



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**EAST MAIN STREET OVERPASS
HIGHWAY 406 FOUR-LANING
GWP 280-99-00
CITY OF WELLAND, ONTARIO**

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

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PML Ref.: 08TF005H
Index No.: 114FIDR
Geocres No.: 30M03-238
January 21, 2009



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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

for
East Main Street Overpass
Highway 406 Four-Laning
GWP 280-99-00
City of Welland, Ontario

1. INTRODUCTION

This report summarizes the results of the preliminary foundation investigation carried out for a possible future East Main Street Overpass at Highway 406 in the City of Welland. 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 EA approved southerly extension of Highway 406 beyond the 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 possible future overpass would carry the realigned Highway 406 traffic over the East Main Street lanes at approximate Sta. 11+052 (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).

2. SITE DESCRIPTION AND GEOLOGY

The contemplated structure is proposed at the existing East Main Street (Sta. 10+000). The subject site is about 270 m from the vehicular tunnel under the Welland Canal.

Land use in the vicinity of the site comprises the transportation corridors of the existing East Main Street and Highway 406. The East Main Street slopes down from west to east towards the west entrance of the East Main Street Tunnel (see Appendix A for a site photograph). The original ground at the structure location was cut down about 10 m (from about elevation 183 to 173) to construct the tunnel access.



The site is located in the Haldimand Clay Plain physiographic region. The soil cover in the region typically comprises lacustrine silts and clays. A bedrock formation comprising evaporites and shale underlain by dolomite bedrock (Salina Formation) is anticipated at the site.

3. INVESTIGATION PROCEDURES

The field work was carried out on October 1 and 16, 2008. Two sampled boreholes were put down at the site. The boreholes were drilled to refusal at depths of 24.5 and 21.6 m at the locations shown on Drawing EM-1. Borehole EM1 was extended by coring 3.8 m into bedrock to a total depth of 28.3 m.

The locations of the 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 of our engineering staff.

Representative soil samples of the soils 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 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 EM1, N casing was extended to the bedrock surface and an approximate 3.8 m length of rock core was recovered using NX rock coring equipment. A PML senior geologist examined and classified the recovered rock core samples. Detailed descriptions of the recovered rock core are provided in Table A and a photograph of the core is shown in Appendix B.

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 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 (28)
- Grain size analyses (8)
- Atterberg limits (7)

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-EM-1 to GS-EM-3. The Atterberg limits results are presented on Figures PC-EM-1 and PC-EM-2.

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 possible future East Main Street overpass are presented on the attached Drawing EM-1.

The subsurface stratigraphy revealed in the boreholes generally consisted of a surficial 0.2 m thick topsoil and a 1.4 m thick surficial fill unit composed of loose sandy silt and stiff silty clay overlying



a soft to hard clayey silt till extending to the bedrock surface. In borehole EM1, at 5.4 m (elevation 168.5), the clayey silt till was interbedded by a 3.0 m thick compact sand and silt deposit which in turn overlaid a 5.9 m thick dense to very dense silty sand till. Salina Formation bedrock including evaporites and shale layers in turn underlain by dolomite was contacted below the native soils at depths of 24.5 and 21.6 m, elevation 149.4 and 149.8.

Bubbles were noticed in water at top of casing during rock coring. The bubbles are likely due to methane gas presence.

4.2 Topsoil

A 200 mm thick of topsoil was encountered in borehole EM2 and penetrated at 0.2 m depth, elevation 171.2.

4.3 Fill

A fill unit was found surficially in borehole EM1 extending to 1.4 m depth, elevation 172.5. The variable fill units included layered loose sandy silt, clayey silt and silty sand overlying stiff silty clay and contained topsoil, sand and thin lenses silt. Two N values of 9 were recorded. Water content determinations were 5 and 17%.

4.4 Clayey Silt Till

A cohesive deposit of clayey silt till was present below the fill in borehole EM1 at a depth of 1.4 m (elevation 172.5) and below the topsoil in borehole EM2 at a depth of 0.2 m (elevation 171.2).

In borehole EM1, the stratum consisted of upper and lower layers interbedded by cohesionless sand and silt between a depth of 5.4 m, elevation 168.5 and 14.3 m, elevation 159.6. The upper stratum was 4.0 m thick and the lower stratum was 10.2 m thick extending to the bedrock surface at 24.5 m, elevation 149.4.



In borehole EM1, as auger refusal was encountered at 21.3 m (elevation 152.6), the borehole was advanced by coring (RC16 and RC17) through a zone of the till containing cobbles, gravel and sand to 24.5 m (elevation 149.4) where bedrock was encountered.

In borehole EM2, the stratum extended to the inferred bedrock surface at 21.6 m depth, elevation 149.8.

The clayey silt till deposit typically exhibited stiff to hard consistency with one soft local layer at a depth of 1.5 m. Penetrometer test results on two samples were about 50 kPa. N values ranged from 3 to 50 blows per 100 mm penetration.

The grain size distribution chart of six representative samples of the clayey silt till is shown on Figure GS-EM-1.

The plasticity charts for the 6 samples are presented in Figure PC-EM-1. The liquid and plastic limits and plastic index for the materials are shown on the corresponding boreholes log sheets and are summarised below.

MATERIAL	BOREHOLE NO.	SAMPLE	DEPTH (m)	WATER CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Clayey Silt (Till)	EM1	3	1.5 – 2.1	18	30	16	14
	EM1	14	16.7 – 17.2	16	20	14	6
	EM2	2	0.8 – 1.4	20	27	15	12
	EM2	4	3.0 – 3.6	19	31	15	16
	EM2	8	9.1 – 9.3	15	18	12	6
	EM2	11	13.7 – 14.3	16	20	12	8

Natural moisture content determinations ranged widely from 8 to 38%. The values were typically in the 16 to 19% range in the upper layers decreasing to typically 10 and 11% in the lower layers. The water contents were typically lower than the liquid limits of the soils indicating relatively low compressibility characteristics.



4.5 Sand and Silt

A 3.0 m thick sand and silt deposit containing trace clay and cobbles was encountered beneath the clayey silt till at a depth of 5.4 m (elevation 168.5) in borehole EM1. The layer extended to the silty sand deposit at a depth of 8.4 m (elevation 165.5). The sand and silt deposit was compact with N values of 19 and 24. The moisture content of the layer was 19 and 21%.

4.6 Silty Sand Till

A deposit of cohesionless glacial till including dense to very dense silty sand till was encountered in borehole EM1 below the sand and silt deposit at a depth of 8.4 m (elevation 165.5). The N values in the till typically ranged from 32 to 50 blows per 80 mm penetration. The thickness of till deposit was 5.9 m and was underlain by clayey silt till at a depth of 14.3 m, elevation 159.3.

The composition of the silty sand till contains some gravel, trace clay, cobbles and boulders. The grain size distribution chart of one representative sample of the silty sand till is shown on Figure GS-EM-3 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-EM-2. The liquid limit of the silty sand till was 14 the plastic limit 12, giving the plasticity index value of 2. The till sample was non-plastic according to the Atterberg determination and manual examination. The water content of the silty sand till ranged from 9 to 19%, indicating moist to wet conditions.

4.7 Bedrock

Evaporites and shale underlain by dolomite bedrock of the Salina Formation was encountered in borehole EM1 below the native soils at the levels listed in the following table.

BOREHOLE No.	DEPTH (m)	ELEVATION	ROCK CORE LENGTH (m) (*)
EM1	24.5	149.4	3.8
EM2	21.6	149.8	-

(*) NXL diamond rock cores obtained.



The upper layer of the bedrock from 24.5 to 27.3 m depths (elevation 149.4 to 146.6) includes evaporites and shale which exhibited a low to medium strength and was found to be unweathered. The core recovery in this zone was 94 and 100% and the rock typically was of very poor to poor quality (RQD values were 0 and 42%).

From 27.3 m (elevation 146.6) to 28.3 m (elevation 145.6), the dolomite rock core recovery was 100%. The dolomite exhibited a medium strength and was found to be unweathered. The rock typically is of poor quality (RQD value was 48%).

Loss of drilling water was not experienced during drilling. The detailed rock core descriptions are provided on Table A and a photograph of the core is shown in Appendix B.

4.8 Groundwater

Upon completion of drilling, groundwater was measured in borehole EM2 at a depth of 2.1 m (elevation 169.3). The groundwater table was not determined upon completion of drilling in borehole EM1 because the borehole was charged with water for rock coring purposes.

The water level in the Welland Canal which is about 270 m from the overpass site is not expected to affect the groundwater table at the structure location because of low hydraulic conductivity for impervious clayey silt deposit which dominates the site.

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.P. Nascimento, P.Eng., Project Manager. The drilling equipment was supplied by Elite Drilling and GeoEnvironmental Drilling.

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
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Highway 406 Four-Laning
GWP 280-99-00
City of Welland, 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 possible future East Main Street Overpass at the Highway 406 in the City of Welland, 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 possible future new overpass would carry the Highway 406 southbound and northbound lanes and ramps over East Main Street at approximate Sta. 11+052. As indicated by MRC, the contemplated overpass will include two single-span structures with spans of approximately 54 m.

In summary, the subsurface stratigraphy revealed in the boreholes generally consisted of a surficial 0.2 m thick topsoil and a 1.4 m thick surficial fill unit made of loose sandy silt and stiff silty clay overlying a soft to hard clayey silt till extending to the bedrock surface. In borehole EM1 (north abutment) the clayey silt till was interbedded at 5.4 m (elevation 168.5) by a 3.0 m thick compact sand and silt deposit which in turn overlaid a 5.9 m thick dense to very dense silty sand till. Salina Formation bedrock including evaporites and shale underlain by dolomite was contacted below the native clayey silt till at 24.5 and 21.6 m depths in the boreholes.

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

The pile lengths will vary in view of the variable relative density of the native soils, including compact/dense to very dense silty sand till and/or hard clayey silt till which mantle the bedrock. It is likely that the piles will find refusal in the dense/hard glacial tills found at the north and south abutments. The geotechnical resistance of the deep foundations should be designed for bearing on the glacial till layer at the north and south abutments for preliminary design purposes. The



piles should be provided with driving shoes/pile points due to potentially heavy driving through the glacial till deposit which contains cobbles and boulders.

Boreholes were not drilled for the bridge approach embankments. The fills for these embankments will be placed within previously cut areas where the East Main Street was cut down about 10 m to provide vehicular access to the East Main Street Tunnel under the Welland Canal. Subject to a detail design investigation, it is estimated that the new approach fills should not cause significant settlements on the native soils.

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 final 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 potential future overpass structure on pile foundations driven to practical refusal on the dense to very dense cohesionless soils or hard clayey silt till or bedrock is considered feasible. Spread 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, 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.

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 north and south abutments of the two future potential structures should be driven to refusal into the very dense silty sand till or hard clayey silt till. The estimated preliminary founding reference level of silty sand till for the north abutments and hard clayey silt till for the south abutments are provided in the following table:

FOUNDATION ELEMENT	BOREHOLE No.	FOUNDING DEPTH (m)	STRATUM FOUNDING ELEVATION	ESTIMATED PILE TIP ELEVATION
North Abutments	EM1	11.9	162.0	160.0
South Abutments	EM2	7.4	164.0	162.0

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.

For piles driven into the very dense silty sand till at the north abutment and hard clayey silt at the south 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. 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 ULS factored capacity.

The piles will be driven through hard soils and into very dense glacial till soils or may set on the bedrock found under the abutments. Consequently, the piles should be equipped with steel H-pile driving shoes (OPSD-3000.100) or the Titus H Bearing Pile Point Standard Model (SP 903S01) to minimize the potential for damage when driving through the glacial till soils containing cobbles and boulders. All piles should be re-tapped to ensure adequate seating into the glacial till.

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 very dense silty sand or hard clayey silt till or bedrock anticipated at the depths/elevations and axial resistance are indicated in the previous section. A 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 a compacted fill. Typically, cohesive very stiff to soft 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:

	FIRM TO STIFF 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)

$$k_s = n_h z/b$$

where n_h = coefficient related to soil density
 = 10.0 MN/m³ 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 clay overburden should be taken as 28,000 kN/m³ for preliminary design purposes.



6.2.3 Shallow Foundations

6.2.3.1 Spread Footings on Native Soil

As previously indicated in Section 6.2.1, supporting the abutments of the overpass structures on conventional spread footings founded on native soil is considered to be feasible.

Spread footings should be constructed on the native soils comprising typically stiff clayey silt at the proposed elevations 172.3 (1.6 m depth) and 168.0 (3.4 m depth) at the north and south abutments (boreholes EM1 and EM2), respectively. The recommended bearing resistance for footings constructed on the native soils is as follows:

Factored Geotechnical Resistance at ULS, kPa	400
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 1.6 and 3.4 m and groundwater levels below founding depth at borehole EM1 and above the founding depth at borehole EM2 were 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.

The free water measured in borehole EM2 is high and at depth of 2.1 m below grade, elevation 169.3. Therefore, locally lowering the water level is required before footing construction.



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 if considered at this site. 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 2.5 m minimum thickness of the structural fill pad was used for the computation. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance. 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.

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 \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) 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 Construction Considerations

6.4.1 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 fill and the upper cohesive stiff to soft clayey silt till encountered in the boreholes are considered Type 3 soil according to OHSA (Ontario Regulation 213/91) criteria.

6.4.2 Groundwater Control

Groundwater was observed during the course of the field work at a depth of 2.1 m (elevation 169.3) at the south abutments and will not affect the construction excavations for deep foundations. Construction of spread footings at the south abutments would require the use of sump pumping to control the water seepage into the excavation. 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 by drilling boreholes to the full depth contemplated for the alternative shallow foundation construction.

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 overpass site. Incorporate the data from the boreholes drilled for this report for the Detail Design, if applicable to the final location of the structures.
2. Determine/evaluate the extent and depth of the localized very dense till soils where these are relevant to the installation of driven piles for the structure foundations.
3. Evaluate the settlement of native soils due to the construction of the future approach embankments considering that the original ground at East Main Street was cut down up to 10 m to provide vehicular access to the East Main Street tunnel.



8. CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Idib (Adeeb) Sadoun, MSc, P.Eng. Project Engineer, and reviewed 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

NOTE: Hard copies signed and stamped

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



TABLE A
ROCK CORE DESCRIPTION

CORE RECOVERY					CORE DESCRIPTION	
LOCATION (BH)	RC	DEPTH (m)	REC (%)	RQD (%)	DEPTH (m)	DESCRIPTION
EM1	18	24.5 – 24.8	100	42	24.5 – 27.3	EVAPORITES AND SHALES: Light brown and white, evaporitic carbonates, anhydrite and gypsum, fine crystalline interbedded with bluish grey dolomitic shales, aphanitic, low to medium strength, unweathered, very close to close spaced flat partings, rough to smooth planar, tight, very poor to poor quality. [Salina Formation]
	19	24.8 – 26.1	94	40		
	20	26.1 – 26.7	100	0		
	21	26.7 – 28.3	100	48	27.3 – 28.3	DOLOMITE: Brown, aphanitic to fine crystalline, medium strength, unweathered, close to moderate spaced flat bedding layers, rough to smooth planar, with some dipping cross joints, tight to 1 mm aperture, infilled with gypsum, poor quality. [Salina Formation]

RQD = Rock Quality Designation

Originated: JFW
 Compiled: FP
 Checked: IS / CN



TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

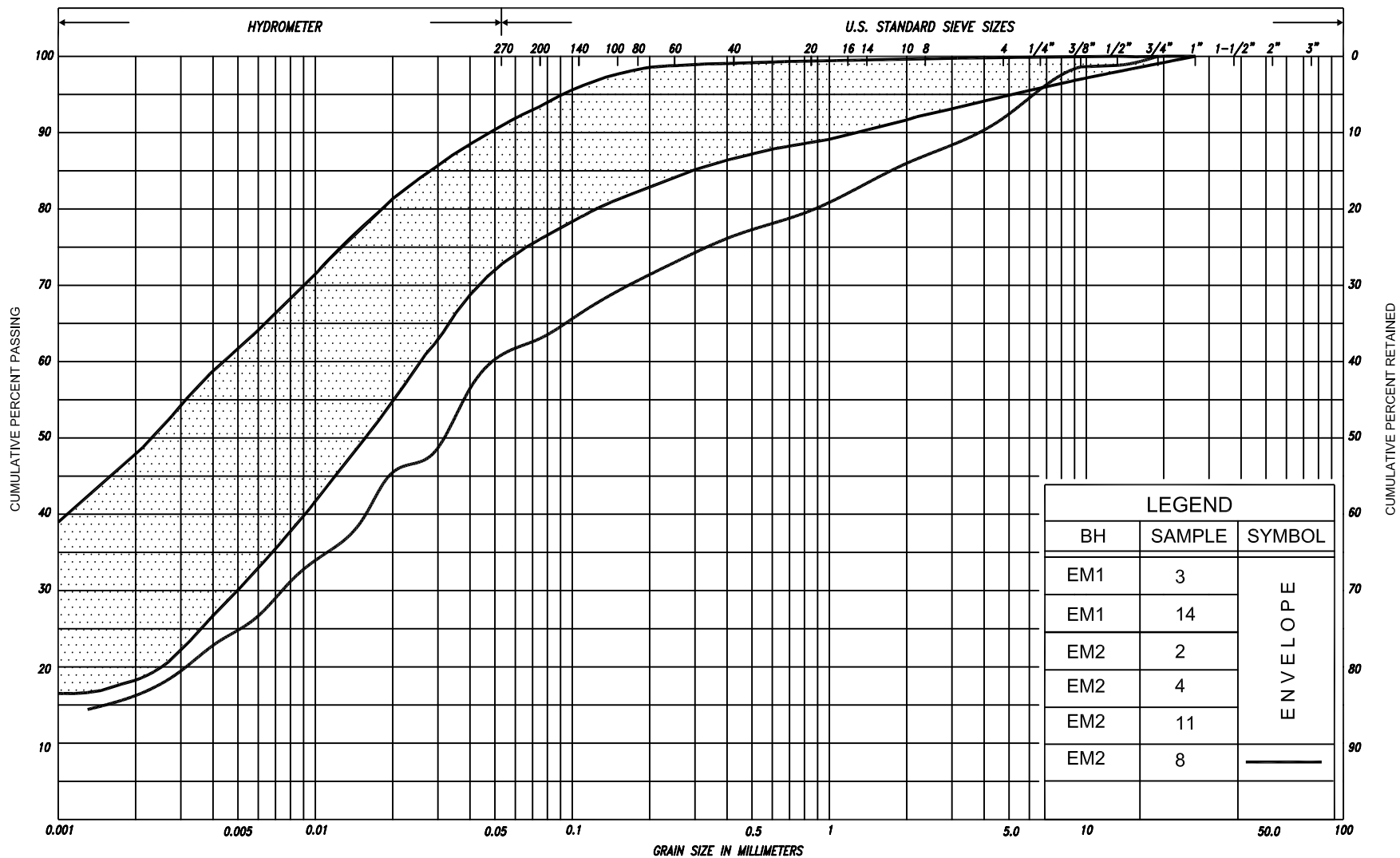
DOCUMENT	TITLE
OPSS 902	Excavation and Backfilling of Structures
SP 405F03	Construction Specification for Pipe Subdrains
SP 599S22	Requirements for The Design, Supply and Construction of Retaining Soil Systems (RSS)
SP 902S01	Excavation and Backfilling of Structures
SP 903S01	Construction Specification for Piling
OPSD-3000.100	Foundation Piles – Steel H-Pile Driving Shoe
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.



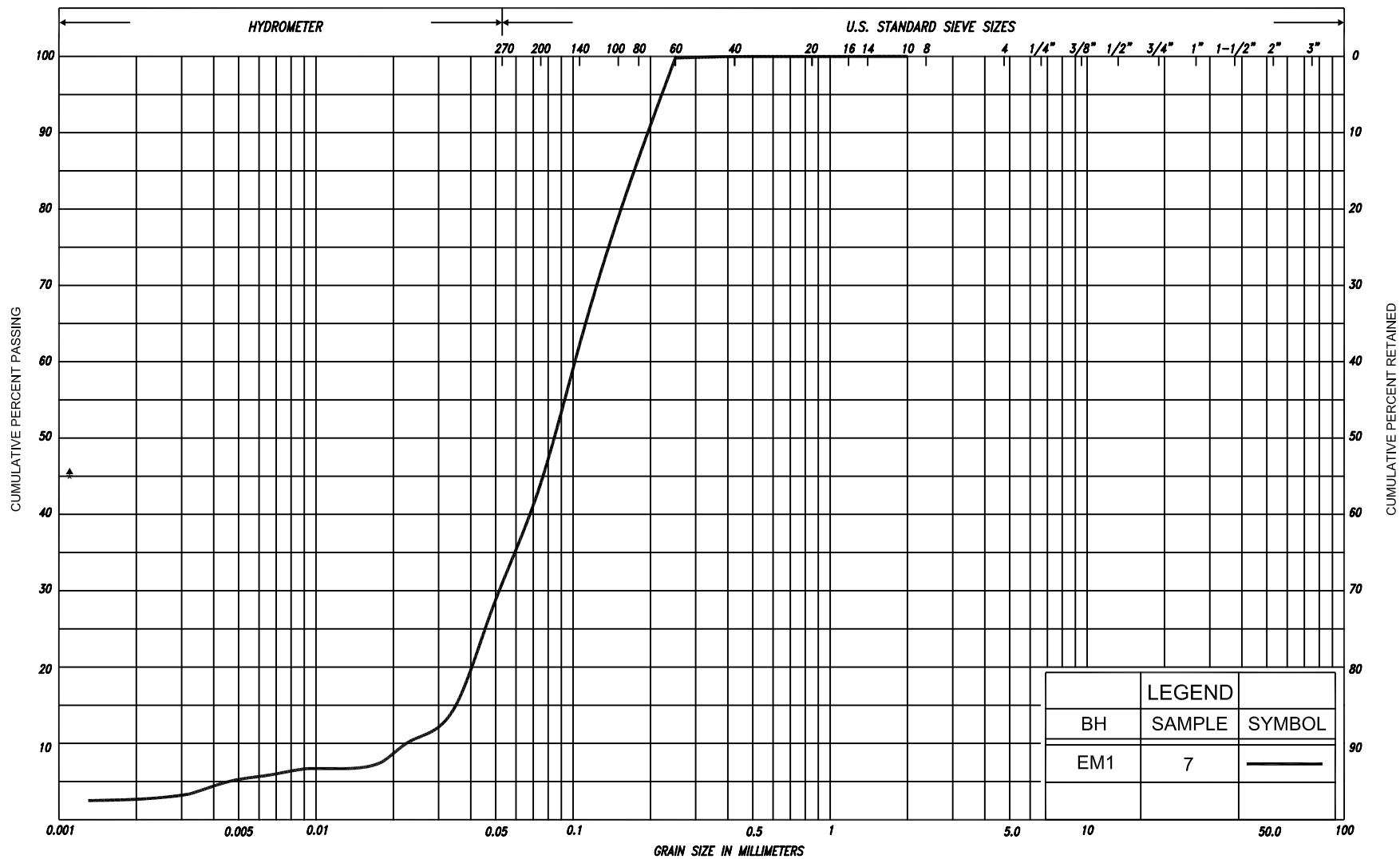
SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE	GRAVEL			COBBLES	M.I.T.	
	SILT					SAND										
CLAY		SILT			V. FINE	FINE	MED.	COARSE	GRAVEL							U.S. BUREAU

GRAIN SIZE DISTRIBUTION CLAYEY SILT, with to trace sand, trace gravel (TILL)

FIG No. GS-EM-1

HWY: 406

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



SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COB BLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.	
	SILT							V. FINE		FINE	MED.	COARSE	GRAVEL			U.S. BUREAU	
CLAY			SILT					SAND									



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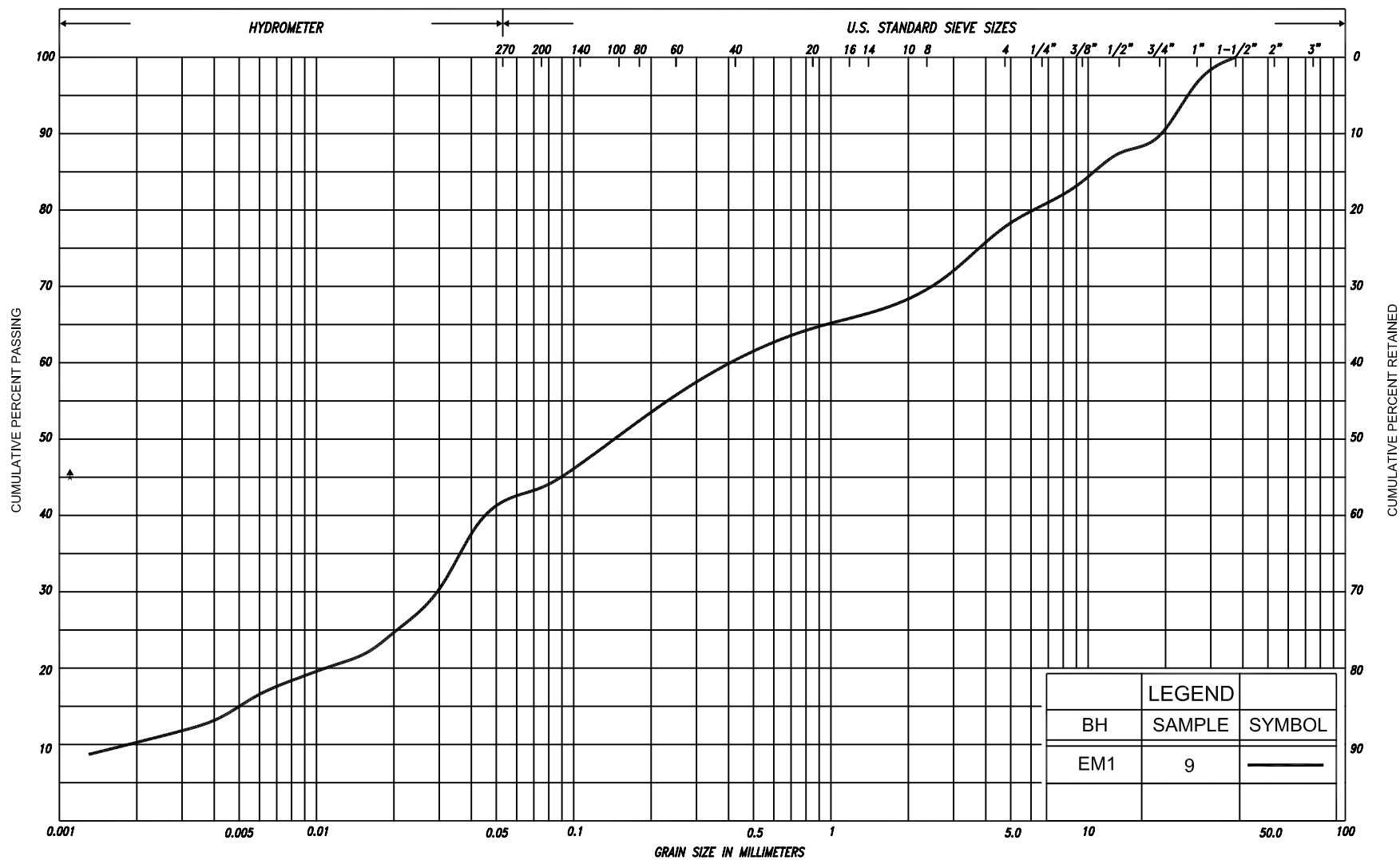
GRAIN SIZE DISTRIBUTION

SAND AND SILT, trace clay

FIG No. GS-EM-2

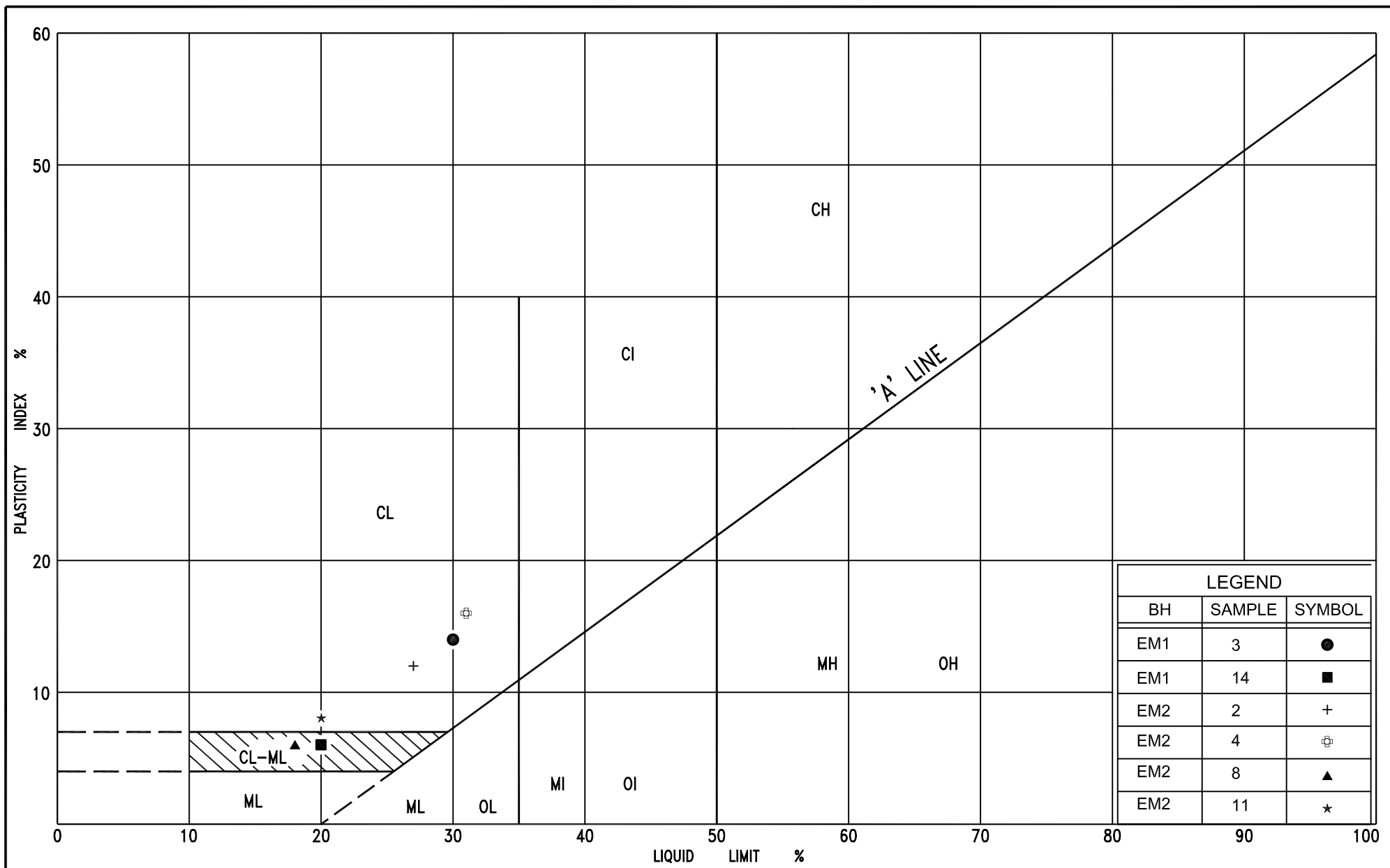
HWY: 406

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

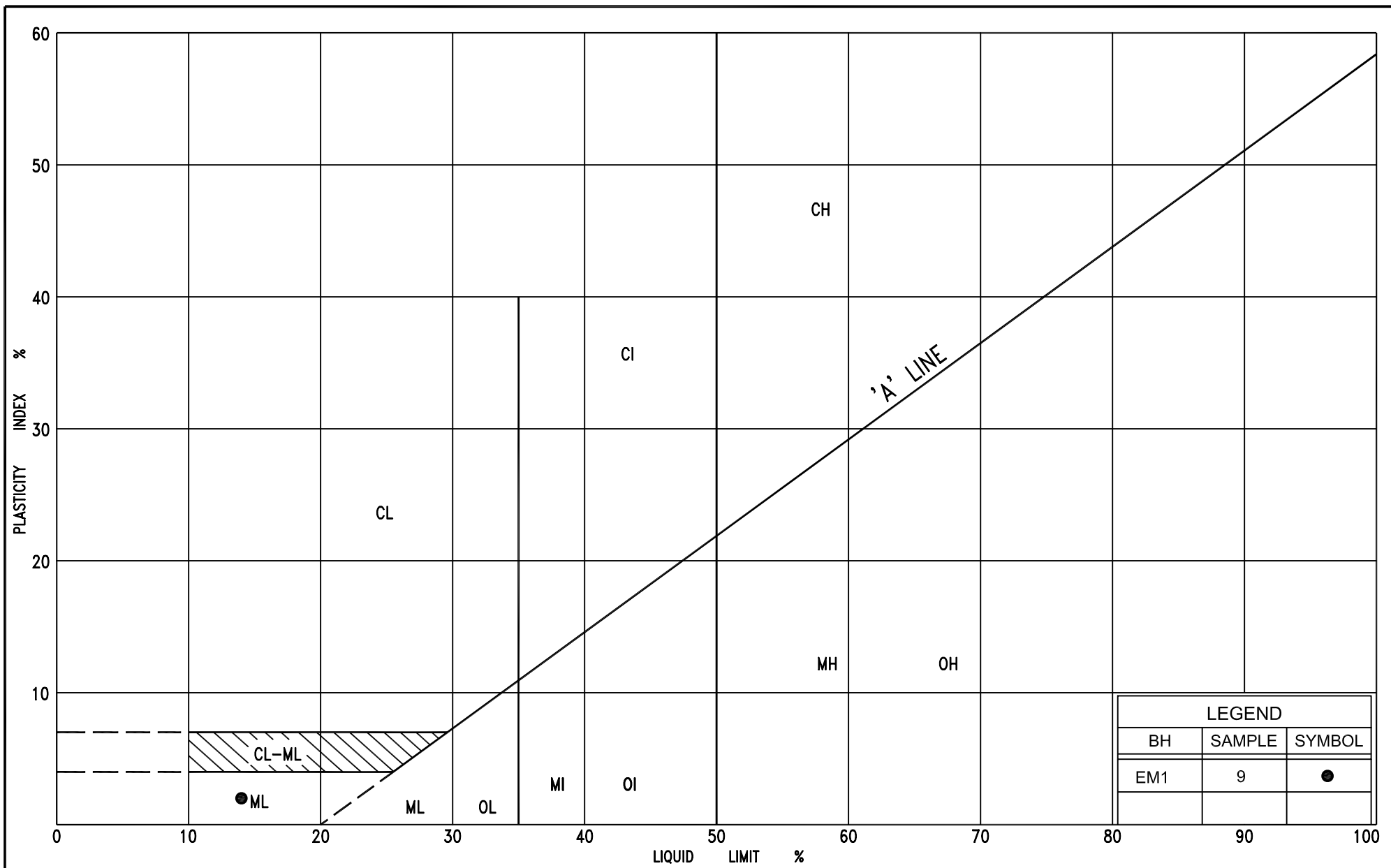


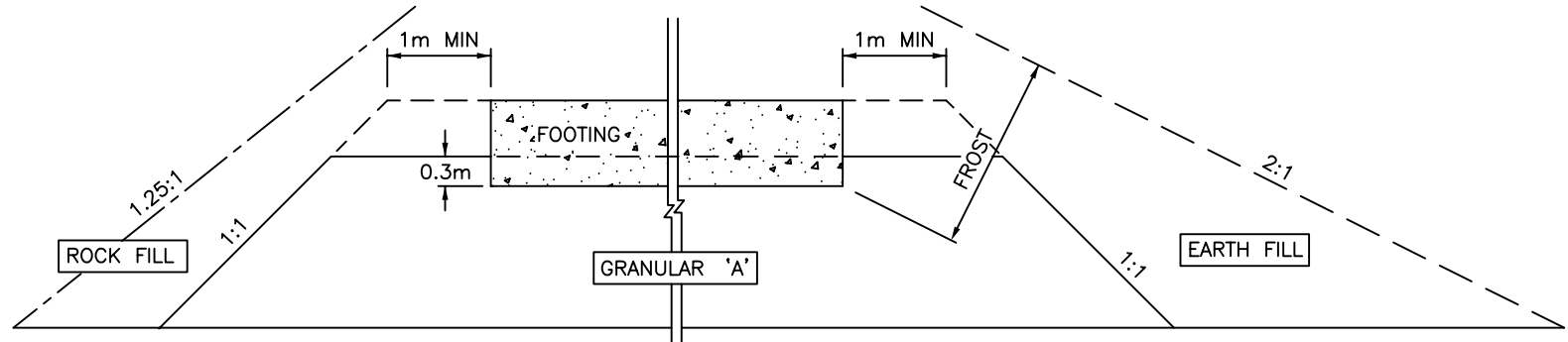
LEGEND		
BH	SAMPLE	SYMBOL
EM1	9	—

SILT & CLAY			FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED
					SAND						
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		GRAVEL		COBBLES	M.I.T.
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL			U.S. BUREAU
				SAND							



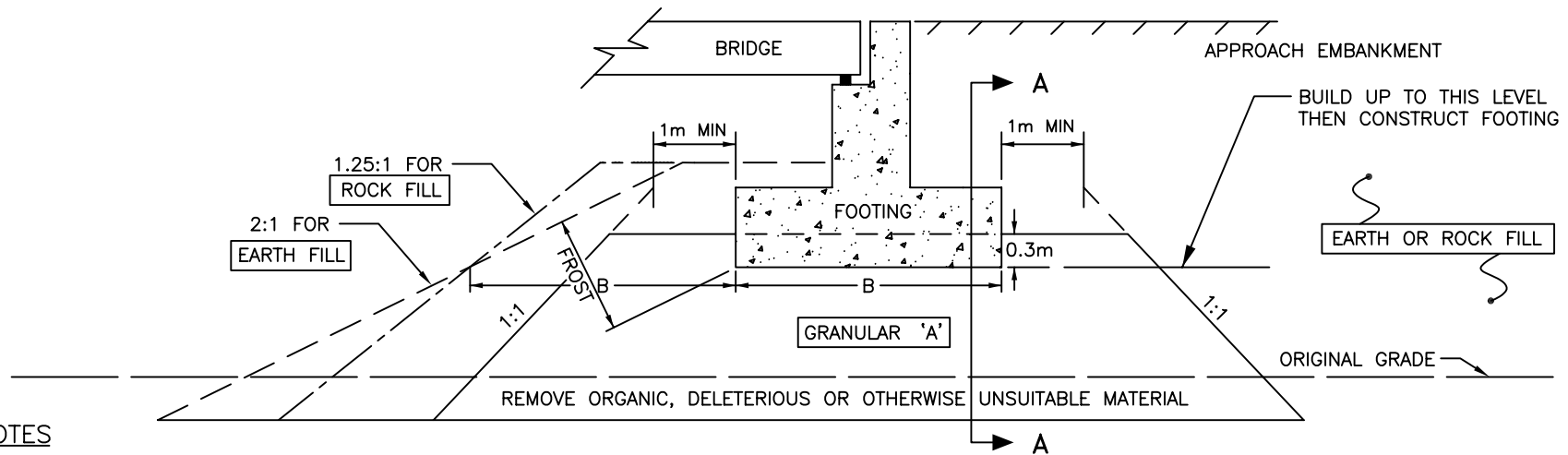
LEGEND		
BH	SAMPLE	SYMBOL
EM1	3	●
EM1	14	■
EM2	2	+
EM2	4	⊠
EM2	8	▲
EM2	11	★





CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

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 (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
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	n	1, %	POROSITY	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m^2	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No EM1

1 of 3

METRIC

G.W.P. 280-99-00 LOCATION Hwy 406/ East Main Street Overpass
DIST CR HWY 406 BOREHOLE TYPE C.F.H.S.A. + NX Diamond Corings
DATUM Geodetic DATE October 01, 2008

ORIGINATED BY M.R.
COMPILED BY A.S.
CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
173.9	Ground Surface																
0.0	Sandy silt, clayey silt silty sand, topsoil, sand		1	SS	9		173										
	Loose Dark Moist brown																
	Silty clay thin silt lenses		2	SS	9												
172.5	Stiff Reddish Wet brown (FILL)																
1.4	Clayey silt trace sand, trace gravel		3	SS	9		172										2 8 52 38
	Stiff Reddish Moist brown																
	layers of silt cobbles		4	SS	42		171										
	(TILL)		5	SS	12		170										
	Wet																
			6	SS	12		169										
168.5	Sand and silt																
5.4	Compact Reddish Wet brown		7	SS	19		168										0 56 41 3
							167										
			8	SS	24		166										
165.5	Silty sand some gravel, trace clay cobbles and boulders						165										
8.4	Dense Reddish Moist brown		9	SS	32												22 34 34 10
	(TILL)						164										
			10	SS	52		163										
	very dense Wet						162										
			11	SS	50/8cm		161										
			12	SS	50/10cm		160										
159.6	Clayey silt trace sand, trace gravel																
14.3																	
158.9	Cont'd						159										

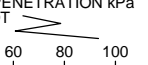


METRIC

— CHECKED BY C.N.

+⁷, ×⁵: Numbers refer to Sensitivity

20
15 — ○ — 5
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No EM1										3 of 3		METRIC					
G.W.P. 280-99-00			LOCATION Hwy 406/ East Main Street Overpass Co-ords: 4 761 572 N; 327 544 E			ORIGINATED BY M.R.											
DIST CR HWY 406			BOREHOLE TYPE C.F.H.S.A. + NX Diamond Corings			COMPILED BY A.S.											
DATUM Geodetic			DATE October 01, 2008			CHECKED BY C.N.											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT <div style="text-align: right;">  </div>					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%) w_p w w_L				
143.9																	
	* 2008 10 01  Water level observed during drilling  Penetrometer test C.F.H.S.A: denotes Continuous Flight Hollow Stem Augers ** Cored thru cobbles and gravel from 21.3 to 24.5m depth, no recovery Noticed bubbles in water at top of casing during rock coring (probable methane gas).																

RECORD OF BOREHOLE No EM2

1 of 2

METRIC

G.W.P. 280-99-00 LOCATION Hwy 406/ East Main Street Overpass
DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Hollow Stem Augers
DATUM Geodetic DATE October 16, 2008

ORIGINATED BY W.L.
COMPILED BY A.S.
CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
171.4	Ground Surface																
170.9	Topsoil		1	SS	7		171										
0.2	Clayey silt some sand, trace gravel																
	Firm Reddish Moist to soft brown		2	SS	8												2 12 52 34
	(TILL)						170										
			3	SS	3												
							169										
	trace sand																
	Stiff to very stiff		4	SS	12		168										0 7 45 48
							167										
			5	SS	25												
							166										
	cobbles and boulders																
			6	SS	34		165										
							164										
	Hard		7	SS	50/13cm												
							163										
	with sand		8	SS	50/13cm		162										8 29 47 16
							161										
			9	SS	50/10cm												
							160										
			10	SS	94/25cm		159										
	some sand						158										
			11	SS	50												
							157										5 18 55 22
156.4																	

RECORD OF BOREHOLE No EM2

2 of 2

METRIC

G.W.P. 280-99-00 LOCATION Hwy 406/ East Main Street Overpass
 DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Hollow Stem Augers
 DATUM Geodetic DATE October 16, 2008

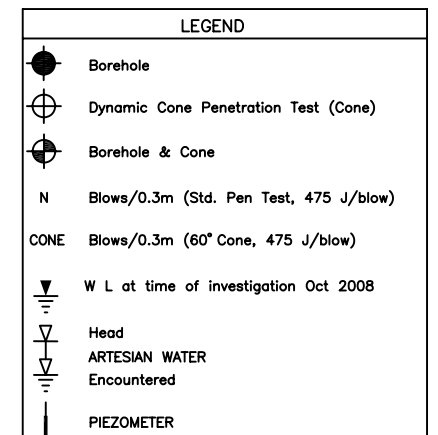
ORIGINATED BY W.L.
 COMPILED BY A.S.
 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa												
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
156.4							20	40	60	80	100									
15.0	Clayey silt some sand, trace gravel cobbles and boulders		12	SS	50/15cm															
	Hard Reddish Moist brown																			
	(Cont'd)																			
	(TILL)																			
				13	SS	50/10cm														
	— Grey — — —																			
			14	SS	69/25cm															
										</										

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



Peto MacCallum Ltd
CONSULTING ENGINEERS



- NOTE -

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

Geocres No. 30M03-238

Geocres No. 30M03-238					
HWY No 406				DIST CENTRAL	
SUBM'D AS		CHECKED CN	DATE JAN. 09, 2009		SITE --
DRAWN NA		CHECKED CN	APPROVED BRG		DWG EM-1

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
2. LOCATION OF STRUCTURES WERE INFERRED FROM MRC PRELIMINARY PLANS.



Geocres No. 30M03-238					
HWY No 406				DIST CENTRAL	
SUBM'D AS		CHECKED CN	DATE JAN. 09, 2009		SITE --
DRAWN NA		CHECKED CN	APPROVED BRG		DWG EM-1



APPENDIX A

Site Photograph



Photograph 1: Viewing towards east at centreline of East Main Street. Note drilling rig setting up on Borehole EM2. West entrance of East Main Street tunnel under the Welland Canal is also shown in photo. (October 16, 2008)



APPENDIX B

Rock Core Photograph



Photograph 1: Rock Core from Borehole EM1 - RC-18 to RC-21