Peto MacCallum Ltd.
CONSULTING ENGINEERS

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

REPLACEMENT OF MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11, SITE NO. 39W-033
GWP 154-98-00
TOWN OF HEARST, DISTRICT OF NEW LISKEARD

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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1. INTRODUCTION

This report summarizes the results of the preliminary foundation investigation carried out for the reconstruction of the Mattawishkwia Bridge on Highway 11 in the east outskirts of the Town of Hearst, Ontario. Peto MacCallum Ltd. (PML) conducted the preliminary foundation investigation for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The proposed new bridge will carry the Highway 11 eastbound and westbound traffic over the Mattawishkwia River. The Ontario Northland Railway (ONR) crossing the Mattawishkwia River runs parallel and about 20 m south of Highway 11.

The data from a 1982 report for previous bridge repairs at the site was used for the current investigation (Ref. "Foundation Investigation Report for Mattawishkwia River Bridge" dated March 1982, GEOCRES No. 42G-016, Contract No. 82-213).

This preliminary report pertains to the bridge structure and approach embankments and is considered to be suitable for planning and preliminary design purposes and should not be used for detail design.

2. SITE DESCRIPTION AND GEOLOGY

The site is located at the existing crossing of the Mattawishkwia River Bridge on Highway 11 in the Town of Hearst, Ontario.

Land use in the vicinity of the site comprises the existing transportation corridor of Highway 11 and (ONR) which is running parallel and south of Highway 11. The local topography of the structure site is generally flat with a gentle down gradient to the north. In the immediate vicinity of the site the land is used mainly for industrial and commercial purposes with some residential housing.

According to the 1982 report, the Mattawishkwia River at the structure site is relatively slow flowing and approximately 60 m in width and 1.5 m in depth. The stream bed is covered with scattered large boulders. The river banks at some locations are up to 6 m in height and near vertical.

The existing bridge over the Mattawishkwia River is a continuous, four span reinforced concrete 'T' beam superstructure with an asphalt wearing surface. According to the original structural plans, the supporting concrete piers are founded on spread footings. Both end spans are cantilevered, therefore, no abutments are present.

The east and west piers have tilted toward the river, whereas the centre pier shows no sign of movement. According to field observations described in the Request for Proposals for the project, the east pier has a tilt of 1 horizontal to 18 vertical (1H:18V) with the top of the pier exhibiting a forward movement of about 380 mm. The west pier is tilted at 1H:30V with a movement of about 178 mm at the top. Cracks of up to 19 mm wide were observed in the diaphragms between the 'T' beams and pier.

The east approach fill beneath the cantilevered end span includes a gabion wall which was originally installed away from the pier but has slid forward against the pier and are buckling upward near the centre of the slope. Approximately 2.7 m of fill is being retained by the east pier.

According to data in the 1982 report, sheetpile-and-tieback systems were installed at both ends of the bridge. The sheetpiles were driven to elevations 229.6 and 230.7 m at east and west ends, respectively. Deadman anchors were provided at both ends about 18 m away from the sheetpiles.

The surface of the west approach fill includes a mixture of sand, gravel and clayey silt with the occasional boulder. The height of fill behind the west pier is about 2.7 m when measured from the stream bed.

The study area site is located within the Abitibi Uplands, part of the Canadian Shield physiographic region. The soil cover in the region typically comprises ground moraine of varying thicknesses. The soils generally comprise Cochrane Till which is described as a non-sorted, non-stratified silty clay, silty clay loam or silt loam. This deposit locally contains lenses, clasts or blocks of clay and silt and granular materials, cobbles and boulders. (Sado, E.V. Fullerton, D.S., and Farrand, W.R., Quaternary Geologic Map of the Lake Nipigon 4º x 6º Quadrangle, USA and Canada, 1994).

3. INVESTIGATION PROCEDURES

The field work was carried out in November 2008. Sixteen sampled boreholes were put down at the site. Four boreholes designated 101 through 104 were drilled along the approximate centreline of the highway (alignment 1), six boreholes designated 201 through 206 were drilled about 12 m to the south of the centreline (alignment 2) and six boreholes designated 301 through 306 were drilled 18 to 35 m to the north of the centreline (alignment 3). The boreholes were drilled to practical refusal or to competent ground at depths of 4.7 and 10.2 m. Locally, one borehole was drilled to 0.6 m, where auger refusal was met on probable cobbles and boulders. The borehole locations are shown on the attached Drawing 1.

The locations of boreholes were selected by PML and the ground surface elevations at the boreholes in the field were surveyed by Stantec. Some of the boreholes were relocated due to interference from overhead and underground utilities or drilling access constraints.

The boreholes were advanced using continuous flight hollow stem augers and NW washboring, 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. Rotary diamond drilling was also utilized to advance the boreholes through cobbles and boulders.

Representative soil samples were recovered using a split spoon sampler in conjunction with standard penetration tests. 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.

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

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

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, soil classification and laboratory testing. The laboratory testing program comprised the following tests:

- Natural moisture content determinations (88)
- Grain size analyses (25)
- Atterberg limits (3)

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-1 to GS-6. The Atterberg limits results are presented on Figures PC-1 and PC-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 strata and groundwater observations.

The borehole locations and the plan of the existing Mattawishkwia River Bridge are presented on the attached Foundation Drawing 1.

The subsurface stratigraphy revealed in the boreholes generally consisted of the typically 1500 mm thick pavement fill of the Highway 11 or 200 to 600 mm topsoil elsewhere, over 0.3 to 5.3 m thick fill units which were placed over 2.6 to 7.6 m thick compact to very dense heterogeneous cohesionless till deposits. A 1.5 to 2.4 m thick firm to very stiff silty clay deposit was found below fill units in boreholes 201, 203 and 301. A 0.4 m thick layer of cobbles and boulders was contacted in borehole 205 at depth of 0.2 m, extended to 0.6 m depth where auger refusal was encountered. Cobbles and boulders were also encountered in the lower zone of the till in boreholes 303 and 304. The soil conditions encountered in the boreholes previously drilled for the bridge site (March 1982 report) are consistent with those encountered in the boreholes recently drilled for this investigation.

4.2 Topsoil

Surficial topsoil layers 200 to 600 mm thick were encountered in boreholes 201, 202, 203, 204, 205, and 303 through 306. The topsoil extended to depths ranging from 0.2 to 0.6 m, elevations 232.3 to 237.3 m. One N value of 1 was obtained.

4.3 Pavement Fill

Asphaltic concrete of 100 mm thickness was found in boreholes 101, 102, 103, 104 and 206 overlying 1400 mm thick granular base materials. This pavement structure extended to 1.5 m in depth, elevations 235.7 to 235.8 m. The granular base contains sand trace to some gravel some silt. Cobbles were mixed with the pavement fill matrix in borehole 104.

The laboratory moisture content of the pavement fill was 3 and 8%. The grain size distribution charts of representative samples of granular base are shown on Figure GS-1.

4.4 Fill

A 1.2 to 5.3 m thick fill layer was encountered at a uniform to 1.5 m depth, elevations 235.7 to 235.8 m, beneath the pavement fill in boreholes 101, 102, 103, 104 and 206. Below the topsoil in boreholes 201, 202, 203, 204, 303, 304, 305 and 306, the fill was encountered at depths ranging from 0.2 to 0.6 m and extended to depths ranging from 0.6 to 2.1 m, elevations 230.4 and 234.8 m. Fill was found surficially in boreholes 301 and 302 and extended to 0.6 and 1.5 m depths, elevations 235.1 to 235.5 m. The fill units were heterogeneous and included sand, silty sand, sand and gravel, sandy silt, silty clay and clayey silt. N values typically ranged from 3 to 14 with one localized higher N value of 35 in borehole 102.

The grain size distribution charts of two silty clay fill samples are presented in Figure GS-2. The Atterberg plasticity limits are shown in Figure PC-1. The liquid limits of the samples were 36 and 43 and the plastic limits were 18 and 22, with plasticity index values of 18 and 21. Moisture contents determined on the fill samples typically ranged from 4 to 30% with a single higher value of 86% which was obtained on a silt sample containing organics from borehole 103.

4.5 Silty Clay

Beneath the fill units at 0.6 to 1.5 m depths (elevations 234.2 to 236.9 m), a discontinuous 1.5 to 2.4 m thick firm to very stiff silty clay layer was encountered in boreholes 201, 203 and 301. The silty clay deposit extended to an underlying silt till at depths of 2.3 to 3.0 m (elevations 232.5 to 234.5 m). Penetrometer test values obtained ranged from 36 to 138 KPa. N values ranged between 5 and 11.

The charts from grain size distribution and Atterberg limits testing conducted on a silty clay sample are presented in Figures GS-3 and PC-2, respectively. The Atterberg liquid and plastic limits were 40 and 20, respectively, with a plasticity index of 20. The moisture content of the representative sample was 33%.

4.6 Silt Till / Silt and Sand Till

Beneath the fill units at depth of 0.6 to 6.8 m (elevations 230.4 to 236.9 m) in boreholes 101, 102, 103, 104, 203, 206 and 302 to 306 and below the silty clay deposit at 2.3 to 3.0 m depths (elevations 232.5 to 234.5 m) in boreholes 201, 203 and 301, a 2.5 and 7.5 m thick silt till / silt and sand till was encountered. The silt till / silt and sand till extended to the borehole termination depths of 3.7 to 9.8 m (elevations 226.1 to 231.0 m). The deposits contain layers of various soil types including clay/silty clay/clayey silt, sandy silt, sand with gravel and silty sand and also contained scattered cobbles and boulders.

The silt/silt and sand till was typically loose to dense within the upper zones becoming very dense with depth. N values varied from 10 to 30 blows for 50 mm penetration. Locally, a low N value of 4 was recorded in borehole 303 within the upper zone of the deposit.

The water content of the silt/silt and sand till ranged between 8 and 23%, typically in range of 11 to 21. A single high moisture content of 34% was recorded in borehole 201 at depth of 6.2 m, elevation 231.3 m. The grain size distribution chart of a representative sample of the silt/silt and sand till is shown on Figure GS-4. The silt sample was non-plastic according to an Atterberg limit determination and visual examination.

4.7 Sand/Silty Sand Till

A deposit of compact to very dense sand/silty sand till was encountered in boreholes 103, 304 and 306 below the silt/silt and sand till at depths of 3.8 and 9.6 m (elevations 227.7 and 229.6 m) and extending to the borehole termination depths of 7.8 to 10.2 m at elevations 224.8 and 227.1 m. The deposit contains trace clay trace to with silt with gravel, and cobbles and boulders. The penetrated thickness of the deposit was about 0.6 and 4.1 m. The N values in the deposits ranged from 25 to 50 blows for 50 mm penetration.

The water content of the sand/silty sand till ranged between 11 and 16%, indicating wet conditions. The grain size distribution charts of representative samples of the sand/silty sand are shown on Figure GS-5. The sand/silty sand samples were non-plastic according to an Atterberg limits determination and visual examination.

4.8 Sand and Gravel Till

A deposit of dense to very dense sand and gravel till was encountered in borehole 204 below the fill unit at a depth of 2.1 m (elevation 230.4 m). The sand and gravel deposit contains trace clay with silt trace gravel and cobbles and boulders. The thickness of the deposit was 2.6 m extending to the 4.7 m termination depth of the borehole elevation 227.8 m. The N values in the deposit ranged from 41 to 30 blows for 50 mm penetration.

The grain size distribution chart of representative samples of the sand and gravel is shown on Figure GS-6.

4.9 Cobbles and Boulders

In borehole 205, a 0.4 m thick layer of cobbles and boulders was contacted below topsoil at 0.2 m depth, elevation 233.2 m, and extended to the borehole termination depth of 0.6 m, elevation 232.8 m, where augering was terminated. These obstructions could be part of the fill for the existing ONR and Highway 11 common embankments. In view of the limited access and winter conditions at the time of the investigation, the origin of this deposit could not be verified. Cobbles and boulders were also encountered in the lower zone of the till in boreholes 303 and 304.

5. GROUNDWATER

Water level was observed during drilling in boreholes 102, 201, 202, 204, 301 through 306 at depths of 0.6 to 4.6 m (elevations 231.9 and 235.1 m). Upon completion of drilling groundwater was measured in boreholes 201, 304 and 306 at depths of 2.1 to 7.6 (elevations 228.0 to 230.6 m). The groundwater table was not determined upon completion of drilling in the remaining boreholes.

The water level in the Mattawishkwia River was at elevation 231.5 m on November 14, 2008.

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

6. MISCELLANEOUS

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

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
Replacement of Mattawishkwia River Bridge
Highway 11, Site No. 39W-033
GWP 154-98-00
Town of Hearst, District of New Liskeard

7. ENGINEERING RECOMMENDATIONS

7.1 General

Part B of this report provides the preliminary foundation engineering recommendations regarding design and comments for replacement of the Mattawishkwia River Bridge on the Highway 11 in the Town of Hearst, Ontario. Comments concerning the design of foundations for a new temporary bridge (detour structure) to carry traffic over the river during construction of the replacement bridge are also provided for possible alignments located north and south of the existing bridge. The recommendations are preliminary and based on the results of the preliminary subsurface investigation that was outlined in the Part A of this report.

The road grade on Highway 11 at the existing about 72 m long and 10 m wide bridge location is near elevation 237.3 m as surveyed by Stantec. The water level in the river was at elevation 231.5 m at the time of the investigation. The existing approach fill embankments are about 5.8 m high above the water level in the river.

Further to the assessments carried out by Stantec and MTO (plans dated April 22, 2009) the following construction alternatives were considered for the bridge replacement.

Alternative 1a: New bridge on existing alignment with single lane detour 16.0 m north of the existing bridge.

Alternative 1b: New bridge on existing alignment, single lane detour 16.0 m south of the existing bridge.

Alternative 2a: New bridge on existing alignment with two-lane detour 16.0 m north of the existing bridge.

Alternative 2b: New bridge on existing alignment with two-lane detour 16.0 m south of the existing bridge.

Alternative 3a: Staged construction with single lane traffic during construction, new bridge on new alignment located 1.4 m north of the existing bridge.

Alternative 3b: Staged construction with single lane traffic during construction, new bridge on new alignment located 1.4 m south of the existing bridge.

Alternative 4: Replacement bridge on new alignment located 16.0 m north of the existing bridge (no detour).

Alternatives 3a or 3b require constructing the replacement bridge on new alignment 1.4 m north or south of the existing alignment. Consequently, their stage 1 involves the removal of a longitudinal portion of the existing bridge and construction of that respective portion of the new bridge while the bridge immediately adjacent to the remaining portion of the existing structure remains in place for temporary vehicular traffic. The stage 2 includes the demolition of the remaining portion of the old bridge and completing the balance of the new bridge.

From the Foundations perspective, the seven listed alternatives result in essentially three possible bridge locations. These are for the new and/or detour bridges on the existing bridge alignment (existing location), to the south of the existing bridge (south location) or to the north of the existing bridge (north location).

Use of conventional procedures to design and construct the replacement or detour bridges on deep or shallow type foundations is considered to be feasible. The use of shallow foundations is considered to be preferred for the temporary detour structure which likely will consist of a modular bridge.

For the replacement bridge, it is anticipated that the piles will find refusal in the very dense cohesionless till soils at both the west and east sides of the river. The geotechnical resistance of the deep foundations should be designed for bearing on the very dense layer at both abutments for preliminary design purposes. The piles should be provided with driving shoes due to potentially heavy driving through the glacial till which contains cobbles and boulders.

The staged construction alternative will have to consider the constraints from the existing sheetpiling and deadman anchors at each abutment. Construction of temporary or permanent approach embankments is considered to be feasible over the typically cohesionless soils at the site. The existing ONR bridge and embankments are only about 20 m south of existing embankments and will require careful considerations and may render impractical the construction of permanent approach embankments at this location.

If alternatives 3a or 3b are selected, precautions should be taken where construction of the replacement bridge requires excavation and pile driving operations near the existing piers and sheetpiles, or when demolishing the existing bridge. Settlement and displacement of existing piers and sheetpiles and/or embankment could occur due to excessive vibration or ground loss. Therefore, settlement and displacement monitoring system should be provided during construction duration.

The water level in the river was at elevation 231.5 m at the time of the field investigation. The groundwater levels measured in the boreholes ranged from elevations 228.0 to 230.6 m however it is considered that the water level in the river will govern the water level during construction.

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 investigations will be required at the selected bridge and detour structure locations 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.

7.2 Foundations

7.2.1 General

The detour structure will likely be a modular steel truss bridge type. The founding of the detour structure on deep foundation such as driven steel H-piles driven to practical refusal on the very dense cohesionless soils is considered to be feasible. Spread footings placed on the native stiff soils or on engineered fill pads should also be adequate for the temporary bridge.

Based on the preliminary data, founding the proposed replacement bridge on piles driven to practical refusal on the very dense cohesionless till soils is considered to be feasible. Pile driving for the replacement bridge near the existing sheet piles and piers (alternatives 3a and 3b) may cause displacement and settlement to the existing sheetpiles and piers. Spread footings placed on the native stiff silty clay (west side) and cohesionless till soils (east side) or on structural fill may be used for semi-integral or conventional abutment design.

Drilled caissons bearing on the native soils to support the detour or permanent structures are not considered to be practical due to the presence of cobbles and boulders in the native soils, as well as groundwater presence above the expected founding levels for caissons.

A discussion of the advantages, disadvantages, costs and risks/consequences of the possible foundation options is presented in Section 7.4 of this report.

Footings and pile caps should be provided with at least 2.6 m of earth cover or equivalent thermal insulation for foundation against frost protection. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Erosion protection measures, such as rip rap or placing the footings below the anticipated scour depth should be considered.

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].

7.2.2 Deep Foundations

7.2.2.1 General

As indicated previously, conventional, integral or semi-integral abutment designs using steel H-piles driven to refusal into the glacial till deposits are considered feasible for replacement or detour bridges at the site. The piles may reach adequate refusal in the layers of cobbles and boulders which are part of the local matrix of the glacial till soils.

The preliminary pile foundation design recommendations for conventional and semi-integral abutments are provided in the following section followed by additional recommendations for integral abutment foundations.

7.2.2.2 Conventional and Semi-Integral Abutments

Piles for the east and west abutments at the bridges for the replacement and detour alternatives should be driven to refusal into the very dense glacial till. The estimated preliminary founding reference levels for driven piles for abutment foundations at the three possible bridge locations are provided in the following table:

ALTER-NATIVE NO.	BRIDGE LOCATION	FOUNDING ELEMENT	REFERENCE BOREHOLE NO.	FOUNDING MATERIAL	FOUNDING DEPTH (m)	STRATUM FOUNDING ELEVATION (m)	ESTIMATED PILE TIP ELEVATION (m)
1a, 1b, 2a, 2b, 3a, 3b (New Bridge)	Existing alignment	West abutment	1, 2, 5, 101 and 102	Silt till	6.1	228.4 to 231.2	226.0
		East abutment	3, 4, 103 and 104	Sandy silt till, silt and sand till	6.8 and 7.5	228.6 to 231.2	226.5 and 228.5
1b, 2b (Detour Bridge)	South of existing alignment	West abutment	202 and 203	Silt till	3.7 and 4.3	231.1	229.0
		East abutment	204	Sand and gravel till	3.7	228.8	226.5
1a, 2a (Detour Bridge	North of existing alignment	West abutment	301 and 303	Silt till and silt and sand till	6.0	229.3 and 231.0	227.0 and 229.0
4 (New Bridge)		East Abutment	304	Sand till silt and sand till	3.8 and 6.0	228.1 and 228.9	226.0 and 229.0

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 depth below these levels should be allowed for pile embedment at refusal.

The piles will have to be driven through fill units and native soils containing typically cohesionless soils and locally a layer of firm to very stiff silty clay (west side) at the abutment locations. For alignments 2 and 3, the existing grade at both abutments will be raised about 4 to 5 m unless the detour bridge is constructed at a lower level than the existing bridge. It is considered, however that the development of negative skin friction on the piles will be minimal due to the relatively thin and very stiff cohesive soil strata. Consequently, negative skin friction will not affect the axial resistance at ultimate limit states (ULS) of the abutment piles. No negative skin friction needs to be considered for bridges constructed on existing alignment since the embankment height will not be raised.

Based on very dense glacial till soils at the pile tips at the both abutments, the preliminary factored geotechnical axial resistance at ultimate limit states (ULS) and geotechnical axial resistance at serviceability limit states (SLS) for the pile sections listed below are recommended.

Pile Section	ULS, kN	SLS, kN
HP 310 x 110	1600	1150
HP 360 x 108	1600	1150

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium.

The piles will set in a very dense glacial till at the abutments and should be equipped with driving shoes according to OPSD-3000.201 or with Titus H-Bearing Pile Standard Model according to SP 903S01. The driving shoes should be used to minimize the potential for damage when driving through the very dense glacial till containing cobbles and boulders.

Pile caps should be provided with at least 2.6 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.

7.2.2.3 Integral Abutment Considerations

For the integral abutment design, the H-piles should be driven to the very dense glacial till anticipated at the depths/elevations and be designed using the axial resistance indicated in the previous section 7.2.2.2 of this report. 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.

7.2.2.4 Bridge Piers

Subsurface data is not available for the pier foundation and should be obtained for detail design. It is considered that the new pier will be adequately founded on spread footings placed on the native soils under the river bed, under similar conditions as the existing piers. The footing depth should allow for the depth of potential scour in the river bed.

7.2.2.5 Lateral Resistances

The soil adjacent to the upper section of the piles is expected to comprise the compacted approach fill and native cohesionless compact to very dense till units. At the west abutments 1.5 to 1.7 m thick cohesive firm to very stiff native cohesive soils is locally present 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 in Section 7.2.2.3 previously should be considered. The assessed lateral passive resistance for the HP pile sections noted previously is as follows.

	SILTY CLAY				**NATIVE TILL/GRANULAR ‘A’ OR ‘B’ TYPE II**			
	FIRM TO STIFF		**VERY STIFF**		**COMPACT TO DENSE**		**VERY DENSE**	
	HP310	HP360	HP310	HP360	HP310	HP360	HP310	HP360
Factored Lateral Resistance at ULS, kN	140	140	200	240	110	150	120	170
Lateral Resistance at SLS, kN	50	60	110	140	40	50	50	70

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

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

Cohesionless Soils (Terzaghi, 1955)

k_s = n_h z/b
where n_h = coefficient related to soil density
= 10.0 MN/m^3 for granular backfill and native cohesionless till
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 silty clay soils should be taken as 25,000 kN/m^3 for preliminary design purposes.

7.2.3 Shallow Foundations

7.2.3.1 Spread Footings on Native Soil

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

A minimum of 2.6 m of earth cover for frost protection will be required for the spread footing foundations. Assuming that the pavement will be constructed at the bridge deck elevation 237.3 m, the highest founding level will be at about elevation 234.7 m for foundation frost protection purposes. The reference founding levels for each of the three possible bridge locations are provided in the Table below:

ALIGNMENT NO.	BRIDGE LOCATION	FOUNDING ELEMENT	REFERENCE BOREHOLE NO.	FOUNDING MATERIAL	SPREAD FOOTINGS	
					FOUNDING DEPTH (m)	FOUNDING ELEVATION (m)
1A, 1B, 2A, 2B, 3A, 3B (New Bridge)	Existing alignment	West abutment	1, 2, 5, 101 and 102	Silt till	0.9 to 4.7	230.6 to 234.6
		East abutment	3, 4, 103 and 104	Silt till, silt and sand till	1.7 to 6.8	230.5 to 232.5

ALIGNMENT NO.	BRIDGE LOCATION	FOUNDING ELEMENT	REFERENCE BOREHOLE NO.	FOUNDING MATERIAL	SPREAD FOOTINGS	
					FOUNDING DEPTH (m)	FOUNDING ELEVATION (m)
1b, 2b (Detour Bridge)	South of existing alignment	West abutment	202 and 203	Silt till, silty clay	0.6 and 0.7	234.2 and 234.7 *
		East abutment	204	Sand and gravel till to silt till	2.1	230.4
1a, 2a (Detour Bridge) 4 (New Bridge)	North of existing alignment	West abutment	301 and 303	Silt till, silty clay	2.3 and 1.6	233.7 and 234.7 *
		East Abutment	304	Silt till	0.6	232.1

Note: (*) Highest level for foundation frost protection.

Based on the borehole data, the founding subgrade will typically include compact to dense silt till and locally firm to stiff silty clay in alternatives 2 and 3 (west abutment). The recommended preliminary geotechnical bearing resistances for spread footings constructed on the native soils is as follows:

	SILTY CLAY		GLACIAL TILL	
	FIRM	VERY STIFF	COMPACT	DENSE
Factored Geotechnical Resistance at ULS, kPa	150	500	500	600
Geotechnical Resistance at SLS, kPa	100	300	300	400

The firm silty clay at the west abutment in Alignment 3 maybe excavated a further 0.7 m below the foundation frost protection depth to about elevation 234.0 m where compact glacial till was encountered to utilize a higher geotechnical bearing 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. A footing embedment depth of 2.0 m and groundwater at the river level (elevation 231.5 m) 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.

The east abutment spread footings constructed on native soil for bridges on the existing alignment or south of the existing alignment (alternatives 1a, 1b, 2a, 2b new bridges and 1b, 2b detour bridges) will be at or below the water level in the river. The construction of these footings below about elevation 232.0 m will likely require cofferdam and dewatering systems. This constraint should be verified during detail design.

As previously indicated, all footings subject to frost action should be provided with 2.6 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.

7.2.3.2 Spread Footings On Structural Fill

Construction of the abutment footings of the replacement bridge and/or detour structure on structural fill placed in the approach embankment could also be employed to support the foundation loads. All existing fill should be removed from the location of the structural fill pad. The structural fill should comprise Granular A material placed in maximum 200 mm thick lifts and compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density.

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

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

A minimum 2.5 m thickness of the structural fill pad was used for the computation of the geotechnical resistances. 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 2.6 m and a groundwater level at elevation 231.5 m was assumed for computation of the ULS resistance.

The construction of the structural fill pads for the east abutments of the bridges on the existing alignment or south of the existing alignment (alternatives 1a, 1b, 2a, 2b new bridges and 1b, 2b detour bridges) will require excavations below the water level in the river. Consequently, the lower zones of the structural fill below about elevation 232.0 m will likely require a cofferdam and dewatering. This constraint should be verified during detail design.

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.

7.3 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 Ø = angle of internal friction of retained soil (35^o for Granular A or Granular B Type II or Type III)
δ = angle of friction between the soil and wall (23.5^o for Granular A or Granular B Type II or Type III)

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

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

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

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

A subdrain system (SP 405F03) should be installed to minimize the build-up of hydrostatic pressure behind the wall. The subdrain pipes 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.

7.4 Comparison of Foundation Alternatives

In view of the site conditions described previously and foregoing considerations, it is considered that the practical or feasible foundation alternatives comprise spread footings on native soil or structural fill and driven piles for the abutments at this site.

A summary of the discussion is provided on the following table:

FOUNDATION ELEMENT	ALTERNATIVE FOUNDATION	NOTES
West and East Abutments	Spread footings on native soils	Feasible where spread footings placed on the native stiff to very stiff soils or compact to dense cohesionless soils
	Spread footing on structural fill	Feasible and practical alternative
	Caisson on native soils	Not feasible due to the presence of cobbles and boulders in the native soils as well as groundwater presence above the expected founding levels
	Driven piles to practical refusal on the very dense cohesionless till soils	Feasible and practical alternative

A comparison of the relative advantages and disadvantages related to each of the foundation alternatives is presented below.

ABUTMENT FOUNDATIONS			
Spread Footings on Native Soils or Footings on Structure Fill			
Advantages	Disadvantages	Relative Costs	Risks/Consequences
• Ease of installation • Lower cost than deep foundations • Reduced height of abutment • May be used with semi-integral abutments	• Construction of structural fill pad requires wider area • Requires construction of a fill pad • Requires placement of mass concrete to provide subgrade above water table • Requires removal of all boulders from foundation footprint (footings on native soil) • Potentially requiring tremie concrete for under water construction	• Typically lower installation costs than for deep foundations	• Potential risk of flooding of excavations for footings causing delays and extra costs • Existing sheetpiling and anchoring system may cause excavation difficulties for new abutments or existing alignment resulting in delays and extra costs
Driven Piles			
Advantages	Disadvantages	Relative Costs	Risks/Consequences
• High bearing resistance • Allows integral abutment design and construction • Lower long-term maintenance costs of deck expansion joints with integral abutment design • Negligible settlements of foundation	• Requires granular fill pads to facilitate the pile driving • Potential boulders may cause difficulties • Heavy equipment for pile driving is required	• Higher installation cost than spread footings	• Pile damage due to presence of cobbles and boulders may require the driving of additional/replacement piles and extra costs • Pile driving for new abutments on existing alignment may interfere with existing sheetpile support system and require field adjustments or additional structural work resulting in added costs
Caisson on Bedrock			
Advantages	Disadvantages	Relative Costs	Risks/Consequences
• Not feasible	• Not feasible	• Not feasible	• Not feasible

Conventional, semi-integral and integral abutments are considered feasible at this site. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundation engineering perspective, use of piles driven to practical refusal for integral abutment is the preferred type of foundation.

7.5 Approach Embankments

Based on the acquired data, the new approach embankments in alternatives 1a, 1b, 2a, 2b or 4 or the possible embankment widening in alternatives 3a and 3b will be placed on the compact to very dense cohesionless till or, locally at the west abutment locations, on the firm to very stiff cohesive soils underlain by compact to very dense cohesionless till.

The approach embankments should be designed and constructed in accordance with OPSD-200.010, 201.010, 202.010, 3101.150 and SP 206S03. Embankment widening should be constructed following OPSD-203.030. 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, based on the encountered competent founding soils.

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.

No additional settlement of the existing embankment is expected for the alternatives 3a or 3b construction assuming that no new fill is placed to raise the grade of the existing roadway or widening of the embankment is required.

The settlements for detour and permanent new embankments utilized in alternatives 1a, 1b, 2a, 2b and/or 4 are estimated to be relatively small and will occur rapidly during construction. It is recommended that both approach embankment fills be placed prior to driving the abutments piles. Further subsurface investigation and laboratory tests should be carried out during detail design for this analysis.

The backfill to the structure should be made of granular materials. The magnitude of the "consolidation" of these fills depends on the height of the embankment, 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 10 to 12 mm for 4 to 5 m high embankments. 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.

Rockfill could be used for construction of the embankments along the detour and permanent embankments.

It is estimated, based on the limited laboratory test results that the new fill for the potential widening of the existing embankment will cause negligible consolidation settlement of the existing embankment. No new settlement of the existing embankment is expected where the detour embankments are constructed about 10 m away.

The construction of the detour or replacement bridges for alternatives 1a, 1b, 2a, 2b or 4 would require construction of new embankments or widening of the existing embankment into the water or construction of a longer bridge than in alternatives 3a or 3b.

7.6 Construction Considerations

7.6.1 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.

Excavation for construction of foundations of the detour structure and the replacement bridge is expected to extend through the fill and into the underlying silt till and locally silty clay soils. The depth of excavation will be established at the detailed design stage. The fill, cohesive firm to stiff silty clay and compact to dense cohesionless till encountered in the boreholes are considered Type 3 soils and very dense till is considered a Type 2 soil according to OHSA (Ontario Regulation 213/91) criteria.

The stabilized groundwater level is expected to be consistent with the water level in Mattawishkwia River, which was at elevation 231.5 m at the time of the investigation. Considering that the silty and sandy soils on site are relatively pervious, conventional sump pumping techniques are unlikely to be adequately handle groundwater seepage if the excavation extends below the water level in the river. It is anticipated that a cofferdam using the steel sheeting will be required for these excavations. Groundwater conditions should be further assessed during the detail design stage.

7.7 Overview of Alternatives

An overview of the advantages, disadvantages, costs and risks/consequences of the various alternative foundation schemes considered in this report are presented in the attached Table 3.

Considering the advantages, disadvantages, costs and risks/consequences of each alternative provided in Table 3, the staged construction of the existing bridge on new alignment 1.4 m to the north (alternative 3a) is considered to be the most favourable alternative with many advantages, and less disadvantages and lowest cost (similar to alternative 3b). The cost is anticipated to be lower than other alternatives due to shorter length of the bridge and reuse of the existing embankment.

The issue of construction near the existing foundations should be addressed by providing a monitoring survey during construction. The monitoring program should specify the frequency of survey readings to record movements of the existing structure. These readings should be carried out at the minimum interval of three times daily and the frequency should be increased when vibrations such as pile driving, heavy excavation or removal of support systems are carried out. Alert levels should be specified by the structural designer who should also review and approve the monitoring survey scheme.

8. ADDITIONAL STUDIES

The recommendations in this report are preliminary and are based on PML's interpretation of the factual information obtained from boreholes and a visual site assessment.

Foundation investigations will be required at the specific locations of the replacement and detour structures during the detail design phase of the project. The detail design foundation investigations should include drilling boreholes at the final abutment and pier locations. Where the construction of the replacement structure includes staged construction involving the existing bridge, slope stability analyses should be considered for the removal of the existing sheet piles and deadman anchor systems. Existing data should be incorporated in the detail design.

9. CLOSURE

This Preliminary Foundation Investigation and Design Report was prepared by Mr. Idib (Adeeb) Sadoun, MSc, P.Eng., Project Engineer and by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. 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.

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TABLE 1
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 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-203.030	Embankments Over Swamp - Existing Slopes Maintained
OPSD-3000.201	Oslo Points for Foundation, Piles, Steel HP310
OPSD-3101.150	Minimum Granular Backfill Requirements - Walls Abutment

TABLE 2
GRADATION SPECIFICATIONS 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.

TABLE 3

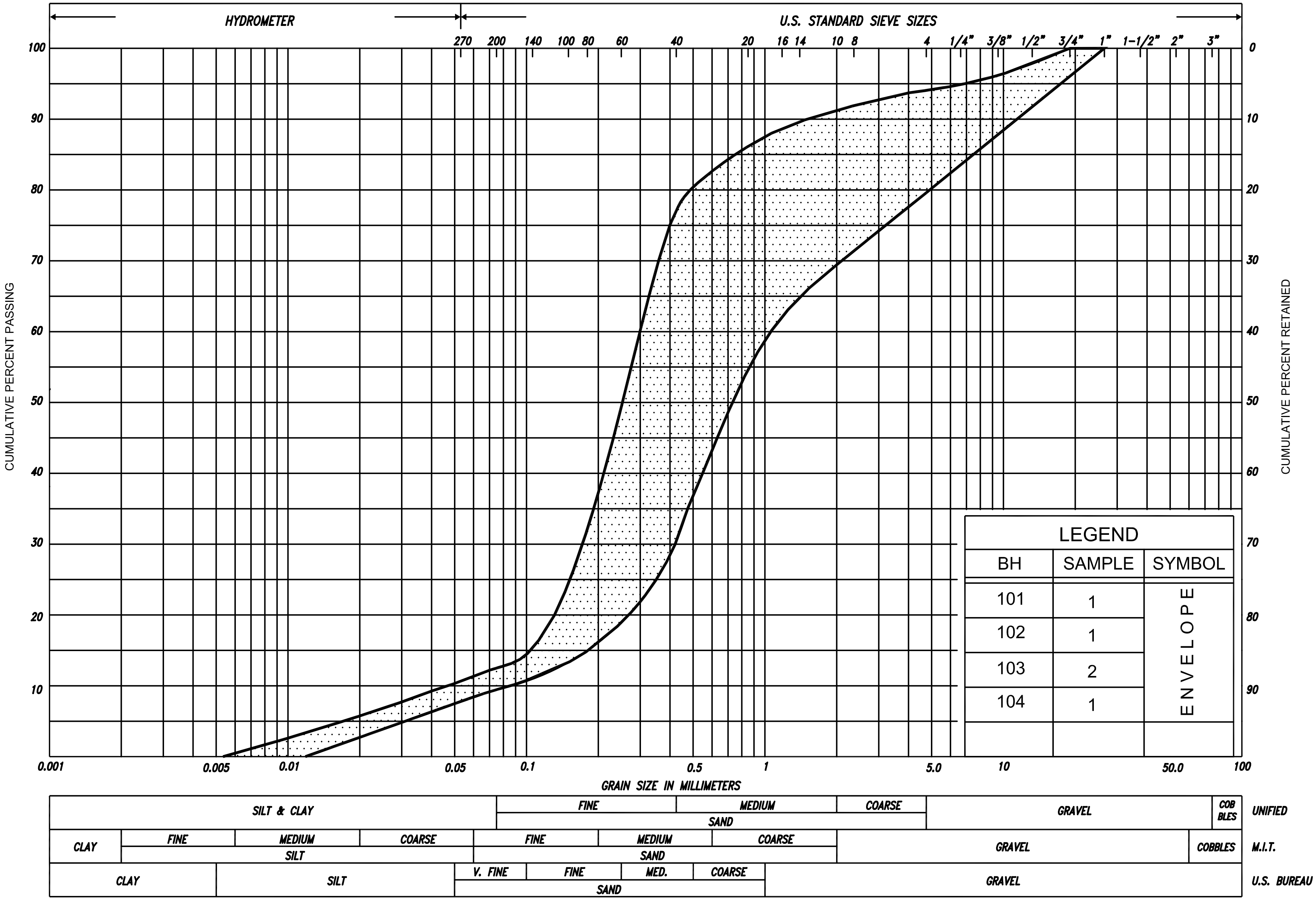
OVERVIEW OF ALTERNATIVES

Alternative No.	Advantages	Disadvantages	Relative Costs	Risks/Consequences	Rank
1a Replacement on the existing alignment with single-lane detour location on the north side - - - - - - 2a Same as 1a with two-lane detour	• The existing embankment is re-used for the replacement bridge. • No long-term lane closure is required • No settlements of the approach embankments • Removal of existing sheet pile behind each abutment is facilitated • No interference with ONR embankment or bridge	• Detour structure and approach embankments are required • Detour structure on this alignment is the longest and may require piers in the river • Allce Road and Riverside Drive detouring or temporary closure is required	• Higher costs than the staged construction alternatives due to detour construction and removal and Allce Road and Riverside Drive detouring • Estimated cost (by Stantec): - 1a: $7,823,000 - 2a: $7,942,000	• Piers in the water for long detour bridge may be affected by ice unless constructed to resist these forces. The consequence would be failure of the detour bridge	4 - - - 5
1b Replacement on the existing alignment with single-lane detour location on the south side - - - - - - 2b Same as 1b with two-lane detour	• The existing embankment is re-used for the replacement bridge • No long-term lane closure is required • No settlements of the approach embankments • Removal of existing sheet pile behind each abutment is facilitated	• Detour structure and approach embankments are required • Construction of detour may interfere with ONR embankment • Installation and removal of new sheetpiles are required at south toe of the slope for spread footings for detour abutment construction • Settlement and displacement monitoring systems are required during and after construction of the detour structure to assess the stability of ONR foundations and embankment	• Higher costs than the staged construction alternatives due to detour construction and removal and monitoring system • Estimated cost (by Stantec): - 1b: $7,467,000 - 2b: $7,916,000	• Driving piles at the abutment location for the detour structure and filling over existing railway embankment may affect the existing railroad tracks. The consequence will be increased maintenance of the railway tracks	6 - - - 7

TABLE 3

OVERVIEW OF ALTERNATIVES

Alternative No.	Advantages	Disadvantages	Relative Costs	Risks/Consequences	Rank
3a Staged construction of the existing bridge (no detour) 1.4 m north	• No detour is required • Removal of existing piers and existing sheetpiles may not be required • Replacement structure is shorter than alternatives 1a, 1b, 2a, 2b and 4 • No temporary detour or closure of Allce Road and Riverside Drive is required	• Long-term closure is required for one lane • Settlement and displacement monitoring system of the existing bridge remaining during construction of stage 1 is required • Installation and removal of new temporary sheetpiles is required	• Cost savings for not removing the existing pier footings and reusing the existing embankment will be offset by increased costs due to installation and removal of new sheetpiles, and a monitoring system requirement • Estimated cost (by Stantec): - 3a: $6,444,000 - 3b: $6,428,000	• Movement of the existing piers and of the existing embankment during removal of existing sheetpiles and deadman anchors for stage 1. Consequences would be failure of the part of the bridge remaining during stage 1	1
3b Same as 3a 1.4 m south					2
4 Replacement on new alignment north of the existing bridge	• No detour is required • No long-term lane closure is required • No interference with ONR embankment or bridge	• New approach embankment is required • Replacement structure on this alignment is the longest • Allce Road and Riverside Drive detouring or temporary closure is required	• Higher costs than the staged construction alternatives due to longer bridge requirement, installation of new sheet piles and a new embankment construction • Estimated cost (by Stantec): $7,482,000	• No perceived risks	3



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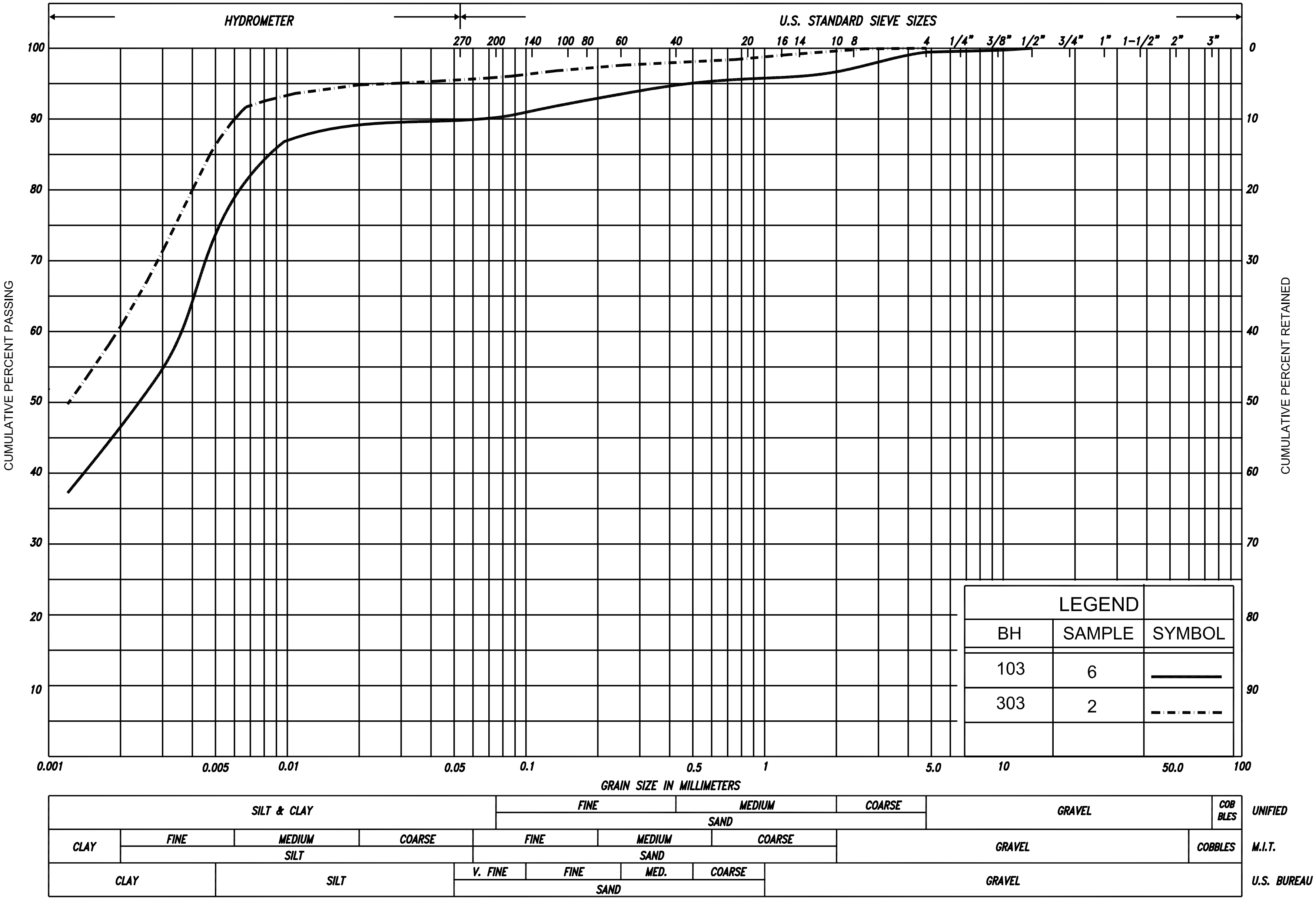
GRAIN SIZE DISTRIBUTION

SAND, trace to some silt, trace to some gravel
(FILL)

FIG No.	GS-1
HWY:	11
G.W.P. No.	154-98-00

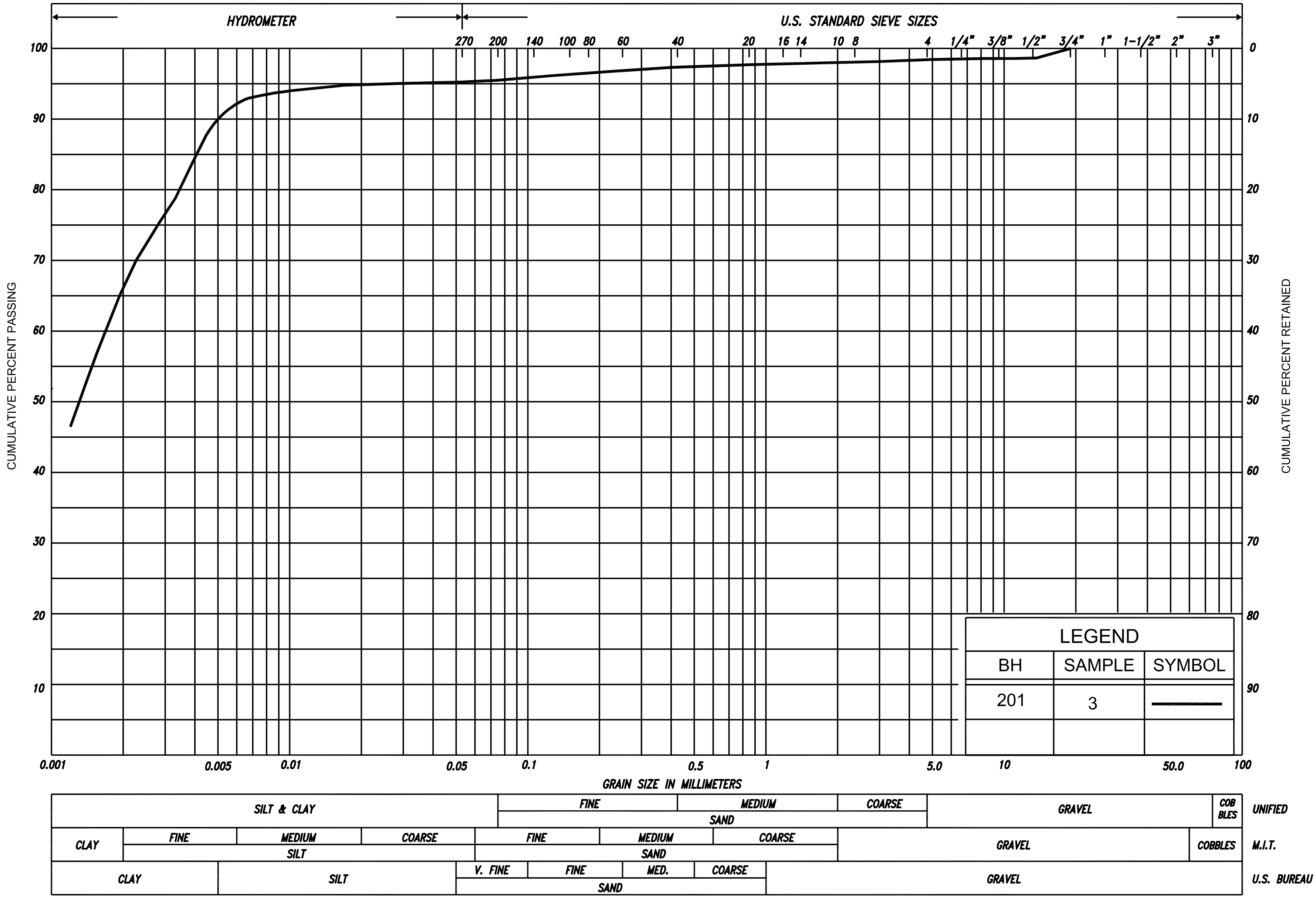


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand, trace gravel
(FILL)

FIG No.	GS-2
HWY:	11
G.W.P. No.	154-98-00



LEGEND

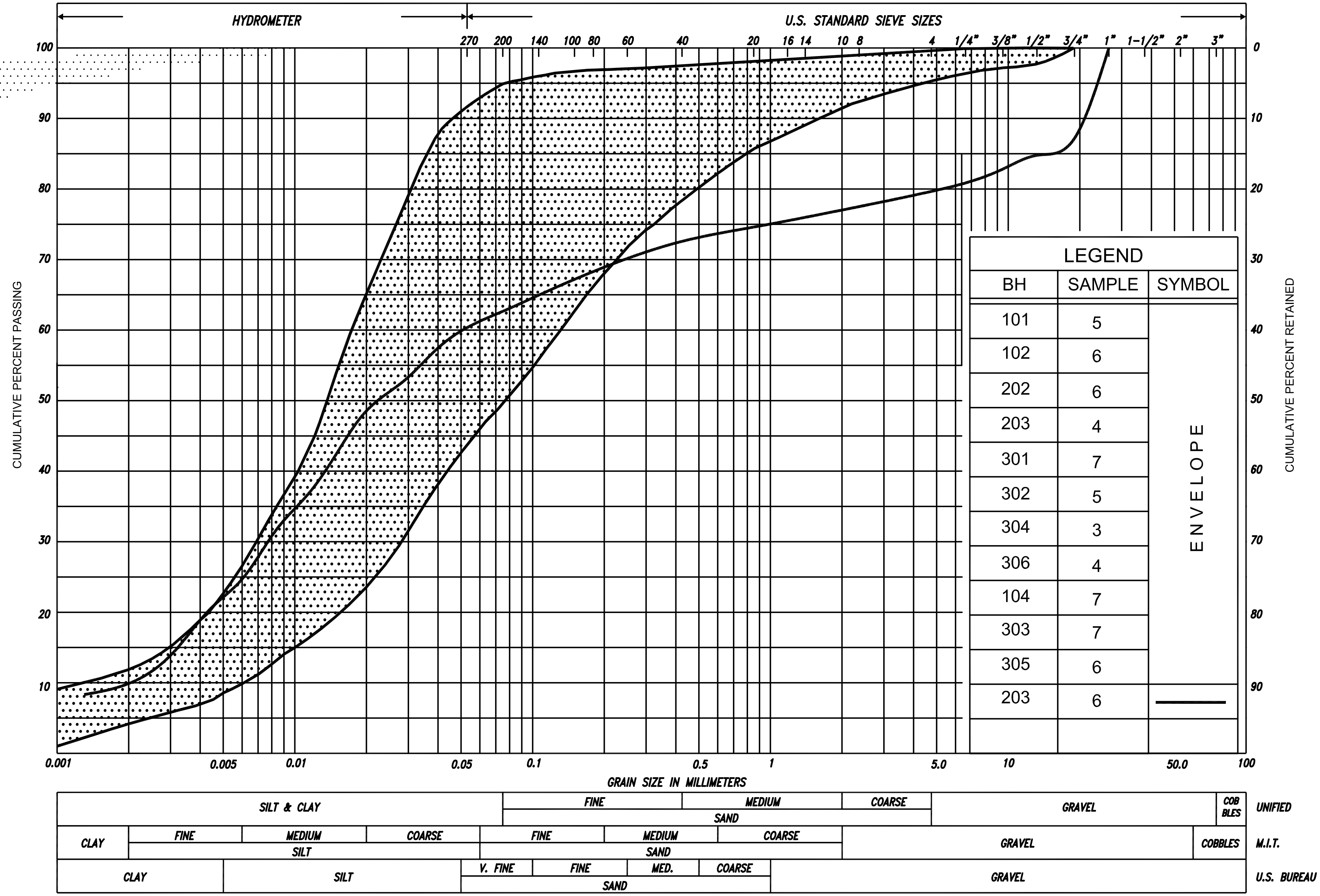
BH	SAMPLE	SYMBOL
201	3	———

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand, trace gravel

FIG No.	GS-3
HWY:	11
G.W.P. No.	154-98-00



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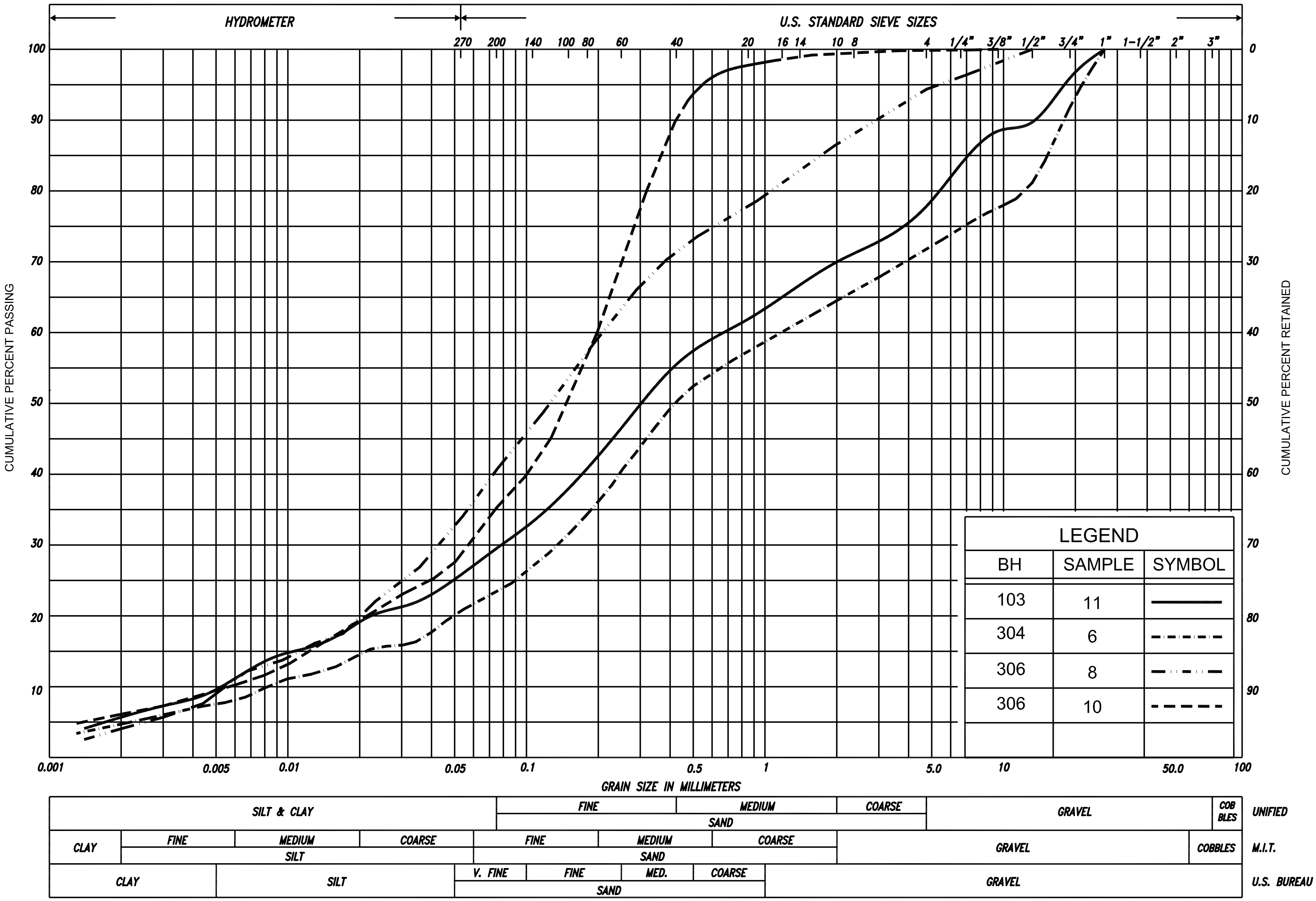
GRAIN SIZE DISTRIBUTION

SILT/SILT and SAND, some to trace clay, with to trace gravel
(TILL)

FIG No.	GS-4
HWY:	11
G.W.P. No.	154-98-00



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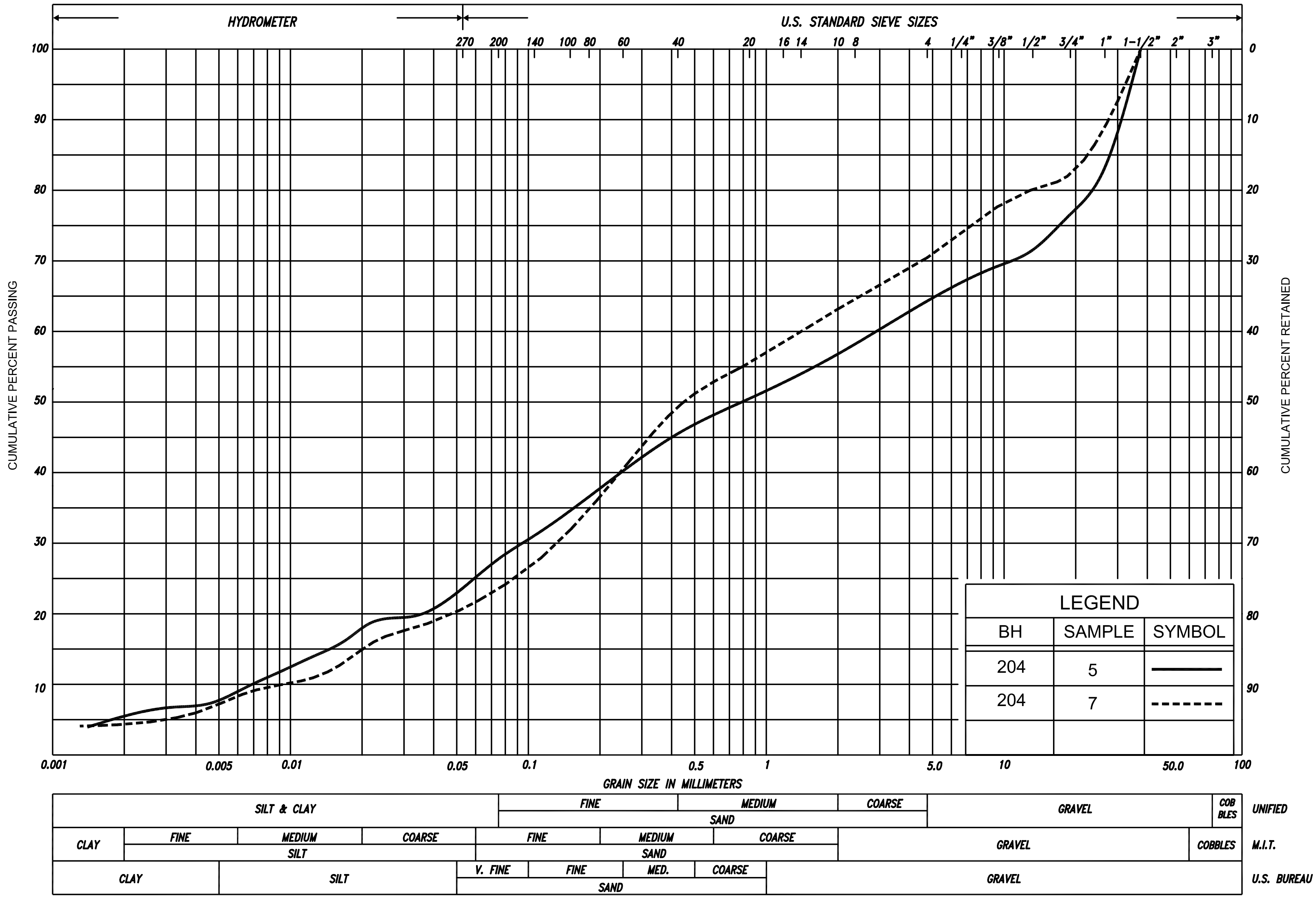
GRAIN SIZE DISTRIBUTION

SAND/SILTY SAND, trace to with gravel, trace clay
(TILL)

FIG No.	GS-5
HWY:	11
G.W.P. No.	154-98-00



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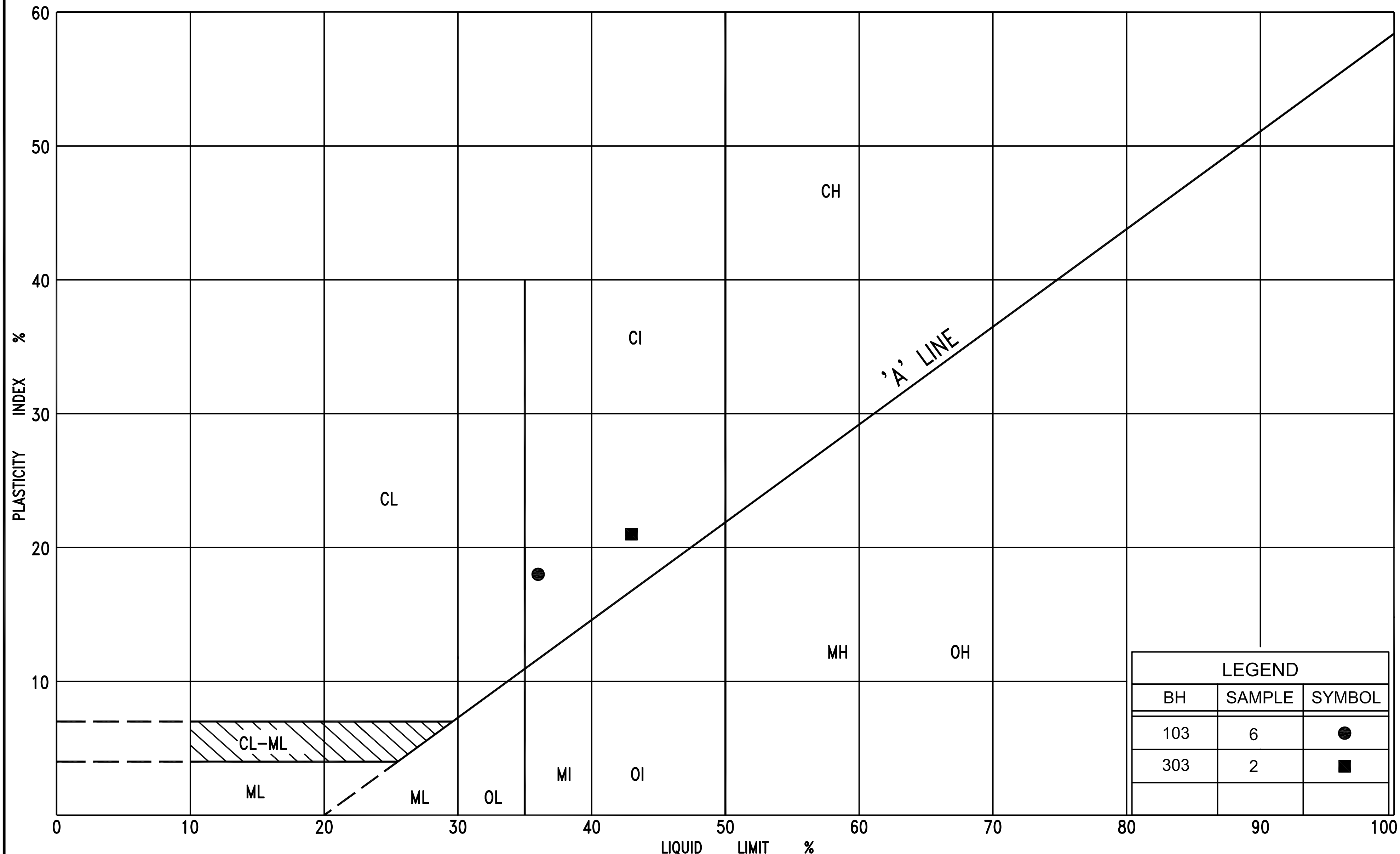
GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL, some to with silt, trace clay
(TILL)

FIG No.	GS-6
HWY:	11
G.W.P. No.	154-98-00



LEGEND

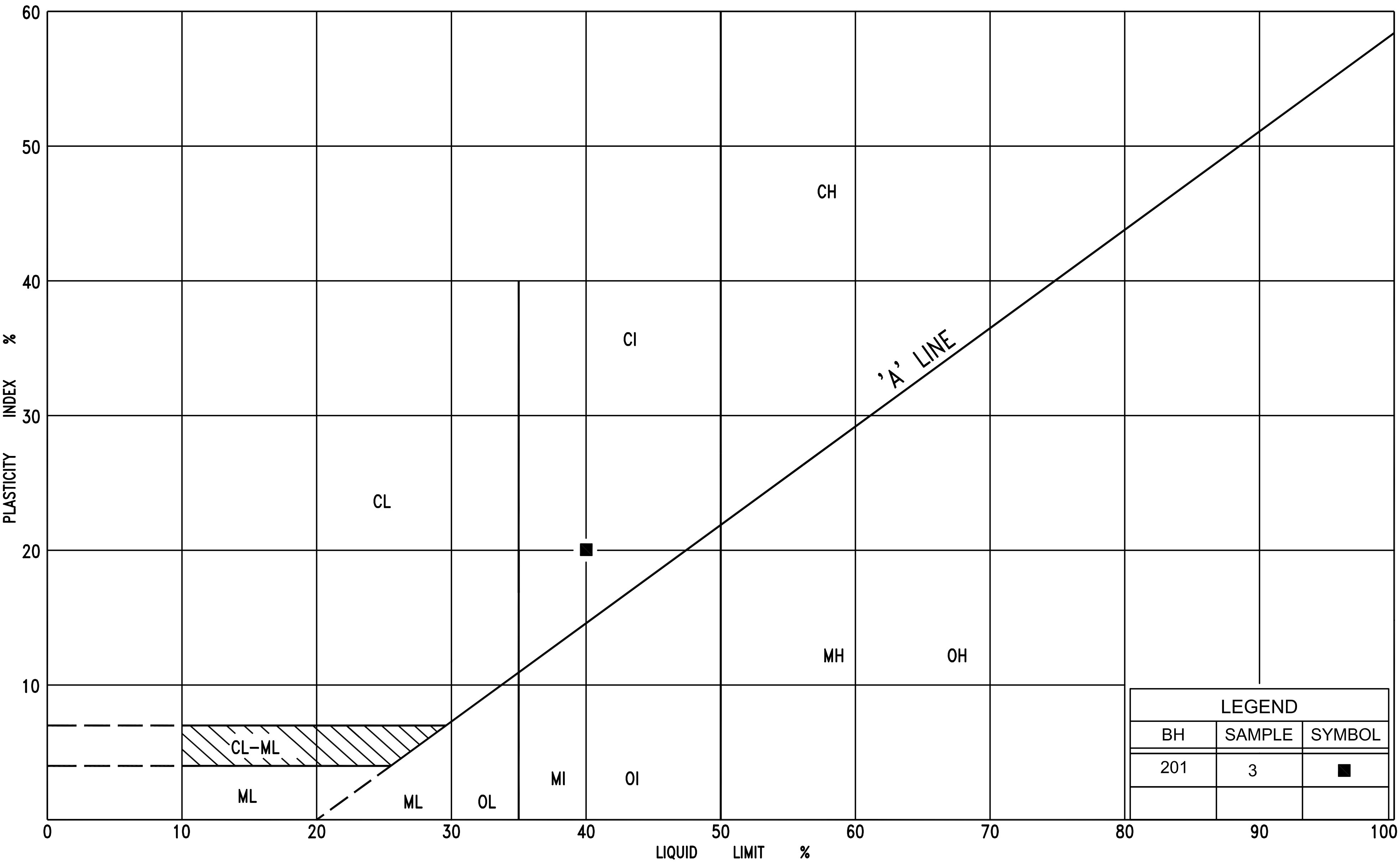
BH	SAMPLE	SYMBOL
103	6	●
303	2	■

Ministry of Transportation
Ontario

PLASTICITY CHART

SILTY CLAY, trace sand, trace gravel
(FILL)

FIG No.	PC-1
HWY:	11
G.W.P. No.	154-98-00



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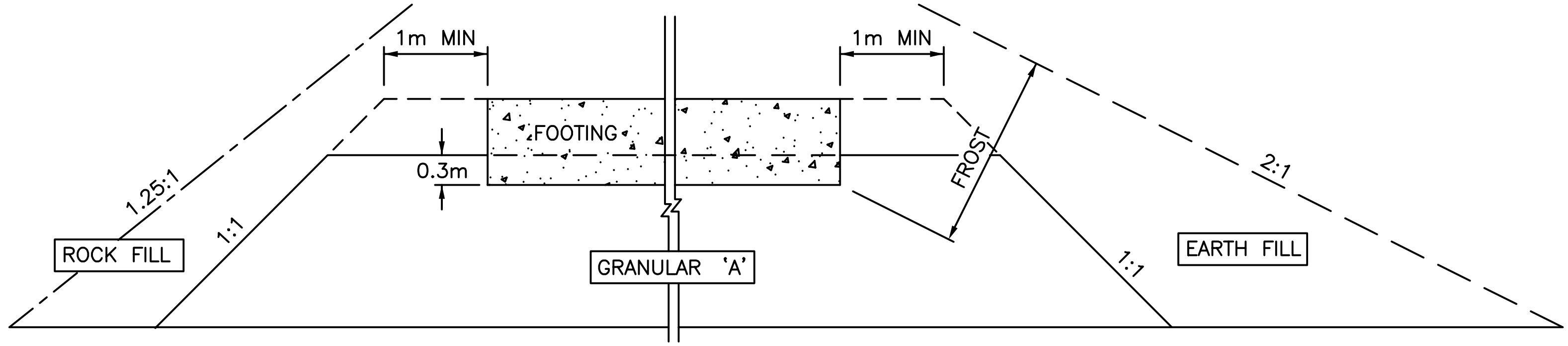
PLASTICITY CHART

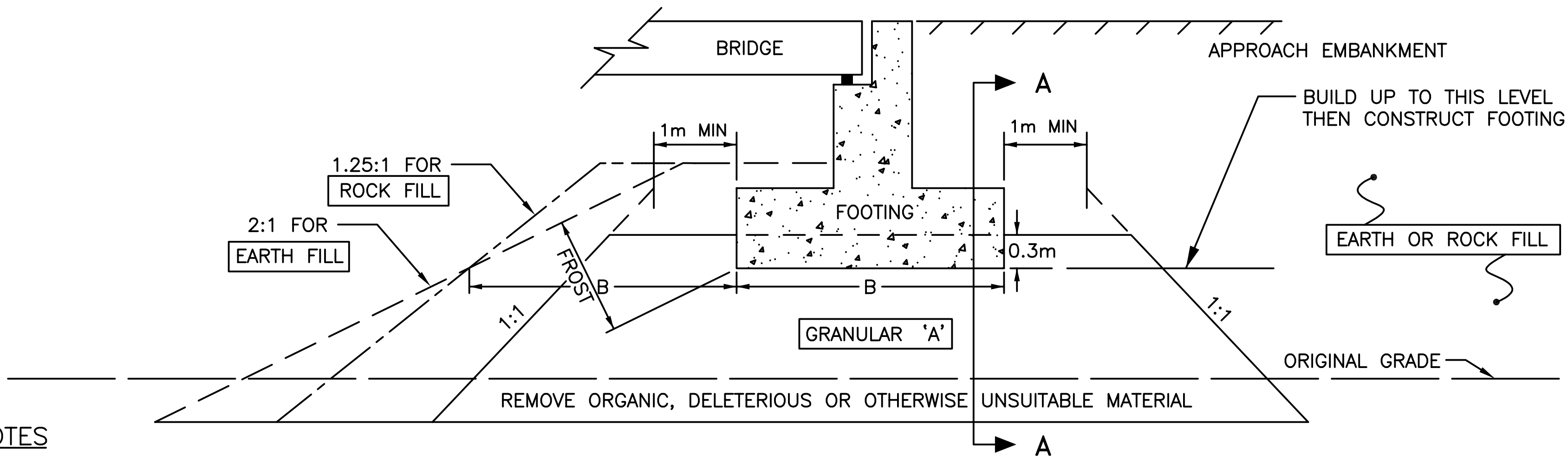
SILTY CLAY, trace sand, trace gravel

FIG No.	PC-2
HWY:	11
G.W.P. No.	154-98-00



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

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO

n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 101

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 350.6 N; 331 190.6 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 17, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS GR	SA	SI	CL
237.3	Ground Surface									
0.0	100mm asphaltic concrete over sand, some gravel, some silt (PAVEMENT FILL)	1	CS	-	237	o	10	77	(13)	
235.8 / 1.5	Silty clay, trace sand organic inclusions Stiff Moist (FILL)	2	SS	11	236					
		3	SS	14	235	o				
234.6 / 2.7	Silt, some sand trace clay, trace gravel Compact to very dense Brown Moist (TILL)	4	SS	16	234	o				
		5	SS	57		o	3	18	72	7
		6	SS	57	233	o				
		7	SS	46	232	o				
231.0 / 6.3	End of borehole	8	SS	50/8cm	231	o				

Sample 8: Sampler bouncing

* Borehole dry upon completion of drilling

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

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

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RECORD OF BOREHOLE No 102

 METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 345.2 N; 331 210.5 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 13, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (w)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.3	Ground Surface								
0.0	100mm asphaltic concrete over sand, some gravel, some silt (PAVEMENT FILL)	1	CS	-		237	o		20 70 (10)
235.8 / 1.5	Sand, trace gravel; Dense Brown Moist (FILL)	2	SS	35		236	o		
	Silt, trace clay, trace sand; Loose Grey Moist	3	SS	8		235, 234	o		
233.6 / 3.7	Silt, some clay, trace sand; Compact to dense Brown Moist (TILL)	4	SS	32	∇*	233	o		
		5	SS	21			o		
		6	SS	31		232	o		0 8 81 11
	trace gravel; Very dense Grey	7	SS	50/10cm		231	o		
		8	SS	50/8cm		230, 229			
228.1 / 9.2		9	SS	50/8cm					
	End of borehole								

Samples 7, 8, 9: Sampler bouncing

* 2008 11 13

∇ Water level observed during drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

RECORD OF BOREHOLE No 103

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 325.0 N; 331 307.7 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 12 & 13, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
237.3	Ground Surface				*							
0.0	100mm asphaltic concrete over sand, some gravel, some silt (PAVEMENT FILL)	1	CS	-		237						
235.8 / 1.5	Sand, trace gravel. Compact to loose, Brown, Moist (FILL)	2	SS	14		236	w ≈ 4		14	76	(10)	
	Silty clay, trace sand, trace gravel, organic inclusions. Stiff, Dark brown	3	SS	8		234						
		4	SS	7								
		5	SS	6								
		6	SS	7		232	wP–w–wL		1	9	43	47
	Organic silt	7	SS	12		231	86					
230.5 / 6.8	Sandy silt, trace gravel, cobbles. Very dense, Grey, Moist (TILL)	8	SS	50/5cm		230						
		9	SS	50/3cm		229						
		10	SS	50/8cm		228						
227.7 / 9.6	Sand with silt with gravel, trace clay. Very dense, Grey, Moist (TILL)	11	SS	95/25cm					22	48	24	6
227.1 / 10.2	End of borehole											

Samples 8, 9, 10, 11: Sampler bouncing

* Borehole dry

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
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RECORD OF BOREHOLE No 104

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 322.2 N; 331 321.1 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 12, 2008 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	GR	SA	SI	CL
237.2	Ground Surface									
0.0	100mm asphaltic concrete over sand, some silt, trace gravel cobbles (PAVEMENT FILL)	1	CS	-	237		6	84	(10)	
					236					
235.7 / 1.5	Silty clay, trace sand organic inclusions; Firm Brown Moist (FILL)	2	SS	8	235	~23				
	Clayey silt, trace sand; Grey	3	SS	5	234	~24				
		4	SS	8	233	~24				
	Organic silt; Dark brown	5	SS	7	232	~22				
		6	SS	5		~25				
231.1 / 6.1	Silt and sand trace clay, trace gravel; Compact Grey Moist (TILL)	7	SS	20	231	~17	5	42	44	9
					230					
	cobbles; Very dense	8	SS	75	229					
228.3 / 8.9		9	SS	30/1cm						
	End of borehole									

Sample 9: Sampler bouncing

* Borehole dry upon completion of drilling

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

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

RECORD OF BOREHOLE No 201

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 366.6 N; 331 097.4 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 16, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS GR SA SI CL
237.5 / 0.0	Ground Surface							
237.3 / 0.2	Topsoil							
236.9 / 0.6	Silty clay, organic inclusions	1	SS	3				
	Soft, Brown, Moist (FILL)							
	Silty clay, trace sand; Firm to stiff, Brown grey, Moist; trace gravel, layers of sandy silt	2	SS	7				
		3	SS	11		■		2 2 30 66
		4	SS	8	∇*			
234.5 / 3.0	Silt, trace sand, trace gravel, silty clay/ clayey silt layers; Compact to loose, Grey, Moist to wet (TILL)	5	SS	11				
		6	SS	9				
		7	SS	7				
		8	SS	12				
		9	SS	6		■		
	sandy				▼*			
		10	SS	26				
228.7 / 8.8	End of borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 237, 236, 235, 234, 233, 232, 231, 230, 229

* 2008 11 16

∇ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

+7, ×5: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 202

METRIC

G.W.P. 154-98-00 | LOCATION Coordinates: 5 505 341.6 N; 331 180.1 E | ORIGINATED BY F.P.

DIST 53 HWY 11 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY A.S.

DATUM Geodetic | DATE November 16, 2008 | CHECKED BY C.N.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.4	Ground Surface								
235.1 / 0.0	Topsoil								
234.8 / 0.3	Sandy silt, trace clay, trace gravel. Very loose, Moist (FILL)	1	SS	1		235			
0.6	Silt, trace sand, trace clay, trace gravel, silty clay layers. Compact to dense, Brown, Moist (TILL)	2	SS	10			~20		
		3	SS	11		234	~20		
		4	SS	37	∇*	233	~19		
	Grey, Moist to wet	5	SS	19		232	~20		
		6	SS	30			~18		1 4 87 8
	Very dense	7	SS	65		231	~17		
229.7 / 5.7		8	SS	81/25cm		230			

End of borehole

Sample 8: Sampler bouncing

* 2008 11 16

∇ Water level observed during drilling

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

RECORD OF BOREHOLE No 203

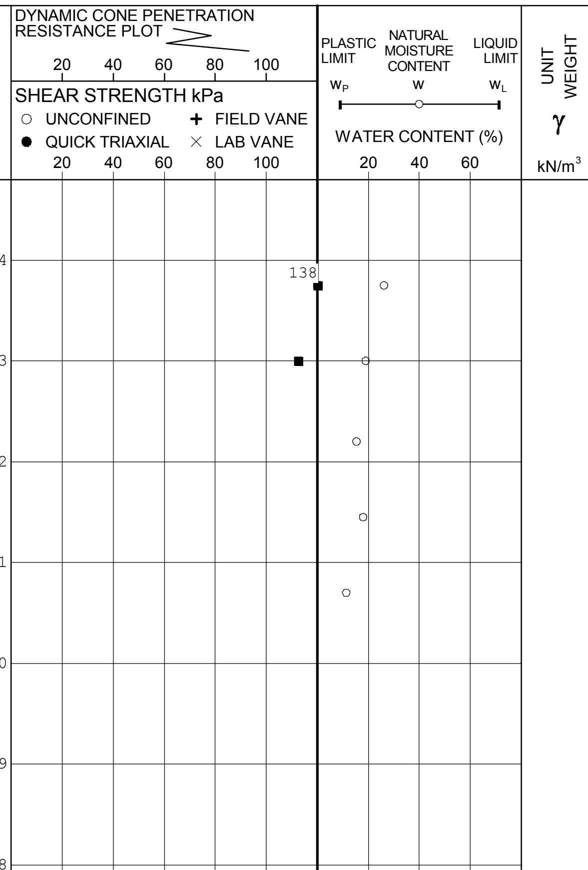
METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 335.0 N; 331 201.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE C.F.H.S.A. + Wash Bore COMPILED BY A.S.

DATUM Geodetic DATE November 16 & 17, 2008 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
234.8 0.0	Ground Surface				*		
234.6 0.2	Topsoil						
234.2 0.6	Silty clay organics inclusions Soft (FILL)	1	SS	3			
	Silty clay, trace sand Very stiff Brown Moist	2	SS	7		234	
		3	SS	5		233	
232.5 2.3	Silt, trace sand trace gravel, trace clay Compact Brown Moist (TILL)	4	SS	21		232	2 6 86 6
		5	SS	31			
	with gravel, some sand some clay Dense to very dense Grey Moist to wet	6	SS	109		231	20 17 53 10
						230	
	trace clay boulders	7	SS	50/13cm			
		8	CS	-		229	
		9	SS	50/13cm			
227.9 6.9	End of borehole	10	SS	30/3cm		228	



Samples 7, 9, 10: Sampler bouncing

* Borehole dry upon completion of drilling

■ Penetrometer test

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

+7, ×5 : Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No 204

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 308.0 N; 331 290.7 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 19, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.5 / 0.0	Ground Surface						
232.3 / 0.2	Topsoil						
	Silty sand. Loose to compact, Brown (FILL)	1	SS	1	∇*	232	
		2	SS	4			
		3	SS	13		231	
230.4 / 2.1	Sand and gravel, with silt trace clay, trace gravel cobbles and boulders. Dense to very dense, Grey, Moist (TILL)	4	SS	30/5cm		230	
		5	SS	41		229	36 36 22 6
		6	SS	50/10cm			
227.8 / 4.7		7	SS	30/10cm		228	30 46 20 4
	End of borehole						

Samples 4, 6, 7: Sampler bouncing

* 2008 11 19

∇ Water level observed during drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No 205

METRIC

G.W.P. 154-98-00	LOCATION Coordinates: 5 505 307.7 N; 331 299.9 E		ORIGINATED BY F.P.
DIST 53 HWY 11	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY A.S.
DATUM Geodetic	DATE November 19, 2008		CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.4 / 0.0	Ground Surface										
233.2 / 0.2	Topsoil										
232.8 / 0.6	Cobbles and boulders		1	SS	6		233				
	End of borehole Refusal on probable cobbles and boulders (possible rockfill) * Borehole dry										

+ 7 , × 5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 206

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 305.7 N; 331 348.3 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 17, 2008 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
237.2	Ground Surface				
0.0	100mm asphaltic concrete over sand, trace gravel (PAVEMENT FILL)	1	CS	-	~6
235.7 / 1.5	Silty clay, trace sand organic inclusions; Firm, Moist (FILL)	2	SS	8	
		3	SS	6	~21
		4	SS	6	
	trace gravel; Dark grey	5	SS	4	~21
231.9 / 5.3	Silt, trace sand, trace gravel, cobbles and boulders; Compact to Very dense, Grey, Moist (TILL)	6	SS	30/3cm	
		7	SS	23	~11
	sandy	8	SS	50/8cm	
228.7 / 8.5	End of borehole	9	SS	50/8cm	

Samples 6, 8, 9: Sampler bouncing

* Borehole dry upon completion of drilling

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (237 to 229); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No 301

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 350.6 N; 331 224.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 14, 2008 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
237.0	Ground Surface					
0.0	Sand and gravel, trace silt Loose Brown Moist (FILL)	1	SS	9		
	Sand, trace silt	2	SS	6		
235.5 1.5	Silty clay, trace sand Stiff to firm Brown Moist	3	SS	9		
		4	SS	8		
234.0 3.0	Silt, some sand trace gravel, trace clay sandy silt layers Compact Brown Wet (TILL)	5	SS	12	∇*	
		6	SS	36		
	Dense Grey Moist to wet	7	SS	36		3 13 76 8
		8	SS	39		
230.5	Very dense	9	SS	85/23cm		
6.5	End of borehole					

Sample 9: Sampler bouncing

* 2008 11 14

∇ Water level observed during drilling

■ Penetrometer test

ELEVATION SCALE: 236, 235, 234, 233, 232, 231

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

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

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RECORD OF BOREHOLE No 302

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 373.3 N; 331 227.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 14, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.7	Ground Surface							
0.0	Silty clay, trace sand organics inclusions	1	SS	7			~20	
235.1 / 0.6	Firm Brown Moist (FILL)					235		
	Silt trace clay, trace sand	2	SS	10			~17	
	Compact Brown Moist to wet (TILL)				∇*			
		3	SS	12		234	~21	
	Grey	4	SS	29		233	~16	
	sandy trace gravel	5	SS	30		232	~15	2 35 57 6
	Very dense	6	SS	50/13cm				
231.0 / 4.7		7	SS	50/5cm		231		
	End of borehole							

Samples 6, 7: Sampler bouncing

* 2008 11 14

∇ Water level observed during drilling

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT.

+^{7}, ×^{5} : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 303

METRIC

G.W.P. 154-98-00	LOCATION Coordinates: 5 505 373.9 N; 331 254.3 E		ORIGINATED BY F.P.
DIST 53 HWY 11	BOREHOLE TYPE C.F.H.S.A. + NW Casing		COMPILED BY A.S.
DATUM Geodetic	DATE November 14, 2008		CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.3	Ground Surface					
0.0	Topsoil	1	SS	4		
234.7 / 0.6	Silty clay, trace sand organics inclusions Firm Moist (FILL)	2	SS	6		0 4 35 61
233.7 / 1.6	Silt trace clay, trace gravel sandy silt layers Loose to dense Brown Moist to wet sandy Compact to very dense (TILL)	3	SS	4	▽*	
		4	SS	35		
		5	SS	25		
		6	SS	52		
230.8 / 4.5	Silt and sand trace clay, trace gravel Very dense Grey Moist (TILL)	7	SS	51		4 46 41 9
		8	SS	57		
		9	SS	50/13cm		
	Cobbles and boulders	10	RC NQ	**		**
		11	RC NQ	**		**
226.1 / 9.2	End of borehole					

Sample 9: Sampler bouncing

* 2008 11 14

▽ Water level observed during drilling

** Rock core sample not recovered

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); ELEVATION SCALE 235–227; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 304

METRIC

G.W.P. 154-98-00 | LOCATION Coordinates: 5 505 333.8 N; 331 337.0 E | ORIGINATED BY F.P.

DIST 53 HWY 11 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY A.S.

DATUM Geodetic | DATE November 18, 2008 | CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (w)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.7 / 0.0	Ground Surface							
232.5 / 0.2	Topsoil							
232.1 / 0.6	Silty clay, trace gravel organic inclusions Brown (FILL)	1	CS	-	∇*	232	~34	
	Silt, trace gravel cobbles. Compact Grey Moist to wet (TILL)	2	SS	14			~21	
		3	SS	11	▼*	231	~16	2 22 64 12
		4	SS	19		230	~16	
		5	SS	17		229		
228.9 / 3.8	Sand, with gravel some silt, trace clay cobbles and boulders. Very dense Grey Moist (TILL)	6	SS	50/10cm			~15	28 49 18 5
		7	SS	25/2cm		228		
		8	SS	50/8cm		227		
	cobbles and boulders	9	RC NQ	**		226		**
		10	RC NQ	**		225		**
224.8 / 7.9	End of borehole							

Samples 6, 7, 8: Sampler bouncing

* 2008 11 13

∇ Water level observed during drilling

▼ Water level measured after drilling

** Rock core sample not recovered

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 305

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 333.6 N; 331 360.0 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 18, 2008 CHECKED BY C.N.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
233.1	Ground Surface					233		
0.0	Topsoil	1	SS	5	∇*		~41	
232.5 / 0.6	Sand, trace silt, trace gravel; Loose, Brown/dark brown, Wet (FILL)	2	SS	8		232	~21	
231.6 / 1.5	Silt, trace sand, trace gravel; Compact, Grey, Moist; sandy (TILL)	3	SS	15		231	~16	
		4	SS	24		230		
		5	SS	15				
229.4 / 3.7	Silt and sand, trace clay, trace gravel, cobbles; Compact to very dense, Grey, Moist (TILL)	6	SS	24		229	~12	4 45 47 4
		7	SS	76			~8	
228.0 / 5.1	End of borehole					228		

* 2008 11 18

∇ Water level observed during drilling

+7, ×5: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 306

METRIC

G.W.P. 154-98-00 LOCATION Coordinates: 5 505 318.5 N; 331 386.9 E ORIGINATED BY F.P.

DIST 53 HWY 11 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.S.

DATUM Geodetic DATE November 11, 2008 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
234.1	Ground Surface					
0.0	Topsoil					
233.9 / 0.2	Sand and gravel silty clay lenses Loose to very loose Dark brown Wet (FILL) Sandy silt clayey silt layers	1	SS	7		
		2	SS	5		
		3	SS	3	▽*	
232.0 / 2.1	Silt, with sand trace clay, trace gravel Compact Brown Wet (TILL)	4	SS	17		3 20 69 8
		5	SS	20		
		6	SS	17		
229.6 / 4.5	Silty sand trace gravel, trace clay Compact Grey Moist (TILL)	7	SS	25		
		8	SS	27		6 53 37 4
228.1 / 6.0	Sand with silt, trace clay Very dense Grey Moist (TILL)	9	SS	50/8cm	▼*	
226.3		10	SS	50/5cm		0 65 29 6
7.8	End of borehole					

Natural moisture content (%), samples 1–8 (approx.): 1: ~17; 2: ~20; 3: ~20; 4: ~17; 5: ~18; 6: ~16; 7: ~18; 8: ~17

Elevation scale: 234, 233, 232, 231, 230, 229, 228, 227

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Sample 9, 10: Sampler bouncing

* 2008 11 11

▽ Water level observed during drilling

▼ Water level measured after drilling

+7, ×5 : Numbers refer to Sensitivity

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

METRIC

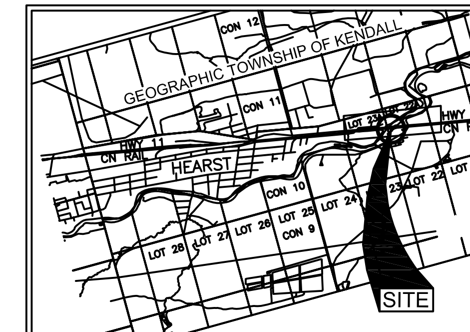
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
GWP No 154-98-00

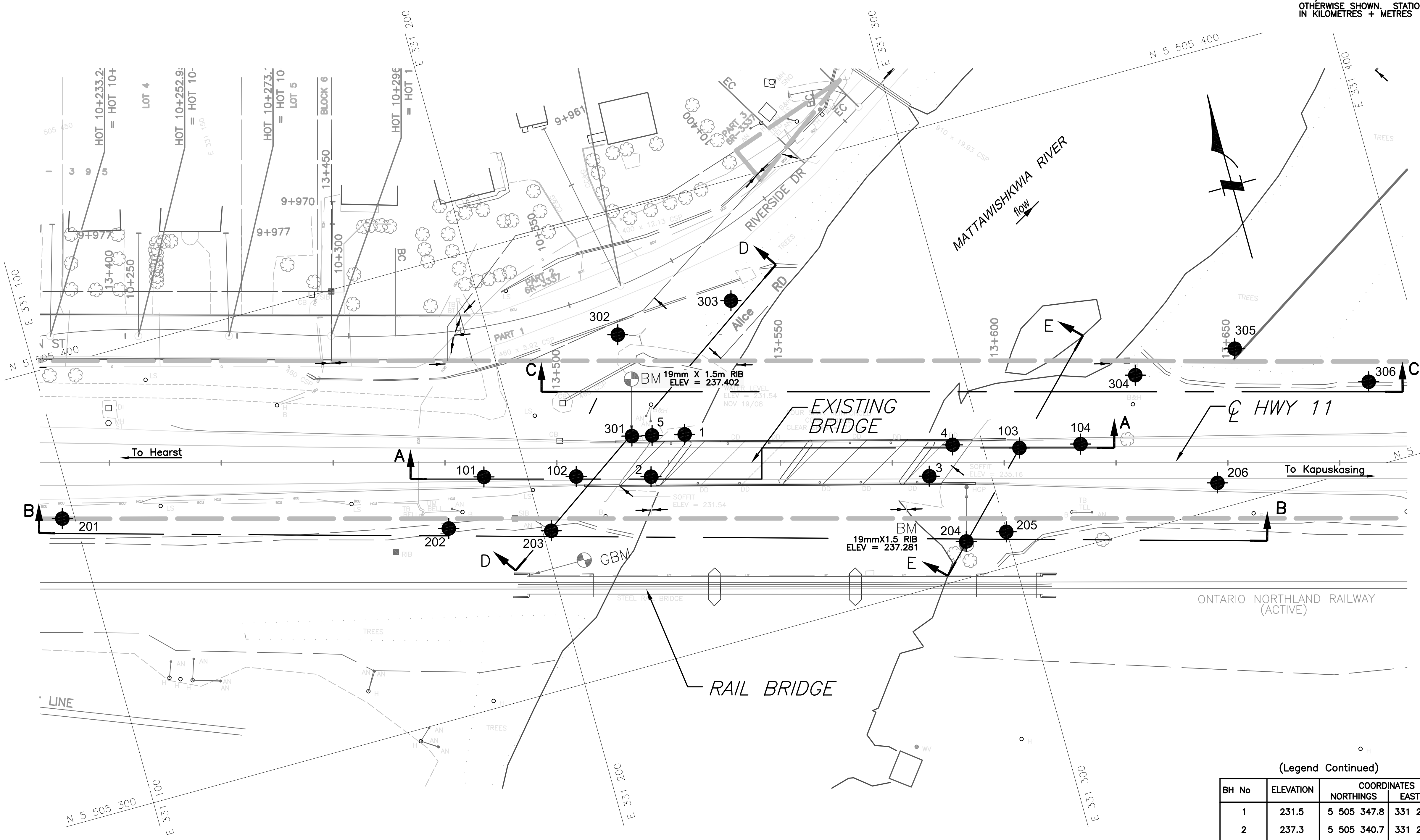
MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11
BOREHOLE LOCATIONS

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



KEY PLAN
SCALE 1:25,000



PLAN
SCALE
10 0 10 20m

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. 2008
- Head ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	COORDINATES NORTHINGS	EASTINGS
101	237.3	5 505 350.6	331 190.6
102	237.3	5 505 345.2	331 210.5
103	237.3	5 505 325.0	331 307.7
104	237.2	5 505 322.2	331 321.1
201	237.5	5 505 366.6	331 097.4
202	235.4	5 505 341.6	331 180.1
203	234.8	5 505 335.0	331 201.9
204	232.5	5 505 308.0	331 290.7
205	233.4	5 505 307.7	331 299.9
206	237.2	5 505 305.7	331 348.3
301	237.0	5 505 350.6	331 224.9
302	235.7	5 505 373.3	331 227.9
303	235.3	5 505 373.9	331 254.3
304	232.7	5 505 333.8	331 337.0
305	233.1	5 505 333.6	331 360.0
306	234.1	5 505 318.5	331 386.9

(Legend Continues)

(Legend Continued)

BH No	ELEVATION	COORDINATES NORTHINGS	EASTINGS
1	231.5	5 505 347.8	331 236.4
2	237.3	5 505 340.7	331 226.6
3	237.3	5 505 324.3	331 286.6
4	237.3	5 505 329.6	331 293.5
5	237.3	5 505 349.5	331 229.3

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

REGISTERED PROFESSIONAL ENGINEER C M P Nascimento June 18 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER B. R. GRAY June 18, 2009 PROVINCE OF ONTARIO

NOTES:
1. CO-ORDINATES OF PREVIOUS BOREHOLES (No. 1 TO 5), DRILLED IN 1977, ARE APPROXIMATE.
2. REFER TO DRAWINGS 2 AND 3 FOR SECTIONS A-A TO E-E.
3. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

REF No. Stantec drawing: 154-98-00-B_PLAN.dwg received on February 02, 2009

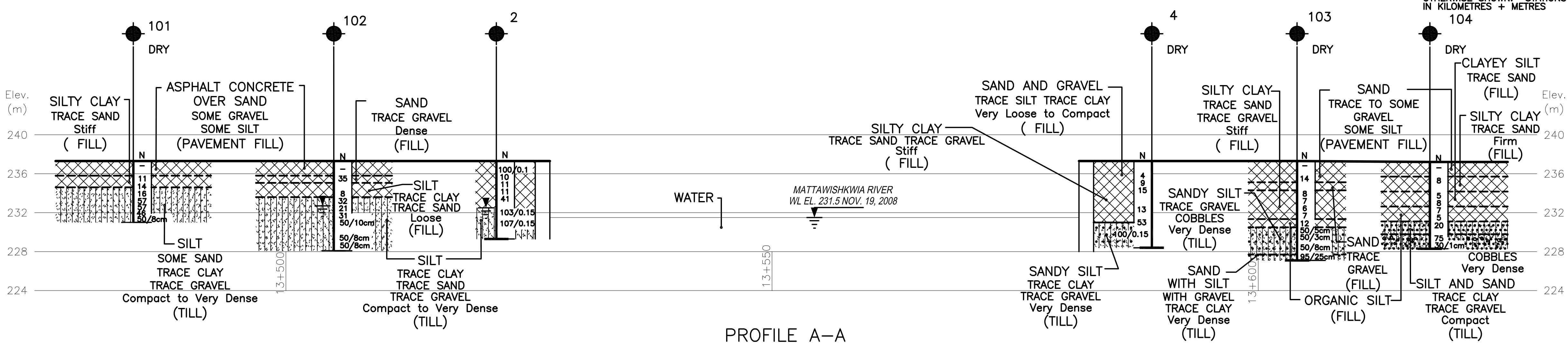
REVISIONS DATE	BY	DESCRIPTION

Geocres No. 42G-29

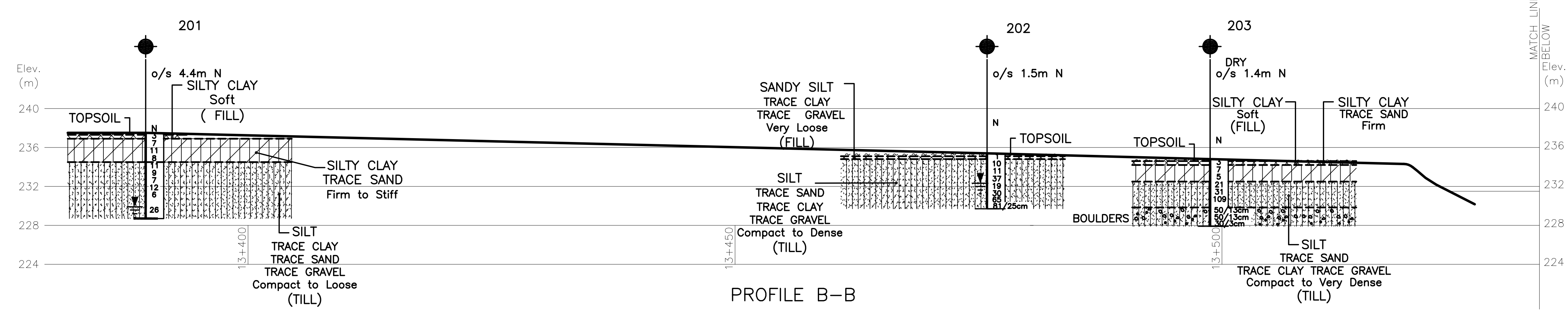
HWY No	11					DIST	53
SUBM'D	AS	CHECKED	CN	DATE	JUNE 18, 2009	SITE	39W-033
DRAWN	NA	CHECKED	CN	APPROVED	BRG	DWG	1

METRIC

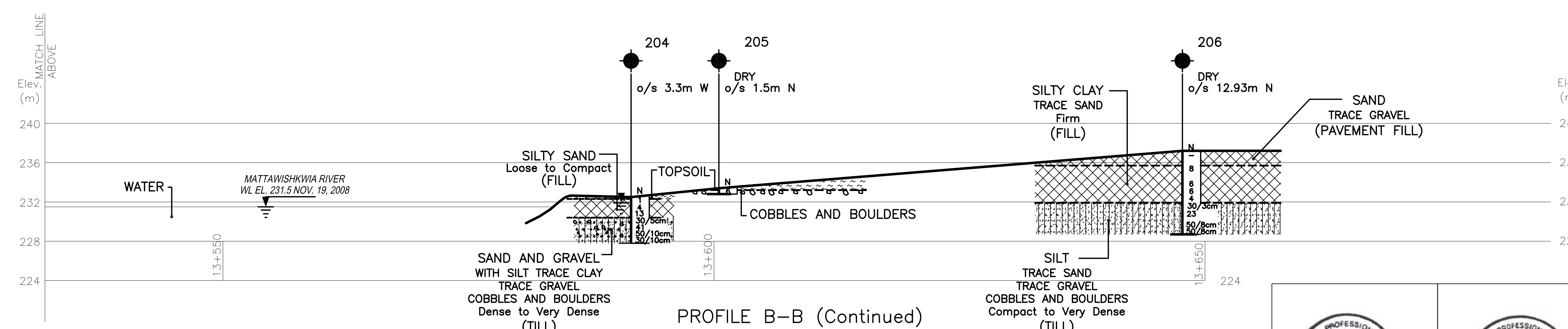
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES



PROFILE A–A



PROFILE B–B



PROFILE B–B (Continued)

NOTES:

1. REFER TO DRAWING 1 FOR BOREHOLE LOCATIONS PLAN AND DRAWING 3 FOR SECTIONS C–C, D–D AND E–E.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

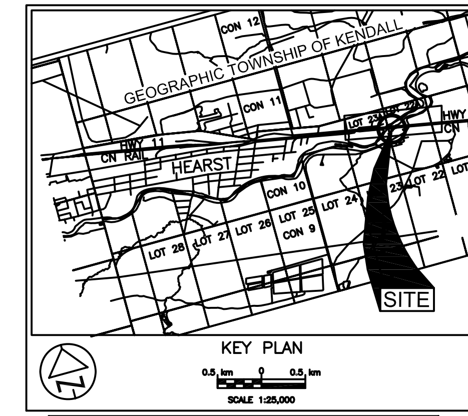
PROFILES
SCALE
5 0 5 10m

REF No. Stantec drawing: 154–98–00–B_PLAN.dwg received on February 02, 2009

CONT No
GWP No 154–98–00
SHEET

MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11
SOIL STRATA I

Peto MacCallum Ltd.
CONSULTING ENGINEERS



KEY PLAN
0.5 km 0 0.5 km
SCALE 1:25,000

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. 2008
- Head ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	COORDINATES NORTHINGS	EASTINGS
SEE DRAWING 1 FOR DETAILS			

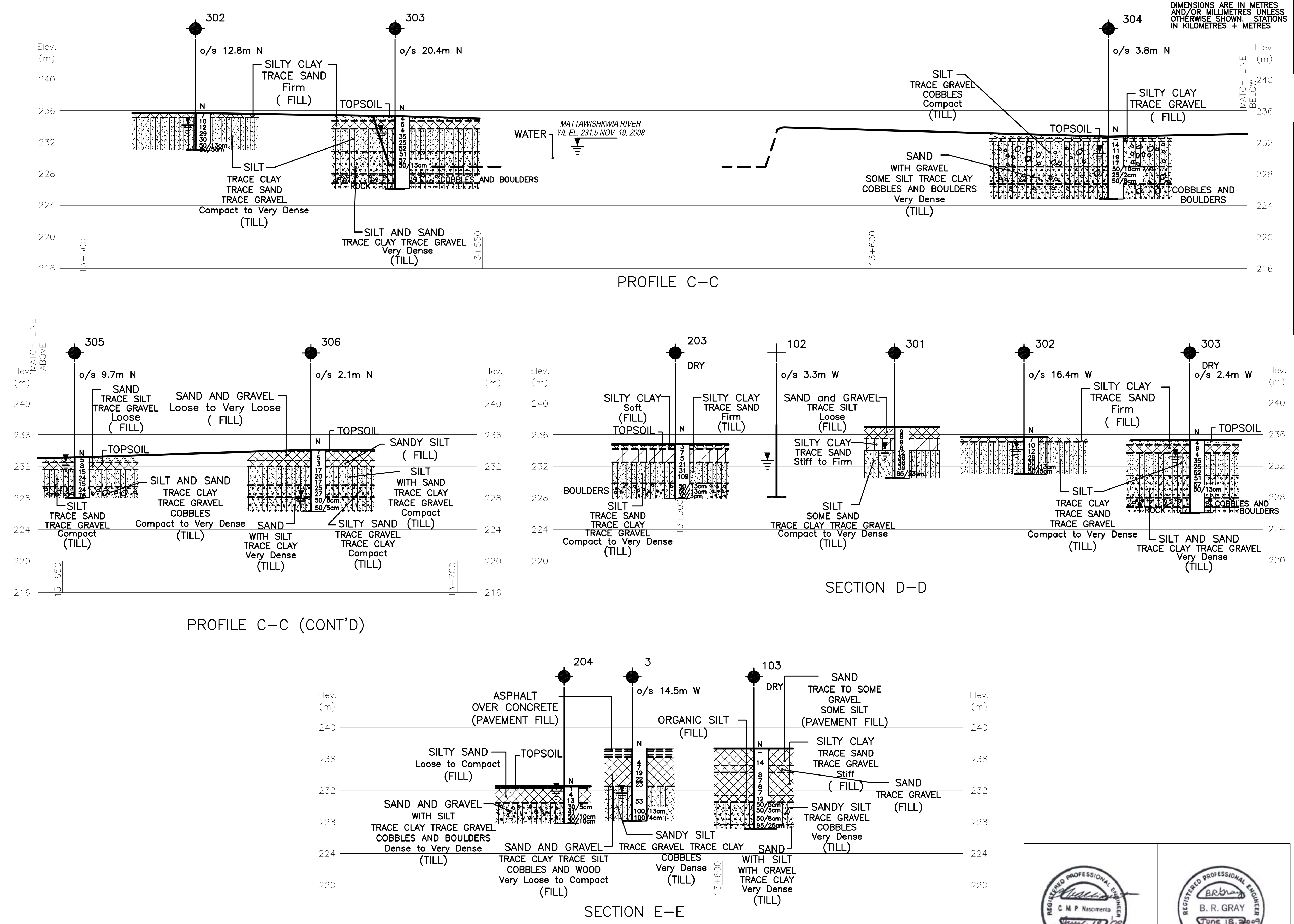
(Legend Continues)

– 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. 42G–29

HWY No	11			DIST	53
SUBM'D	AS	CHECKED	CN	DATE JUNE 18, 2009	SITE 39W–033
DRAWN	NA	CHECKED	CN	APPROVED BRG	DWG 2



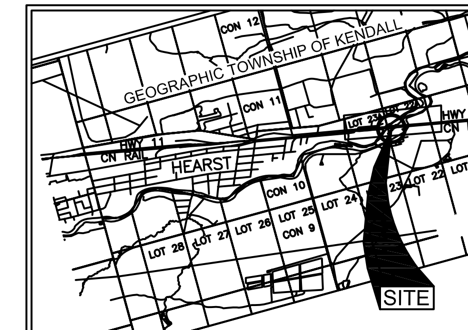
METRIC

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

CONT No
GWP No 154–98–00
SHEET

MATTAWISHKWIA RIVER BRIDGE
HIGHWAY 11
SOIL STRATA II

Peto MacCallum Ltd. CONSULTING ENGINEERS



KEY PLAN

SCALE 1:25,000

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. 2008
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	COORDINATES NORTHINGS	EASTINGS

SEE DRAWING 1 FOR DETAILS

(Legend Continues)

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

NOTES:

1. REFER TO DRAWING 1 FOR BOREHOLE LOCATIONS PLAN AND DRAWING 2 FOR SECTIONS A–A AND B–B.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

PROFILE AND SECTIONS

SCALE 5 0 5 10m

REF No. Stantec drawing: 154–98–00–B_PLAN.dwg received on February 02, 2009

REGISTERED PROFESSIONAL ENGINEER C M P Nascimento PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER B. R. GRAY June 18, 2009 PROVINCE OF ONTARIO

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No. 42G–29

HWY No 11			DIST 53
SUBM'D AS	CHECKED CN	DATE JUNE 18, 2009	SITE 39W–033
DRAWN NA	CHECKED CN	APPROVED BRG	DWG 3

APPENDIX A

Record of Borehole Sheets and Foundaiton Drawing

(Foundation Investigation Report Contract No. 82-213)

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

W P 236-77-00 LOCATION Sta. 380+90.0, Rt. 12.8' ORIGINATED BY C.T.J.

DIST 16 HWY 11 BOREHOLE TYPE Casing and Wash Boring COMPILED BY C.T.J.

DATUM Geodetic DATE July 30, 1978 CHECKED BY [illegible]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
759.3	Ground Level										
0.0	Boulders										
756.3	Clayey Silt, Some Sand, Firm		1	SS	5						
3.0	Sandy Silt, Trace of Gravel and Clay (Glacial Till)		2	SS	34						9 40 45 6
			3	SS	50						
750.8	Dense to Very Dense		4	SS	143						
8.5	End of Borehole						750				

$+^3$, $\times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2

W P 236-77-00 LOCATION Sta. 381+47.0, Lt. 10.5' ORIGINATED BY C.T.J.
DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.
DATUM Geodetic DATE August 1, 1978 CHECKED BY

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
778.2	Pavement Surface										
0.0	Asphalt, Reinforced Concrete										
1.5	Fill Material, Sand & Gravel, Traces of Silt & Clay, Occ. Boulders		1	SS	100/ 4" Bouncing						
			2	SS	10						
	Loose Silty Clay, Traces of Sand and Gravel, Occ. Layers of Organics up to ½" Stiff		3	SS	11		770				
			4	SS	11	▽					0 17 49 34
762.7			5	SS	41						0 6 91 3
15.5	Silt, Tr. of Sand & Clay						760	Cone Sta. 381+52, 24' Lt.			
	Sandy Silt, Trace of Gravel and Clay (Glacial Till) Dense to Very Dense		6	SS	103/ 6"						
752.2			7	SS	110/ 6"						2 34 56 8
26.0	End of Borehole										

+3, ×5 : Numbers refer to Sensitivity

20 15 ◇ 5 (%) STRAIN AT FAILURE 10

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION–ENGINEERING MATERIALS OFFICE–SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 3

W P 236-77-00 LOCATION Sta. 379+01.0, Lt. 9' ORIGINATED BY C.T.J.
DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.
DATUM Geodetic DATE August 1, 1978 CHECKED BY [illegible]

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
778.3	Pavement Surface									
.3	Asphalt & Reinforced Concrete									
1.1	Void									
2.6	Reinforced Concrete									
3.6	Fill Material, Sand & Gravel, Traces of Silt & Clay, Occasional Cobbles & Wood Inclusions Very Loose to Compact	1	SS	4						
		2	SS	7		770	Cone Sta. 379+01. 20' Lt.130/10"			41 52 6 1
		3	SS	19						
	Silty Clay, Trace of Sand & Gravel	4	SS	22						5 11 54 30
762.7	Very Stiff	5	SS	23						
15.6	Sandy Silt, Trace of Gravel and Clay, Occ. Cobbles, Very Dense Boulder at elev. 758 (Glacial Till)				▼	760				
		6	SS	53						
	Sandy Gravel	7	SS	100/ 5"						51 29 17 3
748.2		8	SS	100/ 1½"		750				5 49 42 4
30.1	End of Borehole									

$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⯁ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 4

W P 236-77-00 LOCATION Sta. 378+88.0, Rt. 10.5' ORIGINATED BY C.T.J.
DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.
DATUM Geodetic DATE August 1 & 2, 1978 CHECKED BY [illegible]

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
778.3	Pavement Surface										
.4	Asphalt Reinforced Concrete										
1.4	Void					*					
2.9	Fill Material, Sand & Gravel, Traces of Silt & Clay, Occ. Cobbles & Wood Inclusions, Very Loose to Compact		1	SS	4						
			2	SS	9		770				45 48 5 2
			3	SS	15						
	Silty Clay, Trace of Sand & Gravel, Occ. Layers of Organics Up to ½"		4	SS	13						
757.8	Stiff						760	120/10"			
20.5	Sandy Silt, Trace of Gravel and Clay Very Dense (Glacial Till)		5	SS	53			Cone Sta. 378+82, 22' Rt.			5 43 46 6
			6	SS	100/	6"					
749.3											
29.0	Refusal to Augering Probable Boulders										
	*Note: Water Level Not Established										

$+^3, \times^5$: Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 5

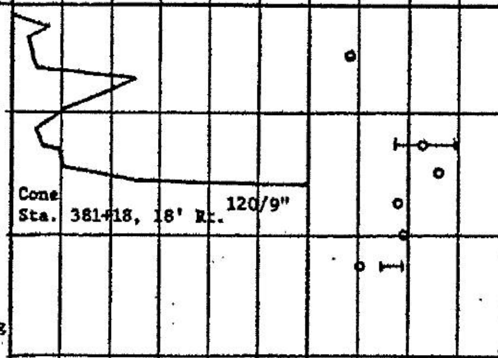
W P 236-77-00 LOCATION Sta. 381+22.5 Rt. 10.5' ORIGINATED BY C.T.J.

DIST 16 HWY 11 BOREHOLE TYPE Continuous Flight Auger COMPILED BY C.T.J.

DATUM Geodetic DATE August 1 & 2, 1978 CHECKED BY [signature]

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
778.3	Pavement Surface									
.3	Asphalt-Reinforced Concrete				*					
1.2	Fill Material Sand & Gravel Traces of Silt & Clay Occasional Cobbles Loose	1	SS	9						
						770				
	Clayey Silt	2	SS	22						
763.3	Silty Clay, Trace of Sand Occ. Layer of Organics Up to 1", Very Stiff	3	SS	24						0 6 56 38
15.0	Silt, Traces of Sand & Clay, Dense to Very Dense	4	SS	66			Cone Sta. 381+18, 18' Rt. 120/9"			0 6 91 3
		5	SS	38		760				
	Sandy Silt, Trace of Gravel and Clay Occasional Cobble Very Dense (Glacial Till)	6	SS	130						
		7	SS	100/5"	Bouncing					4 35 54 7
						750				
745.3 / 33.0	Refusal to Augering Probable Boulder									

*Note: Water Level Not Established



Dynamic cone penetration resistance plot scale: 20 40 60 80 100. Shear strength: o Unconfined, + Field Vane, ● Quick Triaxial, × Lab Vane. Plastic limit W_P, natural moisture content W, liquid limit W_L; water content (%) scale 10 20 30.

$+^3$, x^5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

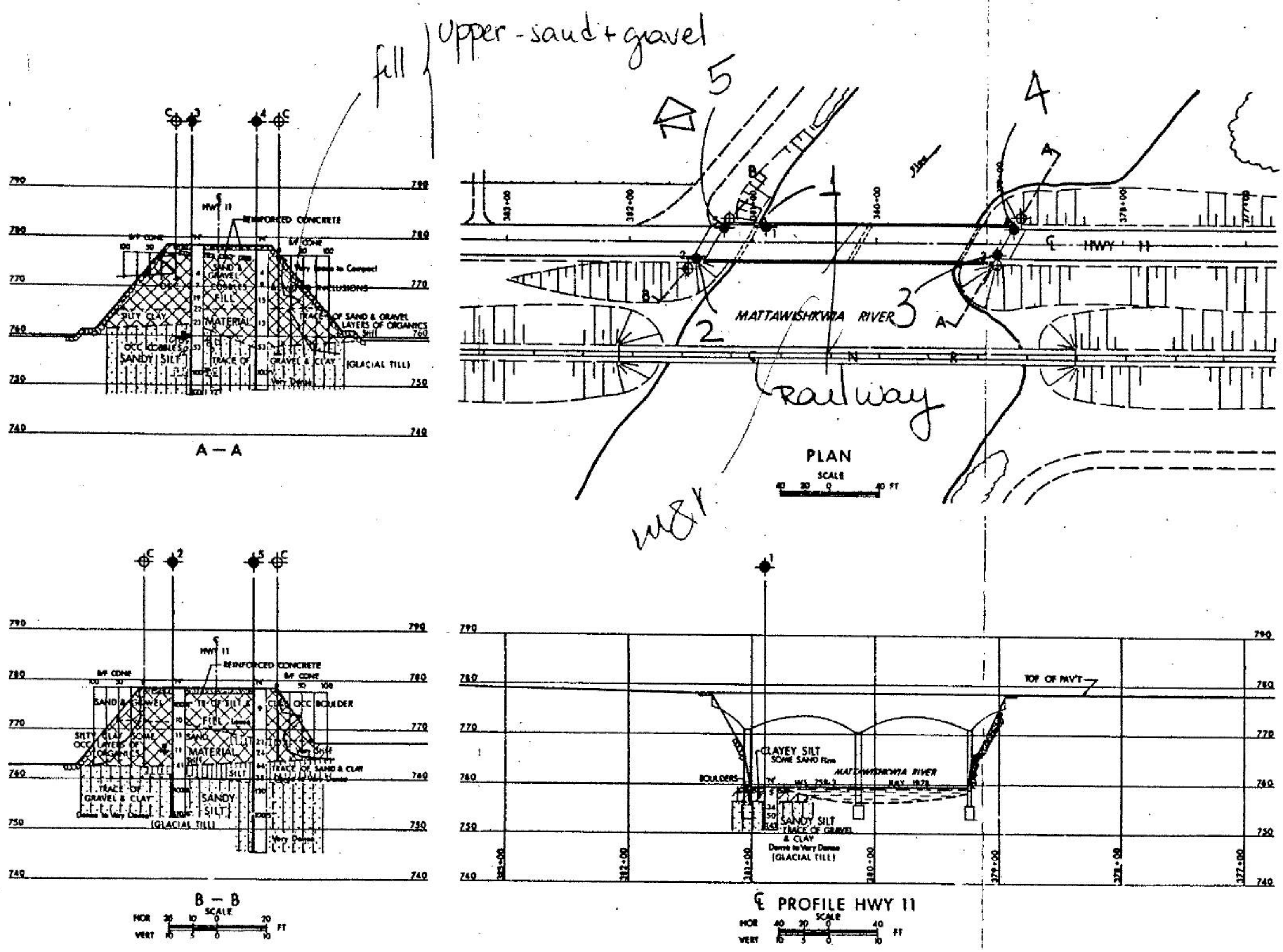
OFFICE REPORT ON SOIL EXPLORATION

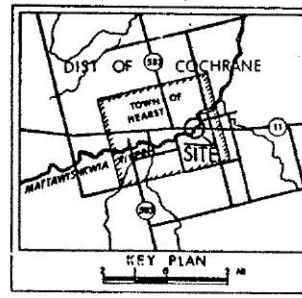
CONT No
WP No 236-77-00

MATTAWISHKWIA RIVER AT HEARST

BORE HOLE LOCATIONS & SOIL STRATA

SHEET





KEY PLAN

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- 'N' Blows/ft (Std Pen Test 350 ft lbs energy)
- CONE Blows/ft (60° Cone, 350 ft lbs energy)
- WL at time of investigation JULY 1978 NO WL observed BH No 4 & 5

No	ELEVATION	STATION	OFFSET
1	759·3	380+90	12·8'RT
2	778·2	381+47	10·5'LT
2C	778·2	381+52	20·0'LT
3	778·3	379+01	9·0'LT
3C	776·8	379+31	18·5'LT
4	778·3	378+88	10·5'RT
4C	776·8	378+82	20·0'RT
5	778·3	381+23	10·5'RT
5C	777·8	381+18	18·0'RT

-NOTE-
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS | DATE | BY | DESCRIPTION

HWY No 11 | DATE 78 10 16 | DIST 16 | SITE 39W-33 | DWG 2367700-A