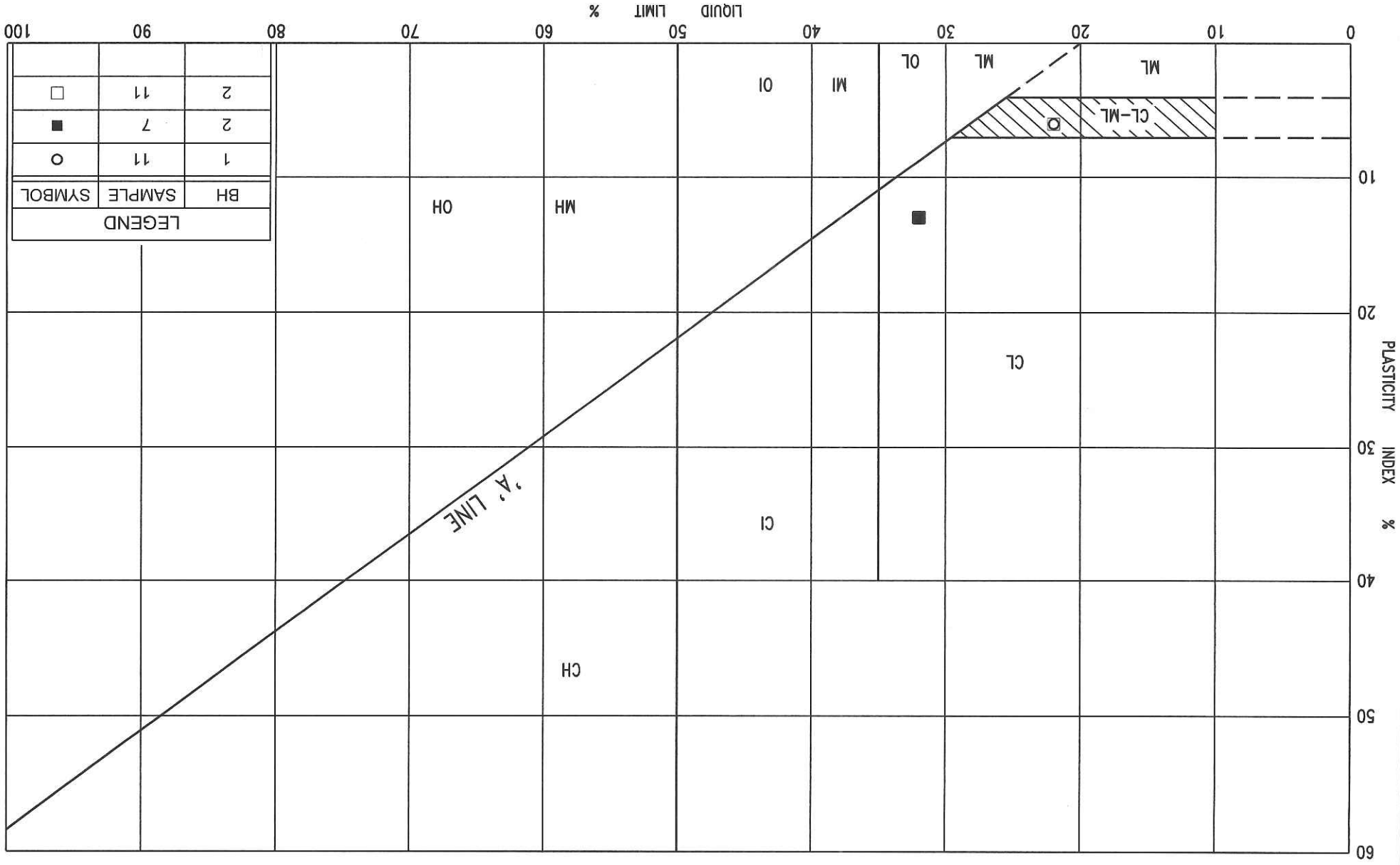
●



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PLASTICITY CHART
CLAYEY SILT, trace sand, trace gravel

FIG No. PC-MR-2
HWY: 406
G.W.P. No. 280-99-00





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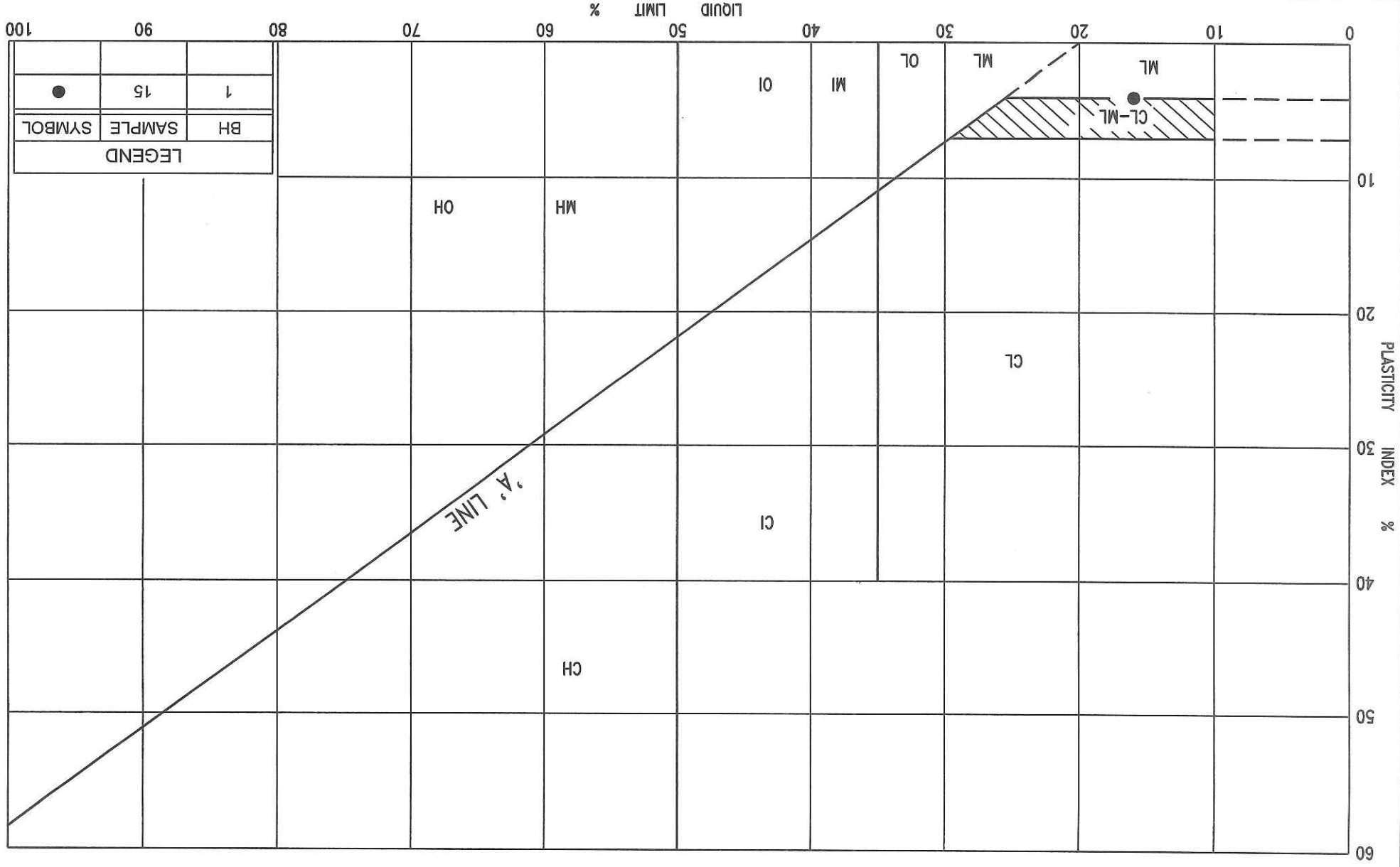
SILT, with sand, with gravel, some clay (Till)

PLASTICITY CHART

FIG No. PC-MR-3

HWY: 406

G.W.P. No. 280-99-00



60

50

40

30

20

10

0

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 N̄.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (15mm 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 0.3 m 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.) AS FOLLOWS:

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

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

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

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:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

	PORE WATER PRESSURE	PORE PRESSURE RATIO	TOTAL NORMAL STRESS	EFFECTIVE NORMAL STRESS	SHEAR STRESS	PRINCIPAL STRESSES	LINEAR STRAIN	PRINCIPAL STRAINS	MODULUS OF LINEAR DEFORMATION	MODULUS OF SHEAR DEFORMATION	COEFFICIENT OF FRICTION
	u_w	u_o	σ	σ'	τ	$\sigma_1, \sigma_2, \sigma_3$	ϵ	$\epsilon_1, \epsilon_2, \epsilon_3$	E	G	μ
	kPa	1	kPa	kPa	kPa	kPa	%	%	kPa	kPa	1

MECHANICAL PROPERTIES OF SOIL

m_v	$k_p a^{-1}$	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
e'_o	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
T_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	$^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$^\circ$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{C_u}{C_c}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	DENSITY OF SOLID PARTICLES	e	1. %	VOID RATIO
γ_s	UNIT WEIGHT OF SOLID PARTICLES	n	1. %	POROSITY
ρ_w	DENSITY OF WATER	w	1. %	WATER CONTENT
γ_w	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION
ρ	DENSITY OF SOIL	w_L	%	LIQUID LIMIT
γ	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT
ρ_d	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT
γ_d	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$
γ_{sat}	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{I_p}{w - w_p}$
ρ'_{sat}	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
ρ'	DENSITY OF SUBMERGED SOIL	e_{max}	1. %	VOID RATIO IN LOOSEST STATE
γ'	UNIT WEIGHT OF SUBMERGED SOIL			

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	1 of 3	METRIC
-------------------------	--------	--------

1 of 3

RECORD OF BOREHOLE No 1

G.W.P. 280-99-00

Co-ords: 4 766 059 N; 326 598 E

Co-ords: 4 766 059 N; 326 598 E

DIST CR HWY 406 BOREHOLE

THE Continuous Flight Hollow Stem Augers

COMPILED BY P.C.

DATUM Geodetic DATE _____

November 01, 2001

CHECKED BY D.W. K.

[illegible]

ON_MOT VER3 08TF005D.GPJ ON_MOT.GDT 7/29/2008 9:31:21 AM

$+^7, \times^5$: Numbers refer to Sensitivity
 20
 15 — ○ — 5
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 1 2 of 3 METRIC

G.W.P. 280-99-00 LOCATION Hwy 406 / Merritt Road Underpass ORIGINATED BY M.R.
 Co-ords: 4 766 059 N; 326 598 E
 DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY P.C.
 DATUM Geodetic DATE November 01, 2001 CHECKED BY D.W.K.

SOIL PROFILE		SAMPLES			ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC MOISTURE CONTENT		LIQUID LIMIT	UNSATURATED WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			W _p	W			
165.5	Clayey silt, trace sand				165						
15.0	Firm Reddish Wet Brown		11	SS							
					164						
					163						
			12	SS	162						
					161						
					160						
			13	SS	159						
					158						
					157						
156.9	Silt, with sand				156						
23.6	with gravel, some clay				155						
	Very dense Reddish Moist				154						
	to dense brown				153						
	(TILL)		14	SS	152						
					151						
			15	SS							
150.5											

Cont'd

ORIGINATED BY M.R.

COMPILED BY _____ P.C.

CHECKED BY D.W.

* 2001 11 01

Water level measured after drilling


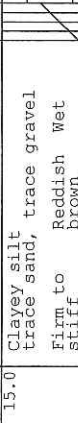





RECORD OF BOREHOLE No 2 1 of 3 METRIC

G.W.P. 280-99-00 LOCATION Hwy 406 / Merritt Road Underpass
Co-ords: 4 766 035 N; 326 542 E
DIST CR HWY 406 BOREHOLE TYPE C. F. S. S. A. + NW Wash Borings + NXL Rock Corings ORIGINATED BY M.R.
D.W.K. DATE November 05, 2001 COMPILED BY P.C.
CHECKED BY D.W.K.

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE							
180.4	Ground Surface									
0.0	PAVEMENT STRUCTURE: 90mm asphaltic concrete over 150mm granular 'A' crushed limestone (FILL)									
0.3	Silty clay, trace sand	1	SS	28						
	Very stiff Mottled Moist to hard Brown	2	SS	30						
		3	SS	34						
		4	SS	18						
	Very stiff to stiff	5	SS	9						
	Firm to soft	FV								
	clayey silt layers	6	SS	4						
		FV								
		7	SS	3						
		FV								
		8	SS	3						
170.3	Silt, trace sand									
10.1	Very loose Reddish Wet Brown	9	SS	3						
168.8	Clayey silt, trace gravel									
11.6	Firm Reddish Wet brown	10	SS	9						
165.4										

RECORD OF BOREHOLE No 2	2 of 3	METRIC
-------------------------	--------	--------

G.W.P.	280-99-00	LOCATION	Hwy 406 / Merritt Road Underpass Co-ords: 4 766 035 N; 326 542 E		ORIGINATED BY	M.R.
DIST	CR HWY 406	BOREHOLE TYPE	C. F. S. S. A. + NW Wash Borings + NXI Rock Corings		COMPILED BY	P.C.
DATUM	Geodetic	DATE	November 05, 2001		CHECKED BY	D.W.K.

SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				W _p	W	W _L		
165.4	Clayey silt trace sand, trace gravel Firm to Reddish Wet stiff		11	SS	10		165					2 1 81 16	
15.0													
	particles of gravel		12	SS	15		162						
	Hard		13	SS	10		159						
154.5	Silt, with sand with gravel, some clay Dense Reddish Moist brown (TILL)		14	SS	34		156						
25.9													
153.1	Gravel, with sand with silt, some clay Very dense Reddish Moist brown (TILL)		15	SS	65		153					45 21 23 11	
27.3													
152.0	Silt, with sand with gravel, some clay Dense Reddish Moist brown (TILL)						152						
28.4													
150.4							151						

RECORD OF BOREHOLE No 2 3 of 3 METRIC

G.W.P. 280-99-00 LOCATION Hwy 406 / Merritt Road Underpass
Co-ords: 4 766 035 N; 326 542 E
DIST CR HWY 406 BOREHOLE TYPE C.F.S.S.A. + NW Wash Borings + NXL Rock Corings ORIGINATED BY M.R.
D.W.K. DATE November 05, 2001 CHECKED BY P.C.

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	W _p	W _L	
150.4	Silt, with sand with gravel, some clay Compact Reddish Moist brown (TILL) Very dense		16	SS	28				
149.4									
148.4									
147.4									
146.4									
145.4	Bedrock Dolostone Buff to grey Low to medium strength Unweathered Poor quality		17	SS	100/10cm				
144.4									
143.4									
142.4			19	RC NQ	REC 93%				RQD 32%
141.4									
140.4	End of borehole		20	RC NQ	REC 100%				RQD 35%
139.4									
138.4									
137.4									
136.4									

* Borehole charged with drilling water
■ Penetrometer test

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

METRIC
DIMENSIONS ARE IN METRES
UNLESS OTHERWISE SHOWN
OTHERWISE SHOWN
IN KILOMETRES + METRES

CONT No 280-99-00

GWP No 280-99-00

MERRITT ROAD UNDERPASS

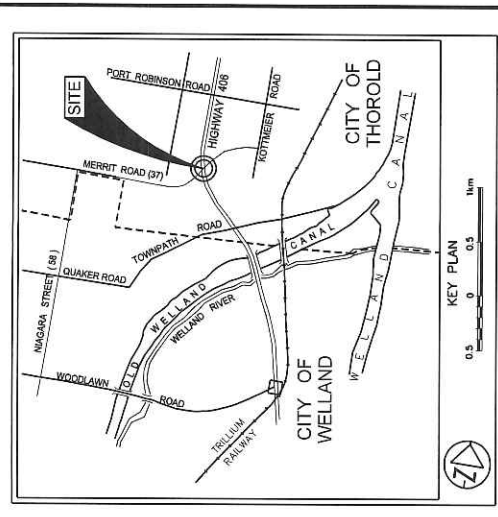
HIGHWAY 406

BOREHOLE LOCATIONS

SHEET

PMI 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 Nov 2001

Head

ARTESIAN WATER Encountered

PIEZOMETER

BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
1	180.5	4 766 059	326 598
2	180.4	4 766 035	326 542

NOTE

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

REVISIONS

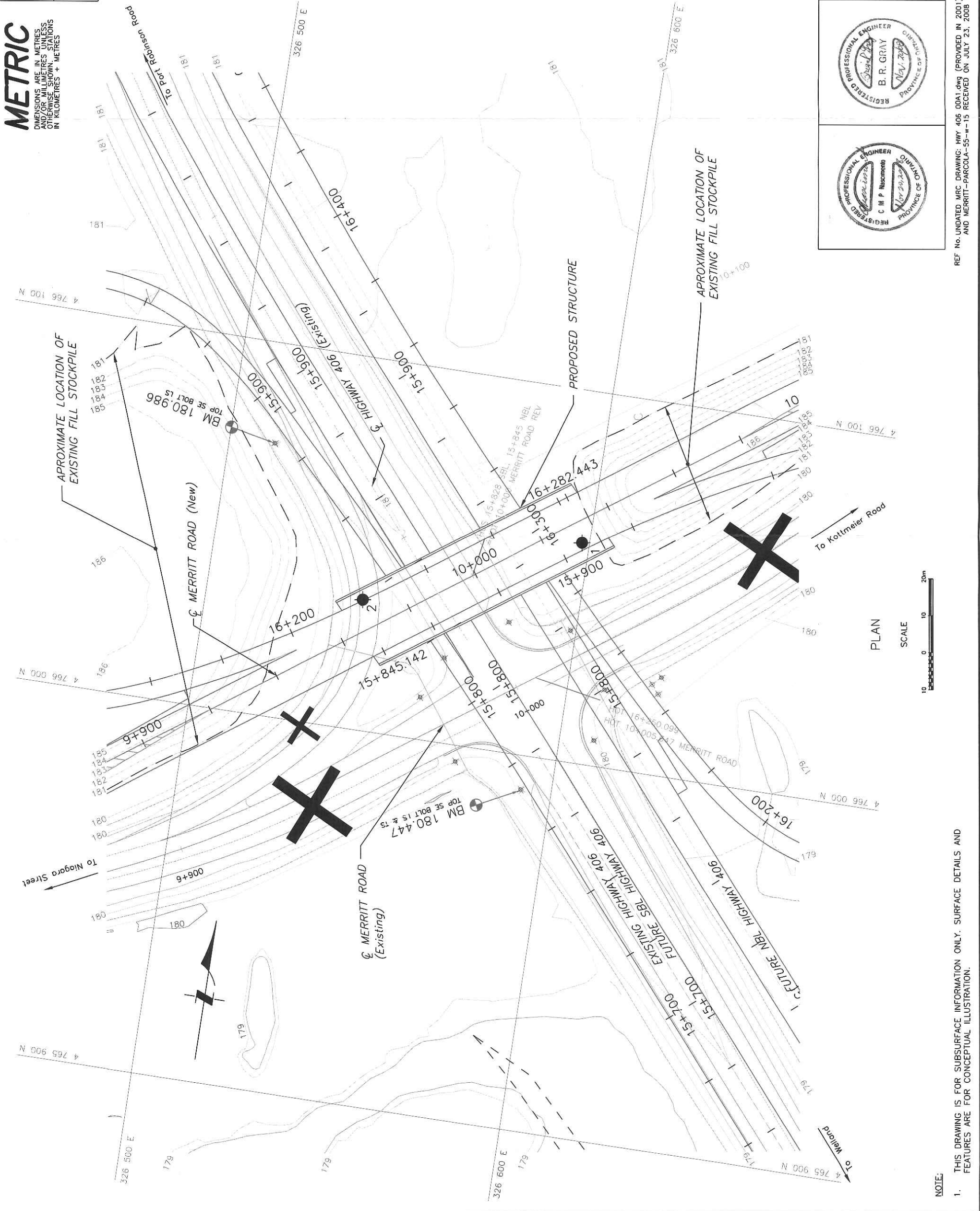
DATE BY DESCRIPTION

Geocres No. 30M03-233

HWY No 406

SUBD NSB CHECKED NSB DATE NOV. 20, 2008 SITE

DRAWN NA CHECKED CN APPROVED BRG DWG MR-1



REGISTERED PROFESSIONAL ENGINEER

B. R. GRAY

PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER

C. M. P. MacCallum

PROVINCE OF ONTARIO

REF No. UNDATED MRC DRAWING: HWY 406 00A1.dwg (PROVIDED IN 2001)

AND MERRITT-PARCOLA-55-w-15 RECEIVED ON JULY 23, 2008

NOTE:

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