



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**for**  
**TRILLIUM RAILWAY OVERHEADS**  
**HIGHWAY 406 FOUR-LANING**  
**GWP 280-99-00**  
**CITY OF WELLAND, ONTARIO**

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

**Distribution:**

- 3 cc: McCormick Rankin Corporation for distribution  
to MTO, Project Manager + one digital copy
- 1 cc: McCormick Rankin Corporation for distribution  
to MTO, Pavements and Foundations Section  
+ one digital copy
- 2 cc: McCormick Rankin Corporation  
+ one digital copy
- 1 cc: PML Kitchener
- 1 cc: PML Toronto

PML Ref.: 08TF005G  
Index No.: 127FIDR  
GEOCRES No.: 30M03-239  
March 3, 2009



## TABLE OF CONTENTS

### PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

1. INTRODUCTION .....	1
2. SITE DESCRIPTION AND GEOLOGY .....	2
3. INVESTIGATION PROCEDURES .....	2
3.1 Previous Investigation .....	2
3.2 Current (2001) Investigation .....	3
4. SUMMARIZED SUBSURFACE CONDITIONS .....	5
4.1 General .....	5
4.2 Topsoil .....	5
4.3 Clayey Silt .....	5
4.4 Clayey Silt Till .....	6
4.5 Sand and Silt Till .....	6
4.6 Gravelly Sand (Shale and Dolostone Fragments) .....	7
4.7 Bedrock .....	7
4.8 Groundwater .....	8
5. MISCELLANEOUS .....	8

### PART B – PRELIMINARY FOUNDATION DESIGN REPORT

6. ENGINEERING RECOMMENDATIONS .....	9
6.1 General .....	9
6.2 Foundations .....	11
6.2.1 General .....	11
6.2.2 Driven H-Pile Foundations .....	11
6.2.2.1 General .....	11
6.2.2.2 Conventional Abutment Considerations .....	12
6.2.2.3 Integral Abutment Considerations .....	13
6.2.2.4 Lateral Resistances .....	14



6.2.3	Drilled Caisson Foundations .....	15
6.2.4	Shallow Foundations .....	16
6.3	Lateral Earth Pressures.....	17
6.4	Approach Embankments .....	19
6.5	Construction Considerations .....	20
6.5.1	Excavation .....	20
6.5.2	Groundwater Control .....	21
7.	ADDITIONAL STUDIES .....	21
8.	CLOSURE.....	22

Table A – Rock Core Description

Table 1 – List of Standard Specifications Referenced in Report

Table 2 – Gradation Specification for Sand Fill in Pre-augered Holes at Integral Abutments

Figures GS-TR-1 to GS-TR-4 – Particle Size Distribution Charts

Figures PC-TR-1 to PC-TR-3 – Plasticity Charts

Figure 1 – Abutment on Compacted Fill Showing Granular ‘A’ Core

Explanation of Terms Used in Report

Record of Borehole Sheets

Drawing TR-1 – Borehole Locations

Appendix A – Previous Data from GEOCRE 30M3-181

**PART A**  
**PRELIMINARY FOUNDATION INVESTIGATION REPORT**  
for  
Trillium Railway Overheads  
Highway 406 Four-Laning  
GWP 280-99-00  
City of Welland, Ontario

---

**1. INTRODUCTION**

This report summarizes the results of the preliminary foundation investigation carried out for the proposed Trillium Railway Overhead structures at the Highway 406 in the City of Welland. Peto MacCallum Ltd. (PML) conducted the preliminary foundation investigation for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The Railway tracks are owned by Canadian National Railway (CNR) and are currently leased by Trillium Railway. The structures are part of the twinning of the Highway 406 section that extends from Port Robinson Road in the City of Thorold southerly 5.6 km to East Main Street in the City of Welland, Ontario.

The proposed four overheads will carry the new Highway 406 southbound and northbound lanes, S-E/W ramp and E/W-S ramp traffic over the existing railway tracks. The existing at-grade crossing will be elevated to carry the new Highway 406 northbound traffic at approximately the same location.

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





## **2. SITE DESCRIPTION AND GEOLOGY**

The contemplated structures are proposed about 250 m south of the existing Woodlawn Road and Highway 406 at-grade intersection. The site is about 2,000 m north of the existing East Main Street intersection at Highway 406.

Land use in the vicinity of the site comprises the transportation corridor of the existing Trillium Railway tracks, the existing Highway 406 and Woodlawn Road at-grade intersection about 250 m to the north. A golf course is present east of Highway 406. An approximately 47 m long concrete culvert was constructed about 60 m south of the existing Highway 406 railway at-grade crossing. The Trillium Railway tracks run roughly east to west on an approximately 35° skew to the existing roadway.

The local topography of the site is relatively flat with some low areas in the south side. The ground cover comprises grasses, bushes and stands of trees.

The site is located in the Haldimand Clay Plain physiographic region. The topography is gently flat to undulating. The soil cover in the region typically comprises lacustrine silts and clays. Dolostone bedrock of the Salina Formation is anticipated at an approximate depth of 30 m.

## **3. INVESTIGATION PROCEDURES**

### **3.1 Previous Investigation**

A foundation investigation was conducted by MTO about 100 m to the west of the current alignment in December 1985 (GEOCRETS No. 30M3-181, titled CNR Overhead Structure). This investigation involved three sampled boreholes to depths of 28.6 to 29.1 m, one sampled borehole to a depth of 17.2 m and a dynamic cone penetration test adjacent to each borehole (total four). Two boreholes were extended 2.9 and 3.0 m into bedrock by coring.



The subsurface information contained in the previous report was reviewed in the preparation of this report and found to be generally consistent with the current findings in 2001.

The stratigraphy revealed in the previous boreholes comprised a 14.6 to 21.3 m thick cohesive deposit of silty clay (to clayey silt as per current soil designation) overlying 7.3 to 14.5 m thick silty sand mantling shaley dolostone bedrock encountered at depths of 28.6 to 29.1 m (elevations 153.4 to 154.5). The soil cover had a typically stiff/hard consistency or compact/very dense relative density. The groundwater level recorded in three boreholes from the previous investigation drilled in November 1985 ranged from 1.3 to 2.0 m depth (elevations 181.1 to 181.2). Groundwater was not observed in borehole 4. Copies of the previous foundation investigation report including laboratory test results, the record of borehole sheets, stratigraphic profile and borehole location plan are provided in Appendix A.

### **3.2 Current (2001) Investigation**

The field work for the 2001 subsurface investigation was carried out on November 8 and 22, 2001. Two sampled boreholes were put down at the site. The boreholes were drilled to refusal at depths of 29.4 and 27.5 m at the locations shown on the attached Drawing TR-1. Borehole 1 was extended by coring 3.0 m into bedrock to a total depth of 32.4 m.

The locations of and ground surface elevations at the boreholes were established in the field by PML. The following benchmark was used for vertical reference:

<b>BENCHMARK</b>	<b>ELEVATION (*)</b>
Cut cross northwest corner of concrete culvert, west side of Highway 406, north of Trillium Railway tracks.	182.173

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

The boreholes were advanced using continuous flight solid stem augers and NW wash boring, powered by truck and track-mounted CME-75 and CME-55 drill rigs, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member off our engineering staff.



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

In borehole 1, casing was extended to the bedrock surface and an approximate 3.0 m length of rock core was recovered using NQ rock coring equipment. The PML senior geologist examined the recovered rock core samples. Detailed descriptions of the recovered rock core are provided in Table A.

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

Upon completion of augering, the boreholes were backfilled with auger cuttings to the ground surface.

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

- Natural moisture content determinations (26)
- Grain size analyses (6)
- Atterberg limits (6)

The results of the laboratory natural moisture content determinations, grain size analyses and Atterberg limits are shown on the Record of Borehole sheets. The grain size distribution charts are presented on Figures GS-TR-1 to GS-TR-4. The Atterberg limits results are presented on Figures PC-TR-1 to PC-TR-3.



#### **4. SUMMARIZED SUBSURFACE CONDITIONS**

##### **4.1 General**

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

The borehole locations and the preliminary layout of the Trillium Railway crossing structures are presented on the attached Foundation Drawing TR-1.

The subsurface stratigraphy revealed in the two boreholes generally consisted 4.1 and 2.8 m thick discontinuous deposits of clayey silt (overlain by 150 mm thick topsoil in borehole 2) overlying 25.3 and 23.1 m thick glacial deposits which included clayey silt till underlain by sand and silt till units. The soil cover had a typically stiff/very stiff consistency or dense/very dense relative density. A localized 1.6 m thick layer of gravelly sand including shale and dolostone fragments was penetrated in borehole 2 below the till between 25.9 and 27.5 m depths. Low to medium strength Dolostone bedrock was contacted below the soil cover at depths of 29.4 and 27.5 m (elevations 154.0 and 155.1). The strata encountered are summarized below.

##### **4.2 Topsoil**

A 150 mm thick topsoil layer was encountered in borehole 2 extending to an approximate elevation 182.4. The topsoil comprised silty clay and was visually judged to be low in organic content.

##### **4.3 Clayey Silt**

A cohesive deposit of stiff to very stiff/ hard clayey silt, with N values ranging from 13 to 30 was encountered surficially in borehole 1 and beneath the topsoil layer at an approximate depth of 0.2 m (elevation 182.4) in borehole 2. The stratum was 4.1 and 2.6 m thick extending to underlying cohesive clayey silt till at depths of 4.1 and 2.8 m (elevations 179.3 and 179.8) in boreholes 1 and 2, respectively.



The grain size distribution chart of a representative sample of the clayey silt is shown on Figure GS-TR-1 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-TR-1. The liquid limit of the clayey silt was 34 and the plastic limit 19, giving the plasticity index value of 15. The water content of the clayey silt varied from 18 to 23%.

#### **4.4 Clayey Silt Till**

Beneath the clayey silt deposit at depths of 4.1 and 2.8 m (elevations 179.3 and 179.8), a 15.7 and 17.0 m thick glacial till deposit of cohesive clayey silt till was present in boreholes 1 and 2, respectively. The unit extended to underlying sand and silt till at a uniform depth of 19.8 m (elevations 163.6 and 162.8).

The cohesive till deposit typically exhibited stiff to very stiff consistency with some local firm to hard layers. N values typically ranged from 7 to 20. N values of 32 and 38 were obtained below depths of 16.6 and 16.5 m (elevations 166.8 and 166.1) in boreholes 1 and 2, respectively. Penetrometer tests results on four samples ranged from 50 to 110 kPa.

The grain size distribution charts of representative samples of the clayey silt till are shown on Figure GS-TR-2 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-TR-2. The liquid limits of the clayey silt till were 25 and 30 and the plastic limits 18 and 19, giving the plasticity index values of 6 to 12. The water content of the deposit typically ranged from 18 to 21%, decreasing to between 9 and 13% below a depth of 15 m (elevations 168.4 and 167.6) in both boreholes. The foregoing properties indicated moist soils with relatively low compressibility characteristics.

#### **4.5 Sand and Silt Till**

The clayey silt till layer was underlain by a cohesionless dense to very dense sand and silt till unit, with N values of 30 to 66. The sand and silt till stratum was encountered at a uniform depth of 19.8 m (elevations 163.6 and 162.8) extending to depths of 29.4 and 25.9 m (elevations 154.0 and 156.7). The layer was 9.6 and 6.1 m thick in boreholes 1 and 2, respectively.



The grain size distribution charts of representative samples of the clayey silt are shown on Figure GS-TR-3 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-TR-3. The liquid limits of this till deposit were 15 and 17 and the plastic limits 13 and 16, giving very low plasticity index values of 1 and 2. The water content of the till was 7 and 8% indicating moist conditions.

#### **4.6 Gravelly Sand (Shale and Dolostone Fragments)**

A 1.6 m thick layer of very dense gravelly sand made mostly of broken-up shale and dolostone fragments was locally encountered beneath the sand and silt till deposit in borehole 2 at a depth of 25.9 m (elevation 156.7). This unit is likely a mixture of highly weathered rock and glacial till (shale-till complex) extending to the underlying probable bedrock at a depth of 27.5 m (elevation 155.1).

The grain size distribution chart of one sample of this soil is presented on Figure GS-TR-4. The material was wet based on tactile examination of the sample.

#### **4.7 Bedrock**

Dolostone bedrock of the Salina Formation was encountered/inferred in both boreholes below the native soils at the levels listed in the following table.

<b>BOREHOLE No.</b>	<b>DEPTH (m)</b>	<b>ELEVATION</b>	<b>ROCK CORE LENGTH (m) (*)</b>
1	29.4	154.0	3.0
2	27.5	155.1	-

(\*) NQ diamond rock cores obtained.



The core recovery was 93 and 96% for both core runs obtained in borehole 1. The rock exhibited a low to medium strength and was found to be unweathered. The rock typically is of very poor quality (RQD value was 0%). Loss of drill water circulation was experienced immediately after start of coring. Loss of drill water is attributed to discontinuities/fractures in the bedrock. The detailed rock core descriptions are provided on Table A.

#### **4.8 Groundwater**

Groundwater was encountered in borehole 2 during drilling at a depth of 27.5 m (elevation 155.1). Upon completion of drilling groundwater was measured in borehole 2 at a depth of 10.9 m (elevation 171.7). It is considered that the measured water level represents a local artesian condition in the layer of gravelly sand found immediately above the bedrock. It is estimated that the hydrostatic head is about 15 m in the deposit. The groundwater table was not determined upon completion of drilling in borehole 1 because the borehole was charged with water for rock coring purposes.

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

#### **5. MISCELLANEOUS**

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

**PART B**  
**PRELIMINARY FOUNDATION DESIGN REPORT**  
for  
Trillium Railway Overheads  
Highway 406 Four-Laning  
GWP 280-99-00  
City of Welland, Ontario

---

**6. ENGINEERING RECOMMENDATIONS**

**6.1 General**

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

Based on the preliminary drawing, the railway grade separation at Highway 406 includes four single span structures (SBL and NBL Highway 406 mainline and 2 structures for ramps). Approach embankments will be about 8 to 9 m high at the abutments. The following table summarizes the approximate location and span of the new structures.

LOCATION	STATION	BRIDGE LENGTH (m)
Highway 406 Southbound Overhead	STA. 13+430(*)	37
Highway 406 Northbound Overhead	STA. 13+460(**)	40
S-E/W Ramp Overhead	STA. 13+480(**)	43
E/W-S Ramp Overhead	STA. 13+420(*)	35

(\*) Refer to new Highway 406 SBL chainage.

(\*\*) Refer to new Highway 406 NBL chainage.





In summary, the subsurface stratigraphy revealed in the two boreholes drilled in 2001 at the site generally consisted 4.1 and 2.8 m thick discontinuous deposits of stiff to very stiff clayey silt (overlain by 150 mm thick topsoil in borehole 2) overlying 25.3 and 23.1 m thick glacial deposits which included typically stiff/very stiff clayey silt till underlain by dense to very dense sand and silt till units. The soil included a localized 1.6 m thick dense/very dense gravelly sand including fractured shale and dolomite (shale-till complex), immediately over the bedrock in borehole 2. Low to medium strength Dolostone bedrock was contacted below the soil cover at depths of 29.4 and 27.5 m (elevations 154.0 and 155.1). Artesian conditions with a piezometric head of 15 m to about elevation 171.7 were noted in borehole 2 within the localized gravelly sand layer (shale-till complex) found over the bedrock.

Use of conventional design and construction procedures for the overhead foundations is expected to be feasible using selected deep or shallow foundations.

It should be noted however, that very poor quality low to medium strength bedrock was identified in the boreholes that may necessitate the use of reduced ULS and SLS capacities for the piles. The foundation investigation conducted for design should call for a thorough assessment of the bedrock quality. The piles should be provided with driving shoes due to potentially heavy driving through the discontinuous gravelly sand till deposit which likely contains fractured limestone and for adequate seating on the bedrock.

In addition, the upper 16 m of the clayey soils that will support the approach embankments may experience consolidation settlement in the order of 80 mm due to the estimated 8 to 9 m high approach embankment fill loading. Further preliminary comments including preloading to minimize negative skin friction on pile foundations or reduce post-construction settlements of spread footings are included in this report.



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

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

## **6.2 Foundations**

### **6.2.1 General**

Based on the preliminary data, founding the proposed structure foundations on piles driven to refusal on the bedrock is considered feasible.

Drilled caissons bearing on the glacial till are also considered to be feasible to support the structure. Caissons extending to the bedrock are not considered to be practical due to the presence of fractured rock and sandy soils locally covering the bedrock surface.

Spread footings placed on the native soils or on engineered fill are not considered to be practical at this site since these foundations would require large footings and an extended period of time of several years to reduce the settlements of the subsoil due to the weight of the proposed approach embankments.

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

### **6.2.2 Driven H-Pile Foundations**

#### **6.2.2.1 General**

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



The preliminary pile foundation design recommendations for conventional and semi-integral abutments and for the pier(s), if utilized are provided on the following section. Additional recommendations for integral abutment foundations are provided in Section 6.2.2.3.

#### 6.2.2.2 Conventional Abutment Considerations

Piles for the proposed new structures should be driven to refusal on the bedrock underlying the site. Encountered bedrock levels are provided in the following table:

STRUCTURE	BOREHOLE No.	FOUNDING DEPTH (*) (m)	FOUNDING ELEVATION
E/W-S Ramp Overhead South Abutment	1	29.4 (**)	154.0 (**)
S-E/W Ramp Overhead North Abutment	2	27.5	155.1

Note: For preliminary design purposes, the founding levels for driven piles at the Highway 406 SBL overhead, north abutment of the E/W-S overhead and south abutment of the S-E/W overhead should be taken as those of the adjacent founding element since the bedrock surface and soil stratigraphy are relatively consistent in the boreholes.

(\*) Depth measured from the existing ground surface at the borehole locations.

(\*\*) Bedrock levels confirmed by rock cores.

Based on low to medium strength bedrock containing scattered voids anticipated at the structure locations, the preliminary factored axial resistance at ultimate limit states (ULS) for a steel HP 310x110 pile is 1,600 kN. The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be non-yielding and the pile length required, the design is not expected to be governed by settlement since the loading required to produce deformation of the pile will be much larger than the factored capacity at ULS at this site.

The piles will have to be driven through native soils containing compressible about 16 m thick upper layers of clayey soils at the structure locations. The existing grade at the structure locations will be raised about 8 to 9 m. Consequently, the development of negative skin friction on the piles due to consolidation of the clayey soils should be considered to affect the axial resistance of the piles at ultimate limit states (ULS).



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

The capacity of the HP 310x110 piles for the abutments should be reduced to allow for negative skin friction of 240 kN if the area is not preloaded and/or surcharged as recommended in Section 6.4 of this report.

The piles will be driven through approximately 11 m thick hard/very dense glacial till soils mantling the bedrock found under the abutments. Consequently, the piles should be equipped with steel H-pile driving shoes (OPSD-3000.100) or the Titus H Bearing Pile Point Standard Model (SP 903S01) to minimize the potential for damage when driving through the glacial till soils which may contain cobbles and boulders or through the gravelly sand (shale-till complex) layer containing fractured shale and limestone over the bedrock.

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

#### 6.2.2.3 Integral Abutment Considerations

For integral abutment design, the H-piles should be driven to very dense silt till or bedrock anticipated at the depths/elevations and axial resistance are indicated in the previous section. The minimum 5.0 m long free pile length required below the abutment stem will not be an issue at this site.

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



#### 6.2.2.4 Lateral Resistances

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

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

	STIFF/VERY STIFF CLAYEY SILT	STIFF/VERY STIFF CLAYEY SILT TILL	DENSE/VERY DENSE SAND AND SILT TILL	GRANULAR BACKFILL 'A' OR 'B' TYPE II
Factored Lateral Resistance at ULS, kN	160	160	120	120
Lateral Resistance at SLS, kN	65	65	50	50

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

To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction,  $k_s$  (MN/m<sup>3</sup>) 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<sup>3</sup> for granular backfill  
           = 1.3 MN/m<sup>3</sup> for sand and silt 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 clayey silt and clayey silt till soils should be taken as 28,000 kN/m<sup>3</sup> for preliminary purposes.

### 6.2.3 Drilled Caisson Foundations

The bedrock at the site is about 27 to 29 m deep and locally covered with a layer of gravely sand made up of fractured limestone and shale, consequently, caissons drilled to bedrock are not considered to be practical. It is recommended that caissons be founded at approximately the following reference levels:

Structure	Borehole No.	Founding Depth (*) (m)	Founding Elevation
E/W - S Ramp Overhead South Abutment	1	18.4	165
S-E/W Ramp Overhead North Abutment	2	18.6	164

Note: For preliminary design purposes, the founding levels for caissons at the Highway 406 SBL overhead, north abutment of the E/W-S overhead and south abutment of the S-E/W overhead should be taken as those of the adjacent founding element since the bedrock surface and soil stratigraphy are relatively consistent in the boreholes.

(\*) Depth measured from the existing ground surface at the borehole locations.

The recommended geotechnical resistance for the caissons are as follows:

Caisson Diameter (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Resistance at SLS (kN)
0.90	1750	1150
1.20	2600	1750



The foundation factor used for the ULS values was 0.5. The full load at geotechnical resistance at SLS would induce an estimated 25 mm of settlement on the caissons.

The caissons should not be founded deeper than about 21 to 22 m, elevation 161 to prevent caisson base disturbance due to the artesian groundwater found in borehole 2 at 25.9 m depth, elevation 156.7, within the shale-till complex.

The installation of the caissons should be monitored in accordance with SP 903S01. Downhole cleaning and inspection of the bed of the caisson should be carried out. All safety precautions related to confined space and fall arrest should be followed in accordance with the Occupational Health and Safety Act (OHSA).

#### 6.2.4 Shallow Foundations

As indicated in section 6.2.1 it is not considered to be practical to found the overhead foundations on spread footings because the upper native soils provide a relatively low geotechnical resistance and large footing widths would be required. These foundations are also subjected to the settlements of the native soils which are in the order of 80 mm and, consequently the construction of the foundations would have to proceed only after a pre-loading or surcharging period of at least 12 months had elapsed.



### 6.3 Lateral Earth Pressures

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

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

where  $K$  = coefficient of lateral earth pressure (dimensionless)  
 $\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$   
 $h$  = 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  $\phi$  = angle of internal friction of retained soil ( $35^\circ$  for Granular A or Granular B Type II or Type III)  
 $\delta$  = angle of friction between the soil and wall ( $23.5^\circ$  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, $\phi$ (degrees)	35
Unit weight, $\gamma$ ( $\text{kN/m}^3$ )	22.8
Coefficient of Active Earth Pressure, $K_a$	0.27
Coefficient of Earth Pressure At Rest, $K_o$	0.43
Coefficient of Passive Earth Pressure, $K_p$	3.69

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





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

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

Where required, a retained soil system (RSS) could also be employed at the abutments provided the estimated settlements noted in Section 6.4 Approach Embankments are accommodated. A high performance, high appearance rated RSS wall should be employed. The design, supply and construction of the RSS wall should conform to SP 599S22.

The bearing resistances and founding levels recommended previously for spread footings constructed on the structural fill should be employed for design of the RSS wall.

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



#### **6.4 Approach Embankments**

The following comments and recommendations apply to the embankment within 20 m of the abutments and are based on site conditions inferred from the two boreholes drilled at the site. These recommendations are considered to be adequate for preliminary design only. We refer to the separate preliminary pavement design report prepared by PML for conditions and recommendations further away from the abutments.

The approach embankment fill will be about 8 to 9 m high at the abutments for the proposed Trillium Railway structures.

The scope of work for this preliminary study did not require that boreholes be carried out for the approach embankments to the Trillium Railway crossing structure locations. Based on the stiff/very stiff to hard cohesive clayey soils underlain by cohesive glacial till followed by compact to very dense sand and silt till at the abutments, the approach embankments are likely to be founded on stiff to very stiff cohesive clayey soils.

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

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

Based on limited laboratory test data and including the plasticity characteristics of the native soils, it is estimated that some 80 mm of consolidation settlement of the clayey subgrade soils will occur at the embankments. This estimated settlement of cohesive soils is likely to take up to 12 months to occur to 80% completion.



For pile and caisson foundations, it is recommended that the approach embankment fill over the pile founding area should be placed 12 months before the pile driving or a 2 m high surcharge be applied for period of at least 9 months prior to driving the abutments piles. This partial pre-loading/ surcharging would eliminate or reduce the negative skin friction on the abutment piles or caisson foundations.

Further subsurface investigation and laboratory tests should be carried out during detail design for the purpose of designing an adequate pre-loading period for the foundations.

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

## **6.5 Construction Considerations**

### **6.5.1 Excavation**

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations. For this purpose, the upper cohesive stiff clayey silt encountered in borehole 2 is considered Type 3 soil according to OHSA (Ontario Regulation 213/91) criteria, while the very stiff clayey silt in borehole 1 is considered a Type 2 soil.



#### 6.5.2 Groundwater Control

Groundwater was observed during the course of the field work at a depth of 10.9 m (elevation 171.7) and is not expected to influence the installation of the pile or caisson cap foundations at the site. It is considered that seepage from pervious seams in the native soil and surface water run-off that enters the excavations should be readily handled by conventional sump pumping techniques.

Groundwater conditions should be further assessed during detail design when drilling the additional boreholes for the proposed structure and embankment foundations.

### 7. ADDITIONAL STUDIES

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

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

1. Carry out the complete scope of detailed field investigations at the structure sites. Incorporate the data from the previously drilled boreholes included in this report for the Detail Design.
2. Determine/evaluate the slope of the bedrock founding surface to evaluate the need for steel pile rock points and the reduction of axial bearing resistance related to the very poor bedrock and extent of the gravely sand (shale-till matrix) mantling the dolostone bedrock. Also determine/evaluate the condition of the artesian conditions encountered in this stratum.



3. Determine/evaluate the quality and strength of the bedrock underlying the site for the purpose of assigning the geotechnical resistances for driven pile foundations.

## 8. CLOSURE

This Preliminary Foundation Investigation and Design Report was prepared by Ms. N.S. Balakumaran, E.I.T. and Mr. C.M.P. Nascimento, P.Eng. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, carried out an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

**NOTE: Hard copies signed  
and stamped**

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

**NOTE: Hard copies signed  
and stamped**

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

CN/BRG:nb-mi



3. Determine/evaluate the quality and strength of the bedrock underlying the site for the purpose of assigning the geotechnical resistances for driven pile foundations.

#### 8. CLOSURE

This Preliminary Foundation Investigation and Design Report was prepared by Ms. N.S. Balakumaran, E.I.T. and Mr. C.M.P. Nascimento, P.Eng. 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.



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



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

CN/BRG:nb-mi



**TABLE A**  
**ROCK CORE DESCRIPTION**

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
1	15	29.4 – 30.8	96	0	29.4 – 32.4	DOLOSTONE: : Buff to grey, fine grained, low to medium strength; unweathered; with numerous black shale partings and occ. seams /partings of gypsum and calcite, occ. sphalerite inclusions; very close to close spaced flat bedding layers, smooth to rough planar, tight; very poor quality. (Salina Formation)
	16	30.8 – 32.4	93	0		

RQD: Rock Quality Designation

Originated: MR  
 Compiled: JFW  
 Checked: NB/CN



**TABLE 1**  
**LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT**

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

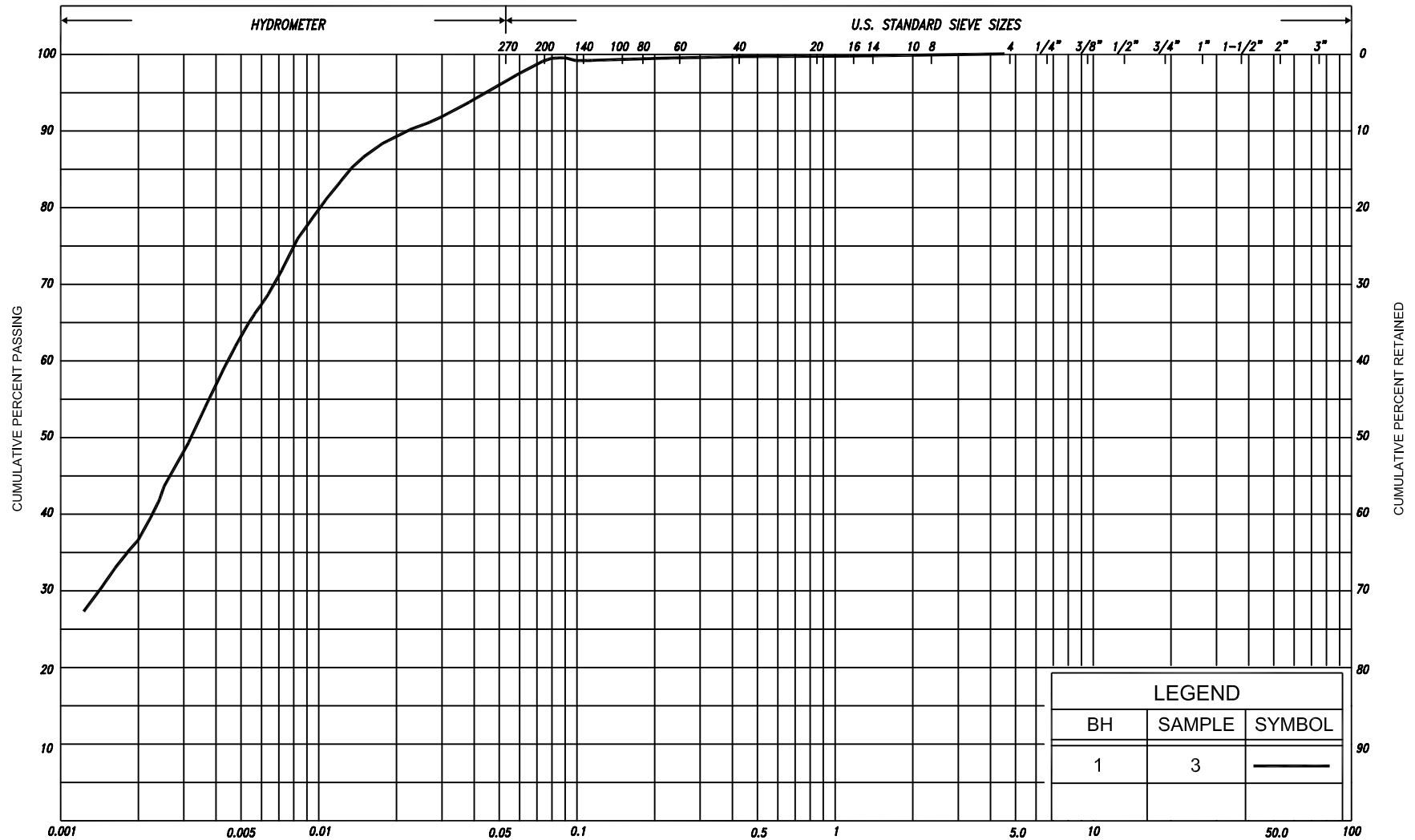




**TABLE 2**  
**GRADATION SPECIFICATION FOR SAND FILL IN**  
**PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS**

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

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



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

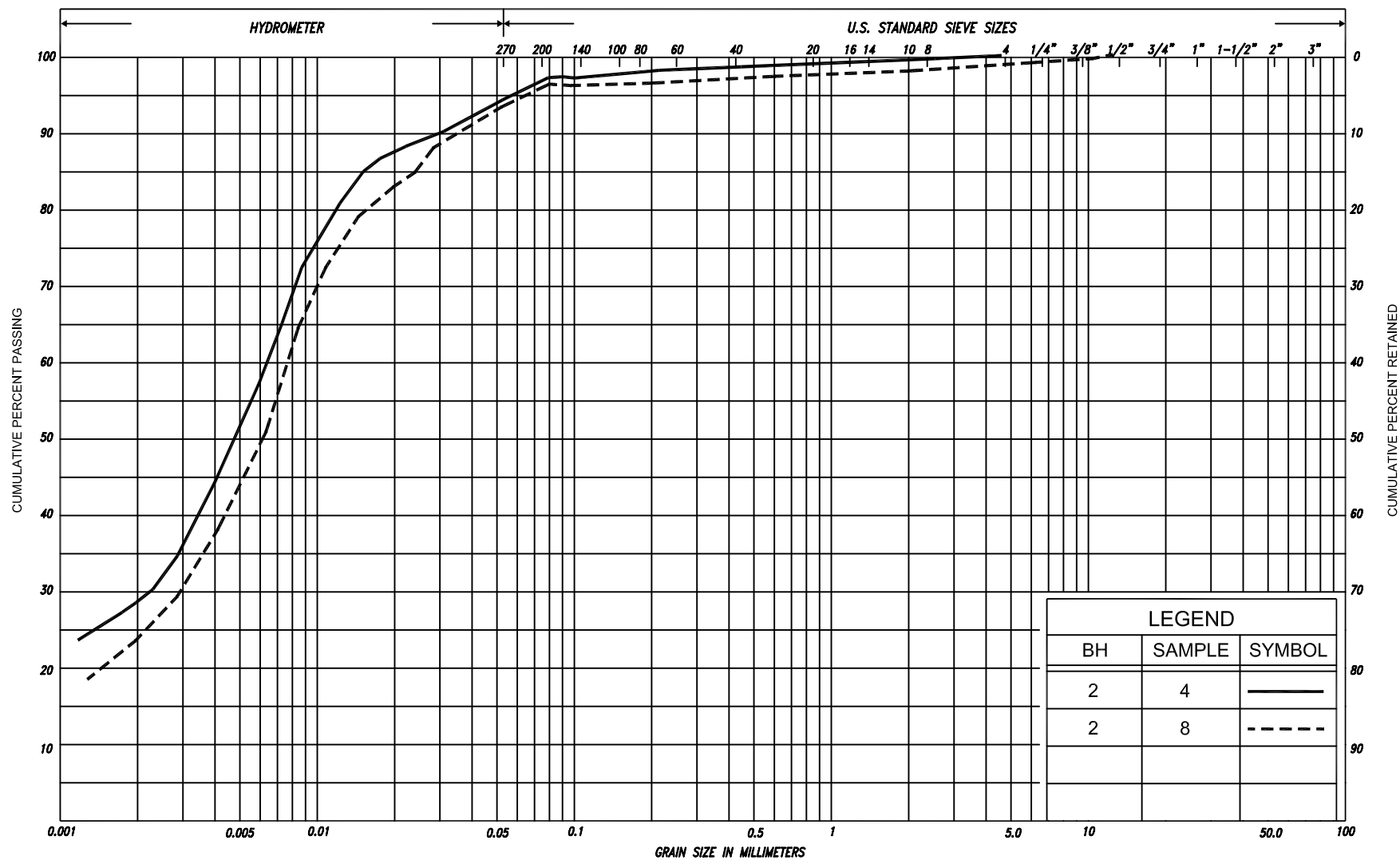
## GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

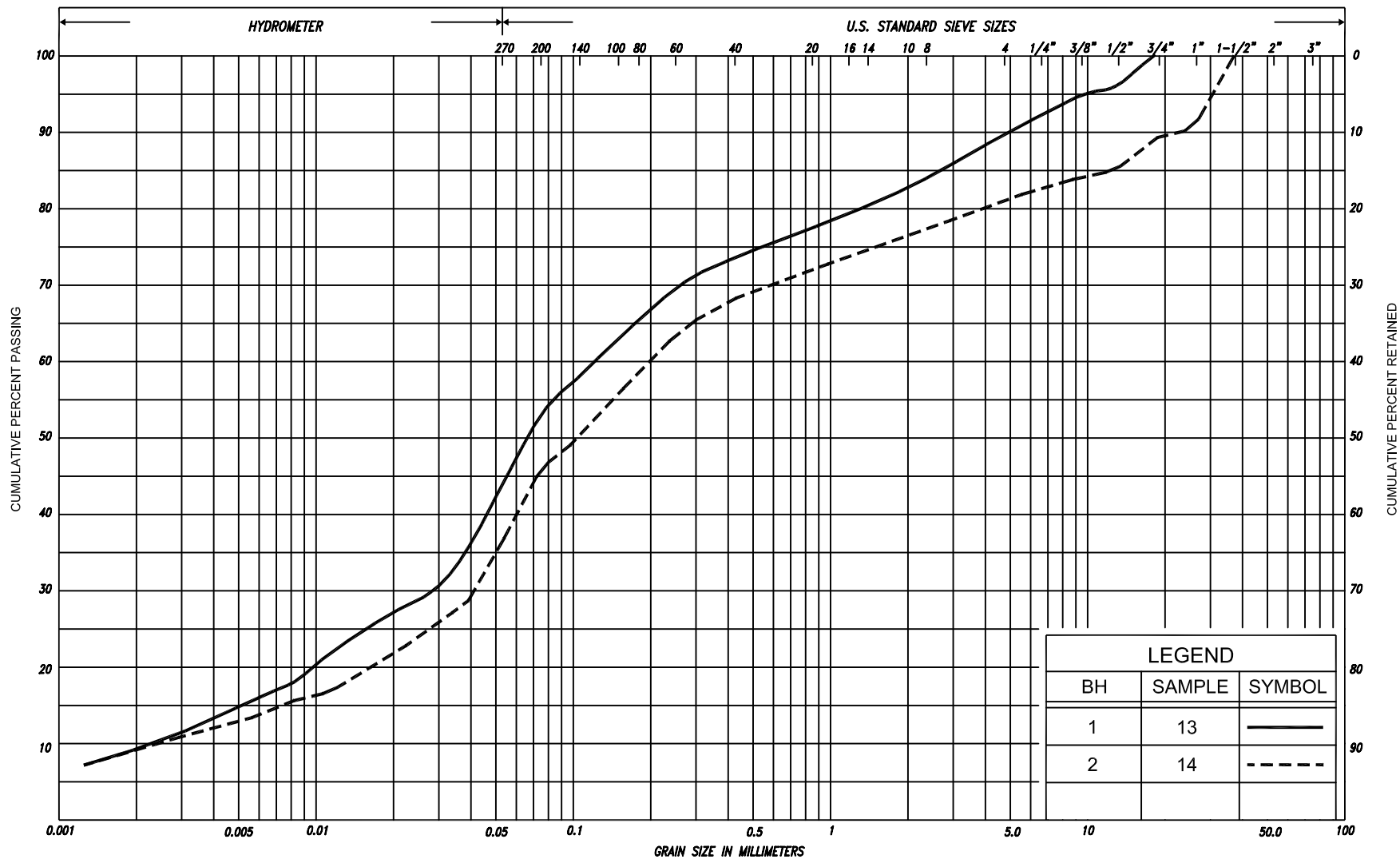
FIG No. GS-TR-1

HWY: 406

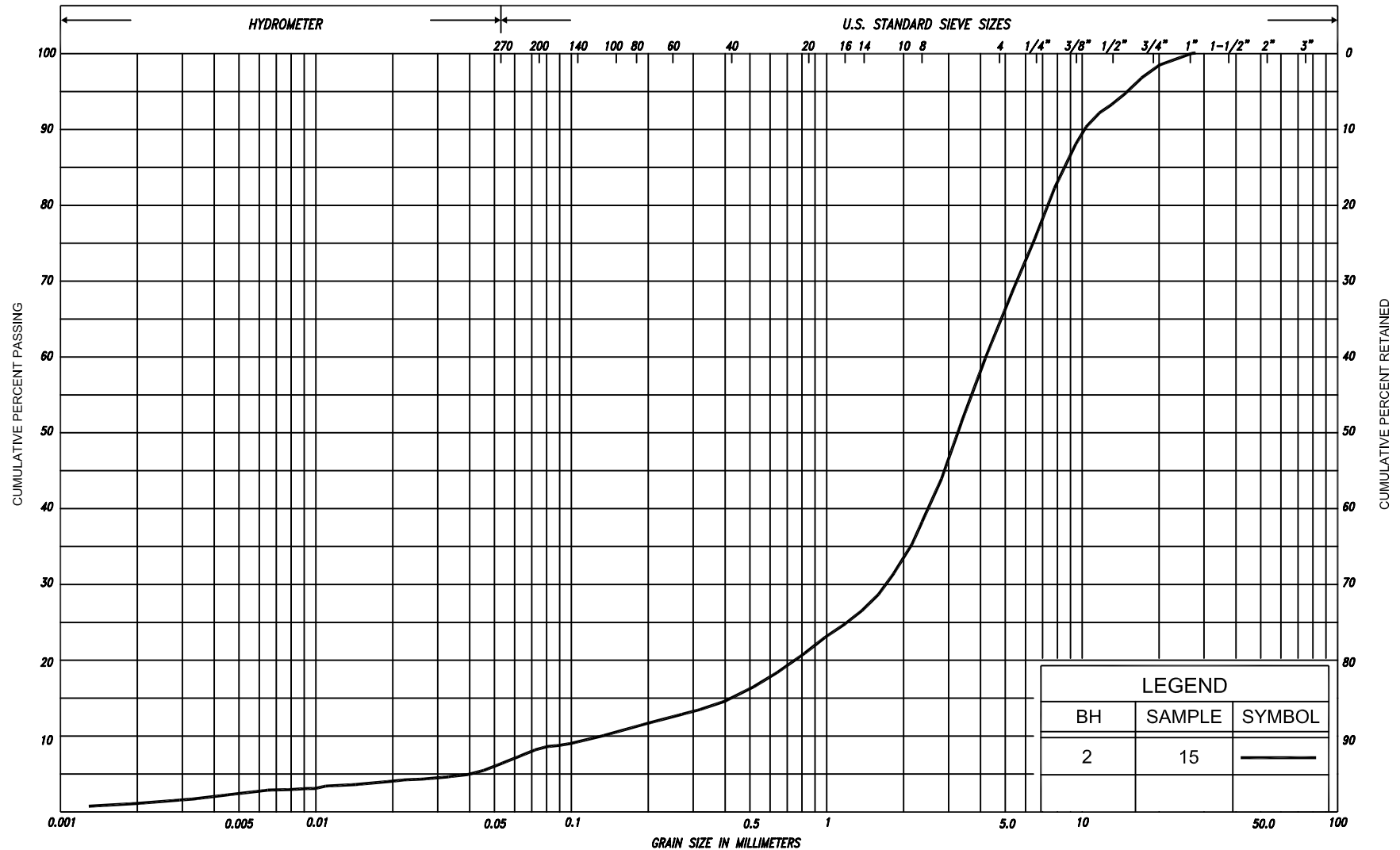
G.W.P. No. 280-99-00



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



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



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



Ministry of  
Transportation  
Ontario

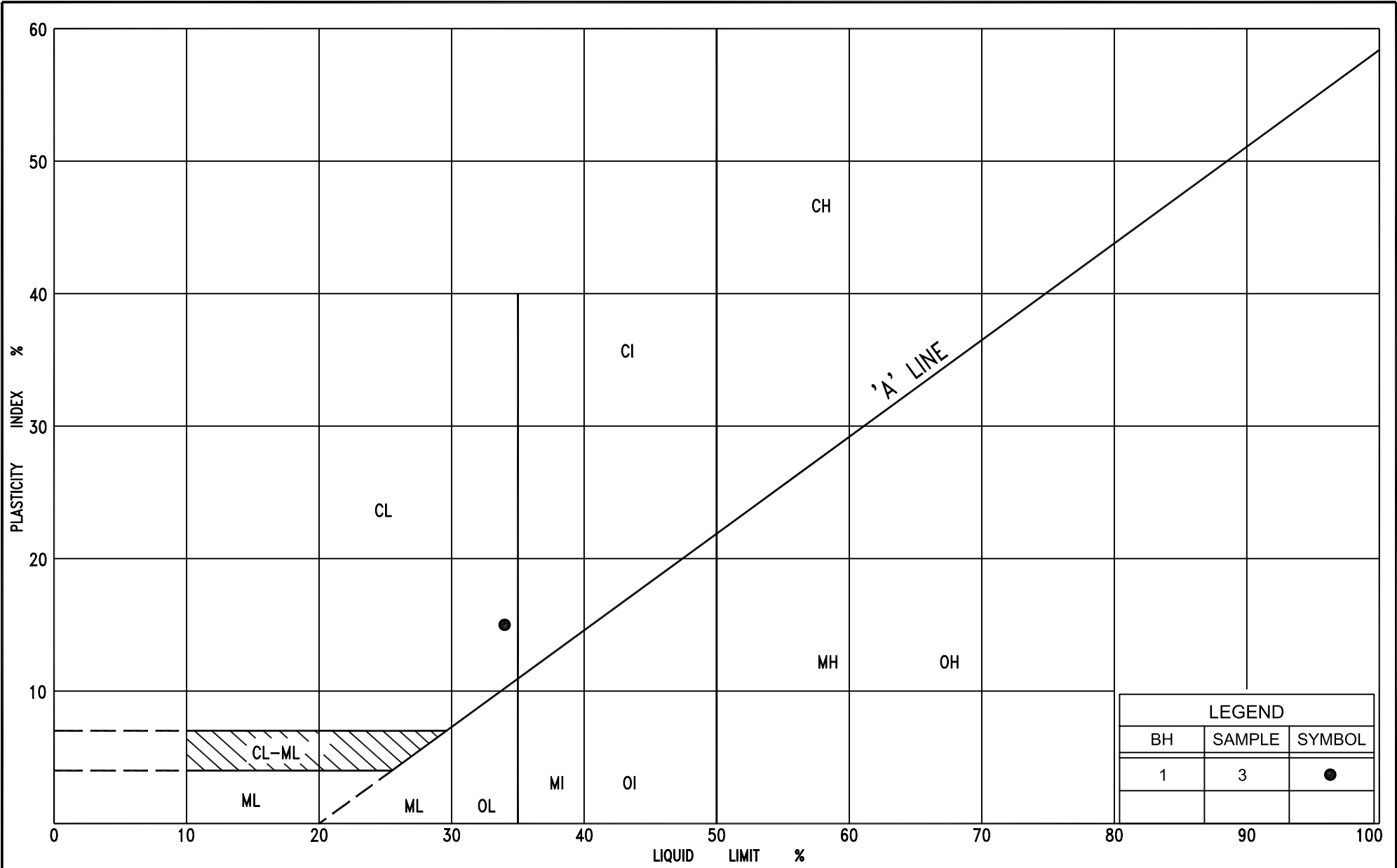
## GRAIN SIZE DISTRIBUTION

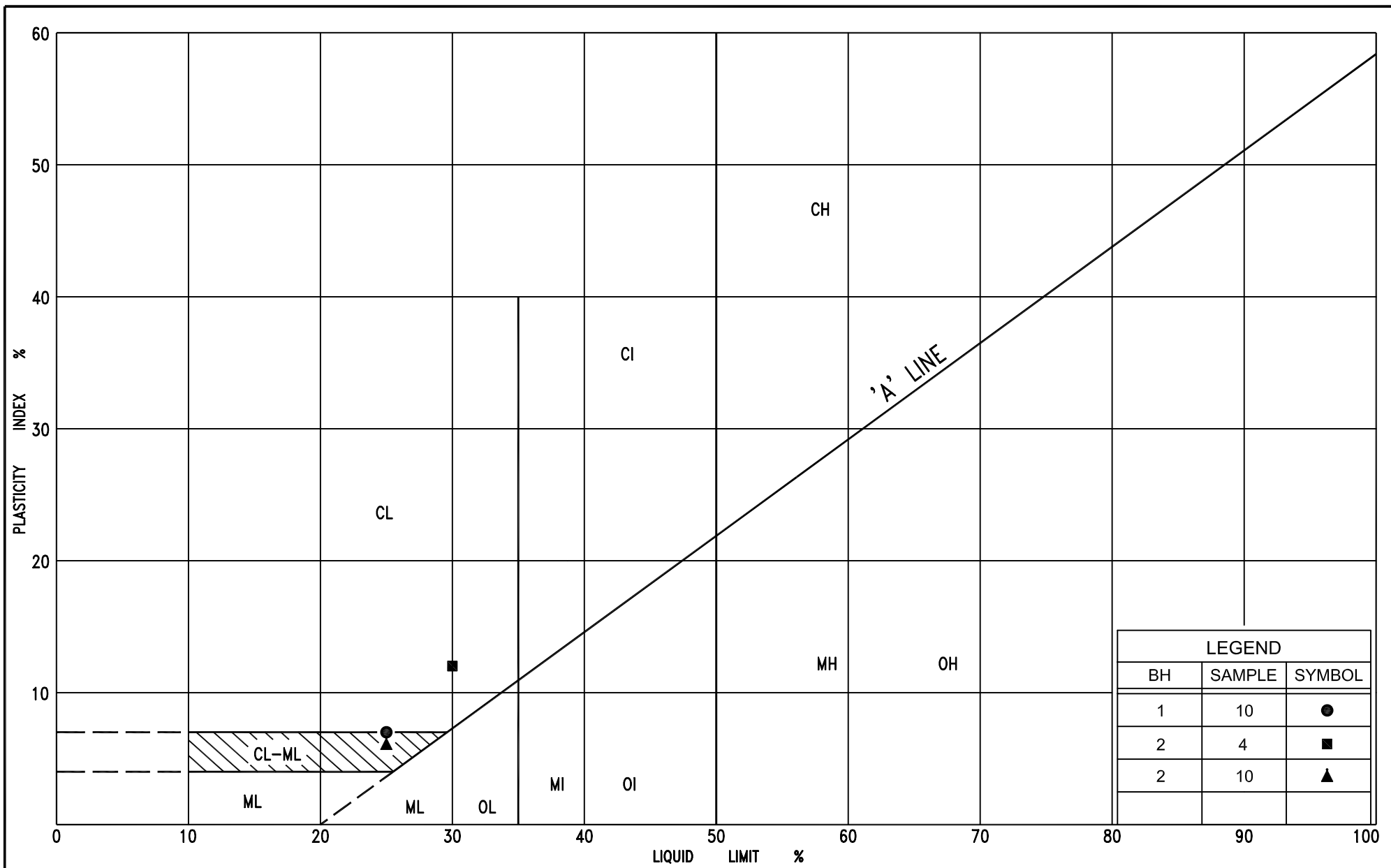
GRAVELLY SAND (SHALE and DOLOSTONE fragments), trace silt, trace clay

FIG No. GS-TR-4

HWY: 406

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

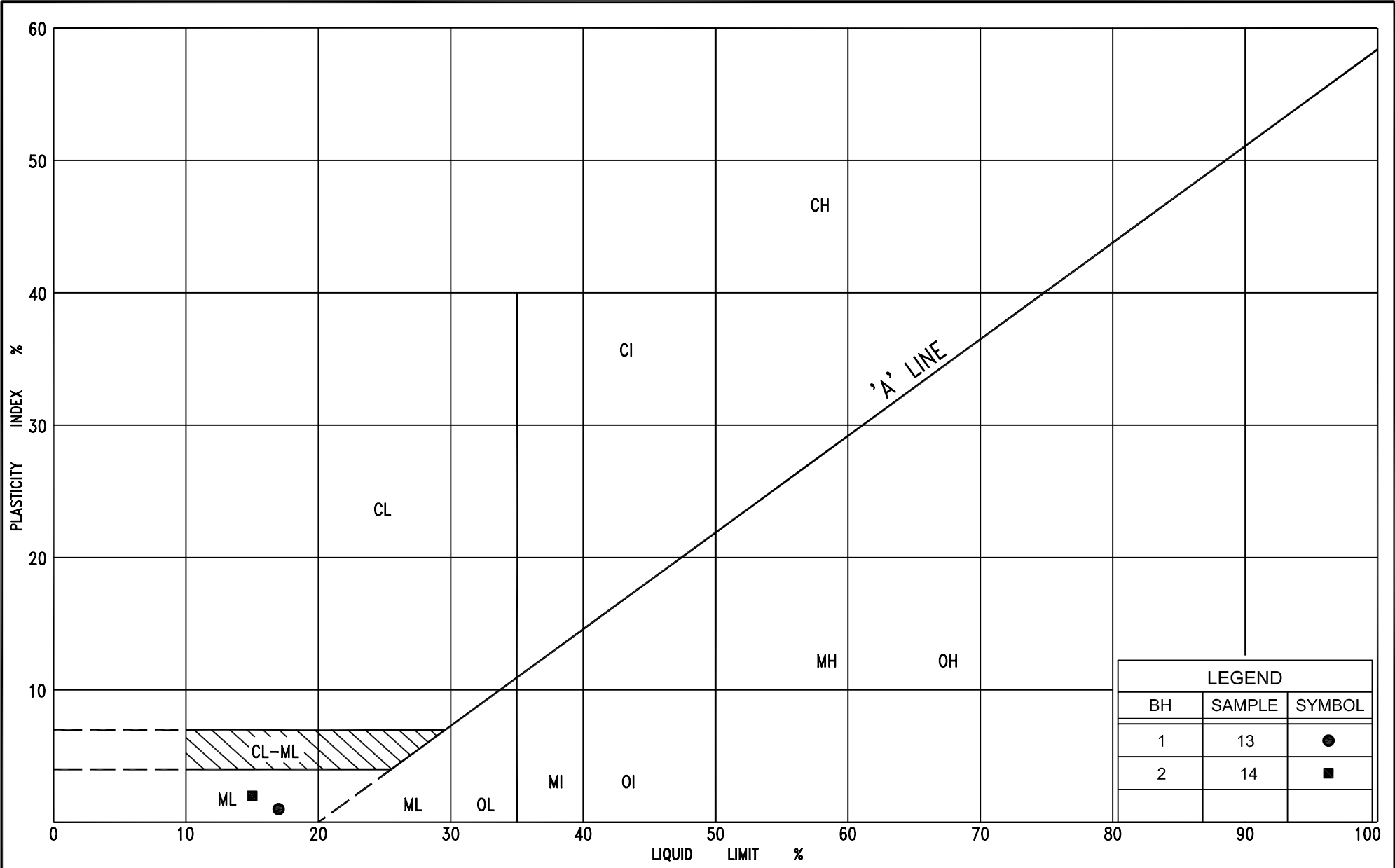




Ministry of  
Transportation  
Ontario

PLASTICITY CHART  
CLAYEY SILT, trace sand, trace gravel  
(TILL)

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



LEGEND		
BH	SAMPLE	SYMBOL
1	13	●
2	14	■

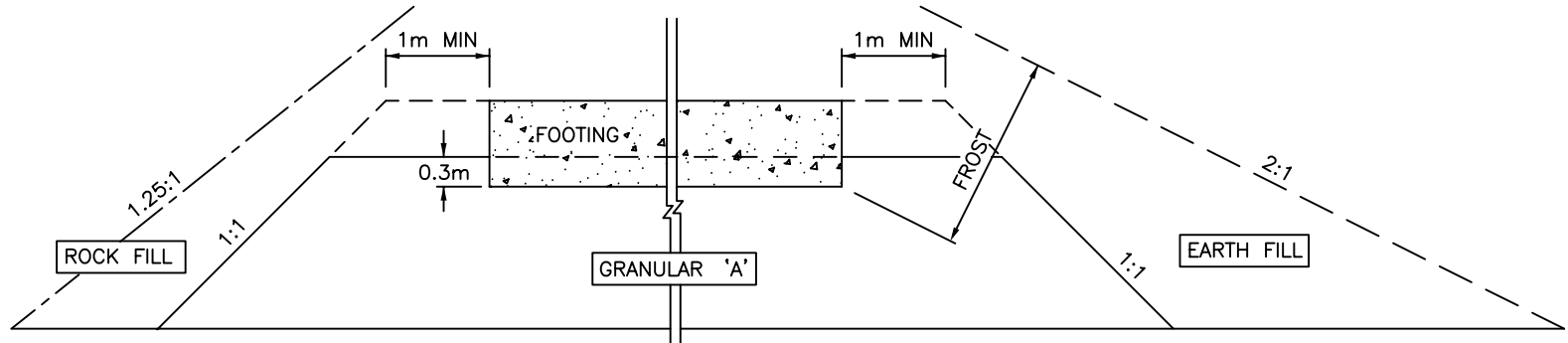


Ministry of  
Transportation  
Ontario

PLASTICITY CHART  
SAND and SILT, some gravel, trace clay  
(TILL)

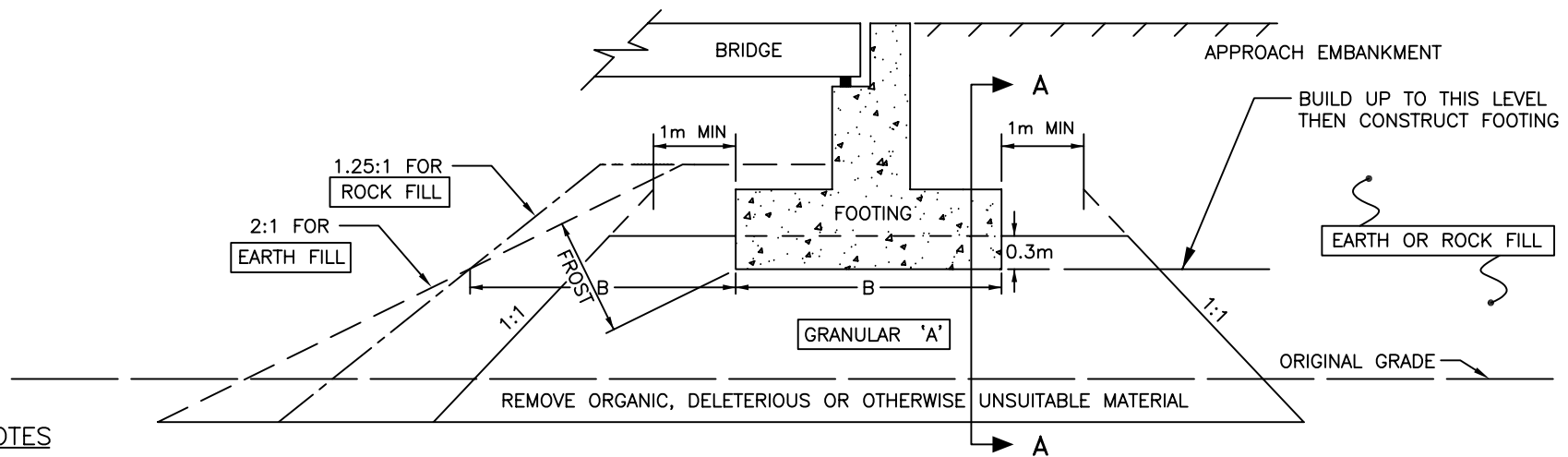
FIG No.	PC-TR-3
HWY:	406
G.W.P. No.	280-99-00





**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 (RQD), FOR MODIFIED RECOVERY, IS:

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

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

**RECORD OF BOREHOLE No 1**

1 of 3

**METRIC**

G.W.P. 280-99-00 LOCATION Co-ords. 4 763 935 N; 327 487 E ORIGINATED BY M.R.  
DIST CR HWY 406 BOREHOLE TYPE C.F.S.S.A. + Rotary Diamond Drilling COMPILED BY P.C.  
DATUM Geodetic DATE November 08, 2001 CHECKED BY D.W.K.




SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE								
183.4	Ground Surface						20	40	60	80	100	20	40	60					
0.0	Clayey silt, trace sand faintly layered thin partings of silt  Very stiff Brown      Moist																		
			1	SS	16								○						
			2	SS	30								○						
			3	SS	23								●	—		0   1   62   37			
			4	SS	23								○						
179.3	Clayey silt trace sand, trace gravel  Stiff to Reddish      Moist very stiff brown (Till)																		
4.1			5	SS	13								○						
			6	SS	12														
			7	SS	20								○						
	Layer of silt		8	SS	11								○						
			9	SS	9														
			10	SS	7								●	—					
									</										

**RECORD OF BOREHOLE No 1**

2 of 3

**METRIC**

G.W.P. 280-99-00 LOCATION Co-ords. 4 763 935 N; 327 487 E ORIGINATED BY M.R.  
DIST CR HWY 406 BOREHOLE TYPE C.F.S.S.A. + Rotary Diamond Drilling COMPILED BY P.C.  
DATUM Geodetic DATE November 08, 2001 CHECKED BY D.W.K.

SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE															
168.4 15.0	Hard		11	SS	18		168								○		10	37	44	9			
			12	SS	32		165								○								
							164																
163.6 19.8	Sand and Silt some gravel, trace clay  Very dense Reddish Moist to dense brown  (Till)						163																
			13	SS	66	162								█									
						161																	
						160																	
						159																	
			14	SS	33	158																	
						157																	
						156																	
						155																	
154.0 29.4	Bedrock Dolostone					154																	
153.4	Cont'd																						

**METRIC**

20  
15 — 5 (%) STRAIN AT FAILURE  
10

**RECORD OF BOREHOLE No 2**

1 of 2

**METRIC**

G.W.P. 280-99-00 LOCATION Co-ords. 4 763 895 N; 327 442 E ORIGINATED BY M.R.  
DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY P.C.  
DATUM Geodetic DATE November 22, 2001 CHECKED BY D.W.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL
182.6	Ground Surface						20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>						
182.0	Topsoil						20	40	60	80	100									
0.2	Clayey silt, trace sand fissured																			
	Stiff to Mottled Moist very stiff Brown		1	SS	13								○							
	distorted lenses of silt		2	SS	19								○							
			3	SS	16								○							
179.8	Clayey silt, trace sand																			
2.8	Very stiff Brown Moist (Till)		4	SS	23								○				0	3	69	28
	layers of brown silt																			
			5	SS	16								○							
	trace gravel																			
	Stiff		6	SS	12								○							
			7	SS	8								○							
			8	SS	10								○				1	3	72	24
			9	SS	8								○							
			10	SS	10								○							

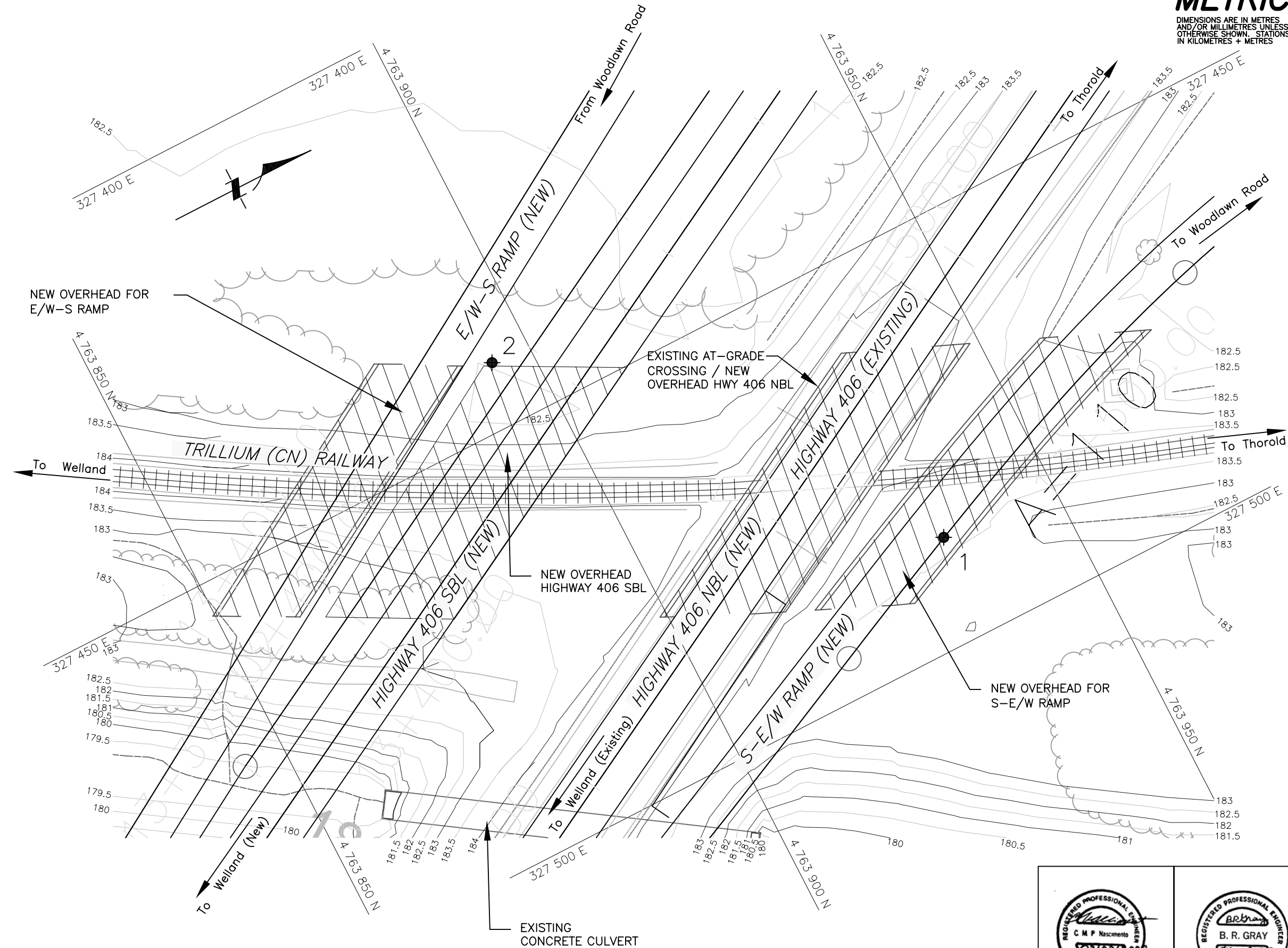
**RECORD OF BOREHOLE No 2**

2 of 2

**METRIC**

G.W.P. 280-99-00 LOCATION Co-ords. 4 763 895 N; 327 442 E ORIGINATED BY M.R.  
DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY P.C.  
DATUM Geodetic DATE November 22, 2001 CHECKED BY D.W.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE								○		
								● QUICK TRIAXIAL		× LAB VANE										
167.6 15.0	Hard		11	SS	12															
			12	SS	38															

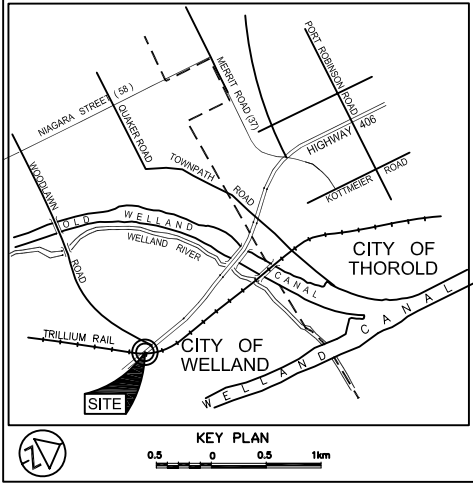


NOTE:  
1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

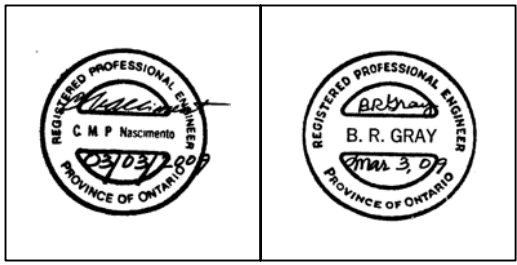


**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES

CONT No	17
GWP No 280-99-00	
TRILLIUM RAILWAY OVERHEADS HIGHWAY 406 BOREHOLE LOCATIONS	
SHEET	



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 November 2001		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
CO-ORDINATES			
BH No	ELEVATION	NORTHINGS	EASTINGS
1	183.4	4 763 935	327 487
2	182.6	4 763 895	327 442



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

REVISIONS								
DATE	BY	DESCRIPTION						
Geocres No. 30M03-239								
HWY No.	406						DIST	7
SUB'N'D	NSB	CHECKED	CN	DATE	MAR. 03, 2009	SITE		--
DRAWN	NA	CHECKED	CN	APPROVED	BRG	DWG		TR-1

REF No. MRC DRAWINGS: PREFERRED-option-22.5m.dwg;  
old-base map-ONE COLOUR.dwg; RECEIVED ON  
SEPTEMBER 25, 2008





## **APPENDIX A**

Previous Data from GEOCRE 30M3-181

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 30M3-181

DIST. 4 REGION

W.P. No. 11-68-10

CONT. No.

W. O. No.

STR. SITE No. 34-307S

HWY. No. 406

LOCATION Hwy 406 & CNR

OVERHEAD

No. of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION**

**WP 11-68-10**

**DIST #4**

**HWY 406**

**STR SITE 34-132-3075**

**C.N.R. OVERHEAD STRUCTURE**

**DISTRIBUTION**

G. C. E. BURKHARDT (3)

R. D. GUNTER

A. WITTENBERG

J. SMRCKA (2)

K. BASSI

J. H. PEER

T. YAKUTCHUK

R. FITZGIBBON (Cover Only)

M. MacLEAN (Cover Only)

FOUNDATION INVESTIGATION REPORT  
For

C.N.R. OVERHEAD STRUCTURE

Hwy. 406, Southbound

W. P. 11-68-10; Site: 34-132-307S

District #4 (Burlington)

INTRODUCTION

A Foundation Investigation was carried out at the aforementioned site between 85 10 24 and 85 11 06. The fieldwork consisted of four sampled boreholes and three dynamic cone penetration tests adjacent to each boring. The boreholes were advanced by a continuous flight auger machine equipped with hollow stem augers and BX size casings.

SITE DESCRIPTION

The structure site is located at the crossing of the C.N.R. tracks and the proposed Hwy. 406, near Cambridge Road and Brown Road corner in the City of Welland. The adjacent land with the exception of the man-made railway embankment and a drainage ditch is relatively flat and in part bush covered.

Physiographically, the site is located in the region referred to as the Haldimand Clay Plain.

SUBSURFACE CONDITIONS

General

In general, the overburden was found to consist of silty clay and silty sand deposits followed by shaley dolostone and shale type bedrock. The boundaries of the different strata, together with the obtained field and laboratory test results are shown on the Record of Borehole Sheets contained in the Appendix of this report. A stratigraphical profile is shown on Drawing No. 116810-A. A description of the different strata encountered is given below.

#### Silty Clay, Traces of Sand and Gravel

This deposit was encountered at every boring location from ground level to a maximum depth of about 21.3 m. The material in the stratum consists of silty clay, traces of sand and gravel. The plasticity in the upper zone is medium, while in the lower zone is only low (See Figure #1). Occasional silt layers were also observed within the main deposit. The physical properties as determined from field and laboratory tests are as follows:

	<u>Range</u>
Natural Moisture Content ( % )	12 - 29
Liquid Limit ( % )	21 - 41
Plastic Limit ( % )	13 - 22
Field Vane Test ( kPa )	55 -109
Sensitivity	2 - 3
Unconfined Shear Strength ( kPa )	87 -121
Unit Weight ( kN/m <sup>3</sup> )	20.4- 21.3

The results of the grain-size distribution tests are shown in an envelope form on Figure #2 of the Appendix.

The undrained shear strength of the overall deposit varies randomly. The consistency may be described as stiff to hard.

#### Silty Sand, Some Gravel and Trace of Clay

This stratum extends from the above described for a maximum depth of about 15.5 m on both sides of the track. The material consists mainly of sand and silt with varying proportions. In addition, gravel and clay sized particles and occasional boulders were also observed. The Natural Moisture Content ranges from 6 to 11%. The results of the grain-size distribution tests are shown in an envelope form on Figure #3 of the Appendix. The Standard Penetration Test 'N' values ranged from 12 to over 100 blows per 30 cm. Occasionally 'Boiling' of the material was observed. Some of the obtained lower 'N' values may be attributed to this fact. Based on the 'N' values, the state of the overall deposit varies from compact to very dense.

Bedrock - Shaley Dolostone to Shale

The bedrock was proved in two boreholes (#1, and #3) by obtaining AX size core samples. In these borings, the upper surface of the bedrock was found to be at El. 153.4 and El. 154.5 in #1 and #3 respectively.

The core samples were examined by Mr. E. R. Magni, MTC Geologist, and his description is appended to this report.

Groundwater Conditions

The following groundwater levels were observed during the field investigation:

BH. #1	El. 181.2
BH. #2	El. 181.2
BH. #3	El. 181.1
BH. #4	Not Observed

APPENDIX

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$ .

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

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

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

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

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{v0}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_r$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_f$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kn/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kn/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kn/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kn/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

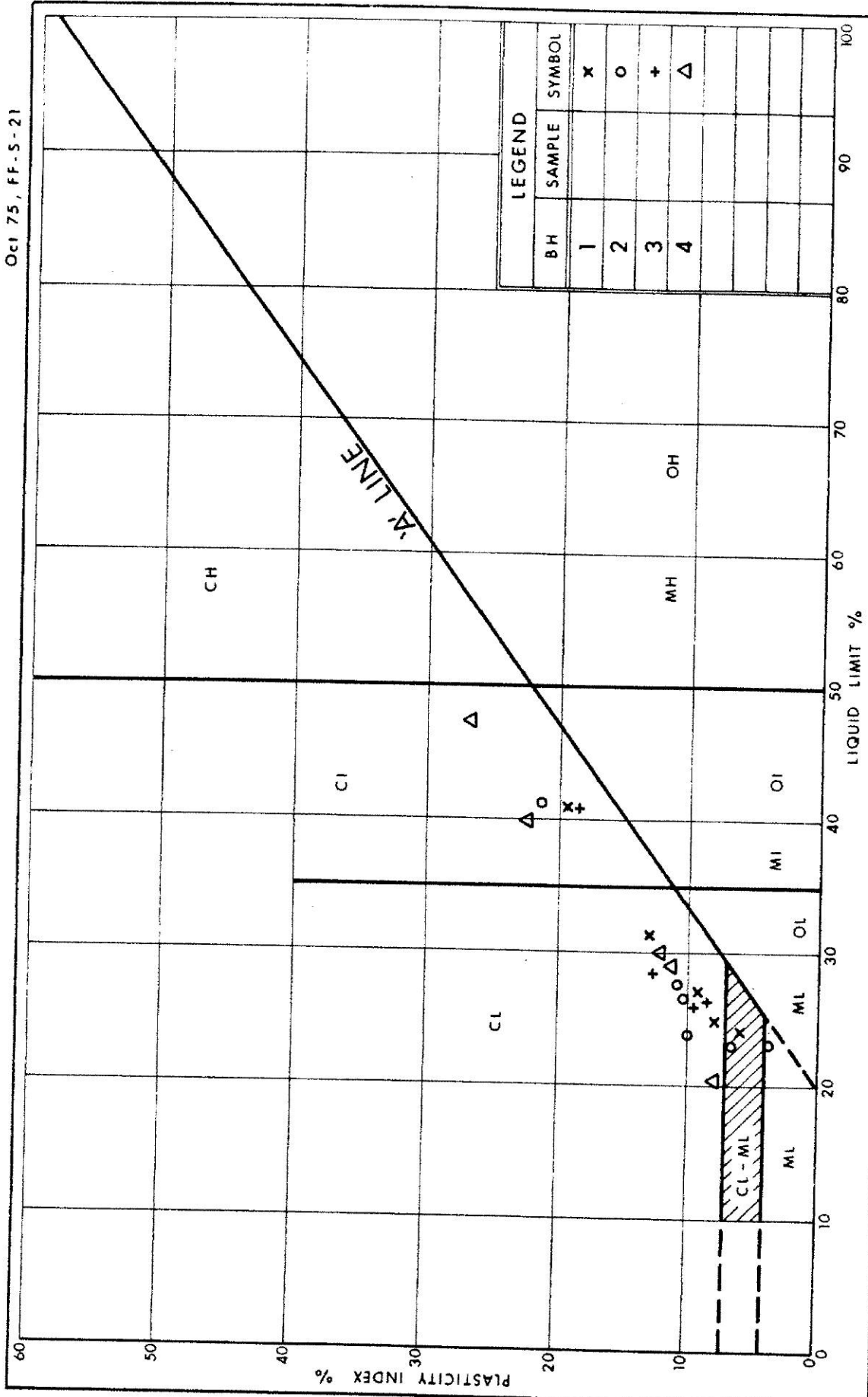


# DESCRIPTION OF ROCK CORE - W.P. 11-68-10

BOREHOLE NUMBER	CORE DESCRIPTION			
	DEPTH (m)	% CR *	% RQD *	DEPTH (m)
1	29.11-30.48 -31.27 -32.00	70 71 76	0 0 0	29.11-32.00
	SHALEY DOLOSTONE, buff alternating with SHALE, dark green grey, with zones of gypsum (.23), slightly weathered with highly weathered and high core loss zones, very closely spaced joints.			
3	28.55-30.07 -31.60	43 70	0 0	28.55-31.6
	SHALEY DOLOSTONE, buff alternating with SHALE, dark green grey, slightly weathered with highly weathered zones, high core loss zones apparently due to poor drilling.			

\* CR = CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

Oct 75, FF-S-21

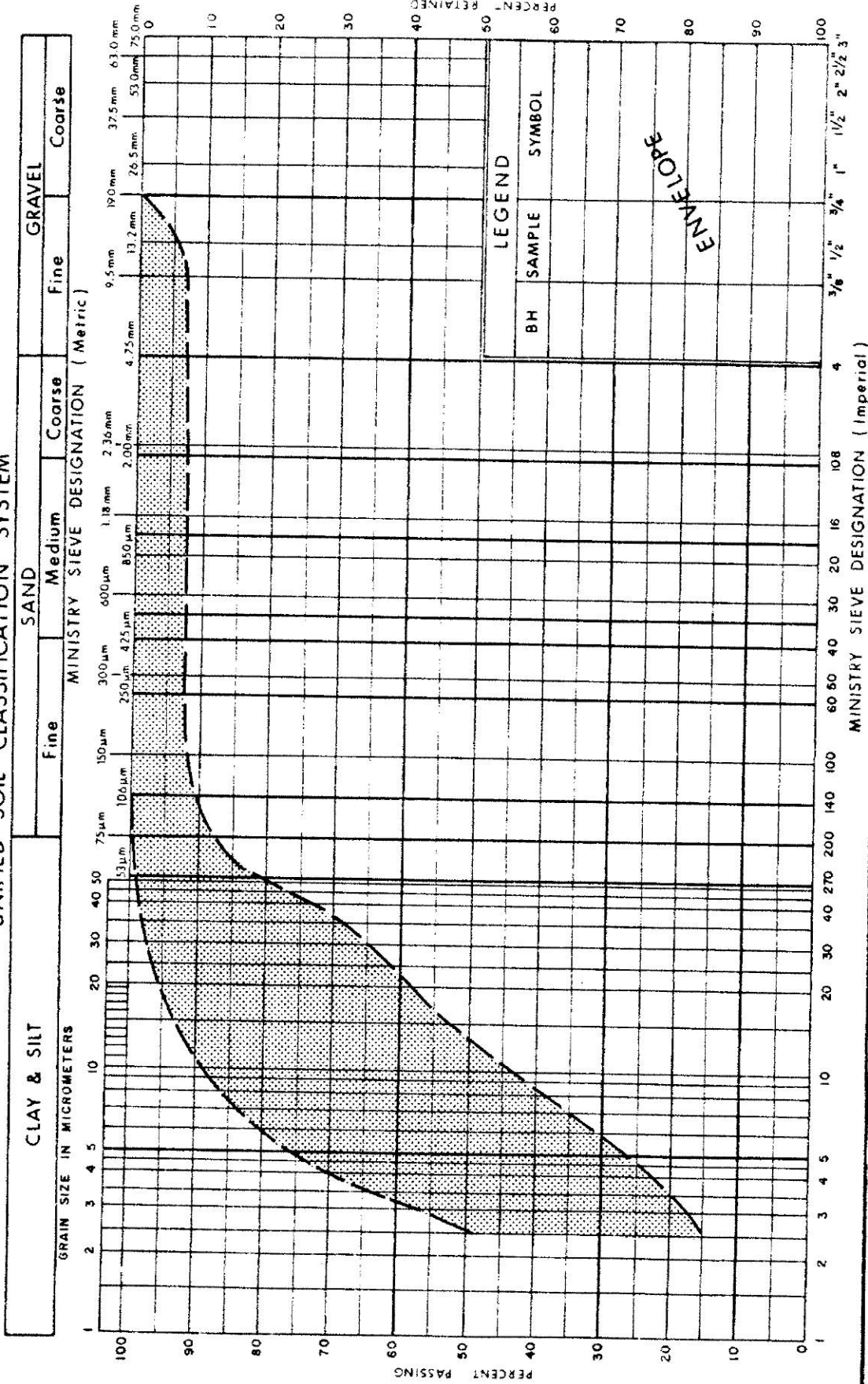


# PLASTICITY CHART SILTY CLAY

FIG No 1

W P 11-68-10

# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation and  
Communications



## GRAIN SIZE DISTRIBUTION SILTY CLAY

FIG No 2

W P 11-68-10

78 12 M

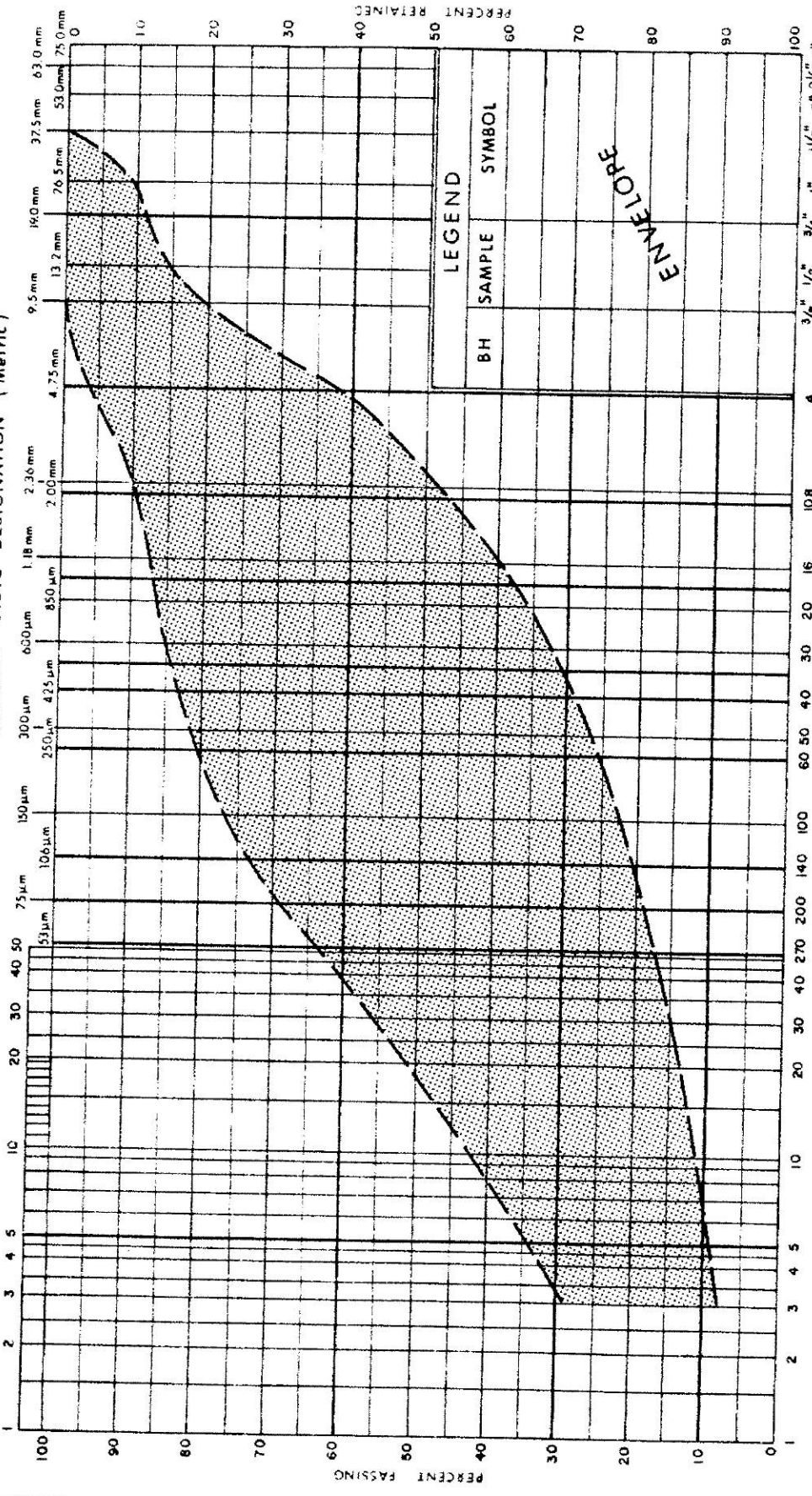
# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT  
GRAIN SIZE IN MICROMETERS

SAND  
Fine

Medium Coarse  
MINISTRY SIEVE DESIGNATION (Metric)

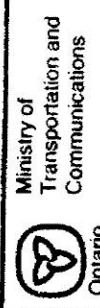
Fine Coarse  
GRAVEL



LEGEND

BH SAMPLE SYMBOL

ENVELOPE



Ministry of  
Transportation and  
Communications

## GRAIN SIZE DISTRIBUTION

SILTY SAND

FIG No 3

W P 11-68-10

# RECORD OF BOREHOLE No 1

METRIC

W P 11-68-10 LOCATION CO-ORDS: N 4 763 751.9; E 327 425.7  
 DIST 4 HWY 406 BOREHOLE TYPE Cont. Flight Auger (H.S.) & BX Casing  
 DATUM Geodetic DATE 85 11 24 to 29

ORIGINATED BY S.C.  
 COMPILED BY P.P.  
 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
182.5	Ground Level												
0.0													
	Silty Clay		1	SS	26		182						0 2 56 42
	Traces of Sand & Gravel		2	SS	23		180						0 2 69 29
			3	SS	66								
			4	SS	32								
			5	SS	16								
	Stiff to Hard		6	SS	22		178						0 1 84 15
			7	SS	31								
			8	SS	30		176						
			9	SS	23								
			10	SS	22		174						0 2 72 26
			11	SS	14								
			12	SS	14		172						
			13	SS	PH								
			14	SS	32		170						2 2 76 20
167.9													
14.6													
	Silty Sand		15	SS	56		168						
	Some Gravel		16	SS	193	20 cm	166						28 33 30 9
	Trace of Clay		17	RC	xx								
	Occ. Boulders						164						
	Dense to Very Dense		18	SS	165		162						
							160						27 38 30 5
			19	SS	152		158						
							156						
			20	SS	37		154						
155.1	Cont'd.												
27.4													

+3, x<sup>5</sup>: Numbers refer to  
 Sensitivity  
 20  
 15 5 (%) STRAIN AT FAILURE  
 10

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 1 Cont

METRIC

W.P. 11-68-10 LOCATION CO-ORDS: N 4 763 751.9; E 327 425.7  
 DIST 4 HWY 406 BOREHOLE TYPE Cont. Flight Auger (H.S.) & BX Casing  
 DATUM Geodetic DATE 85 11 24 to 29  
 ORIGINATED BY S.C.  
 COMPILED BY P.P.  
 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
155.1	Cont'd.												
27.4													
153.4							154						
29.1	Shaley Dolostone and Shale Slightly to Highly Weathered		21	RC AX	Rec 70%		152						
150.5	Bedrock		22	RC AX	Rec 71%								
32.0	End of Borehole		23	RC AX	Rec 76%		150						

OFFICE REPORT ON SOIL EXPLORATION

\*3, x5: Numbers refer to  
Sensitivity

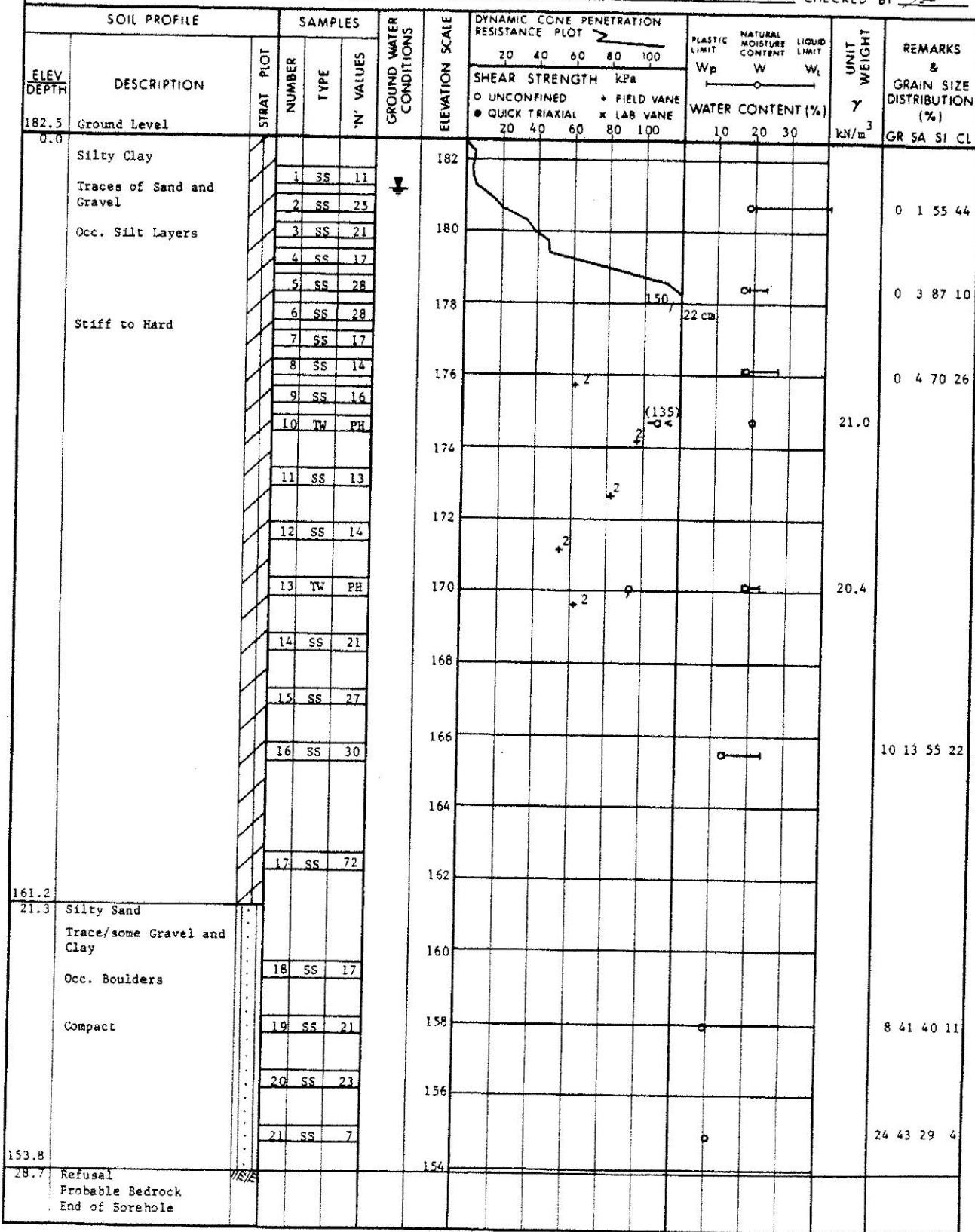
20  
15 5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE No 2

METRIC

W P 11-68-10 LOCATION CO-ORDS: N 4 763 730.1; E 327 447.4  
 DIST 4 HWY 406 BOREHOLE TYPE Cont. Flight Auger (H.S.) & BX Casing  
 DATUM Geodetic DATE 85 10 30 and 31  
 ORIGINATED BY S.C.  
 COMPILED BY P.P.  
 CHECKED BY



+3, x5: Numbers refer to Sensitivity  
 20  
 15  
 10  
 5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION







## RECORD OF BOREHOLE No 3 Cont

METRIC

W P 11-68-10

LOCATION

CO-ORDS: N 4 763 677.2; E 327 474.7

DIST 4

HWY 406

BOREHOLE TYPE

Cont. Flight Auger (H.S.) and BX Casing

ORIGINATED BY S.C.

DATUM Geodetic

DATE

85 11 01, 04, 05 and 06

COMPILED BY P.P.

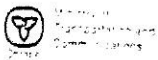
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
155.7	Cont'd.		22	SS	41										GR SA SI CL 14 42 40 4
27.4															
154.5															
28.6	Shaley Dolostone and Shale Slightly to Highly Weathered		23	RC AX	Rec 43%		154								
151.5	Bedrock		24	RC AX	Rec 70%		152								
31.6	End of Borehole						150								

+3, x5: Numbers refer to Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION



## RECORD OF BOREHOLE No 4

METRIC

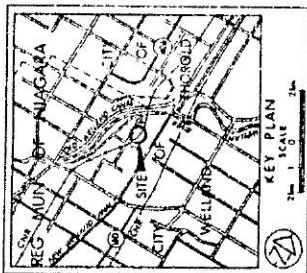
W P 11-68-10 LOCATION CO-ORDS: N 4 763 634.7; E 327 488.5 ORIGINATED BY S.C.  
DIST 4 HWY 406 BOREHOLE TYPE Cont. Flight Auger (H.S.) COMPILED BY P.P.  
DATUM Geodetic DATE 85 11 01 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W	LIQUID LIMIT Wl	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
183.7	Ground Level											
0.0	Silty Clay											
	Trace of Gravel		1	SS	17							0 8 49 43
	Trace/some Sand		2	SS	40							
			3	SS	41							
			4	SS	32							
			5	SS	19							
	Stiff to Hard		6	SS	19							0 0 52 48
			7	SS	22							
			8	SS	17							
			9	SS	19							0 2 77 21
			10	SS	16							
			11	TW	PH						20.6	2 4 60 34
			12	SS	19							
			13	SS	22							
			14	SS	24							
			15	SS	29							3 28 44 25
166.5			16	SS	73							
17.2	End of Borehole											
	* Groundwater Level not observed											

+3, x5: Numbers refer to Sensitivity

20  
15 → 5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION



# LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N
- Blow/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation 1965 II

No	ELEVATION	CO ORIGINATES
1	182.5	4763751.9 327 425.7
2	182.5	4763750.1 327 447.4
3	183.1	4763672.3 327 474.7
4	183.7	4763634.7 327 488.5

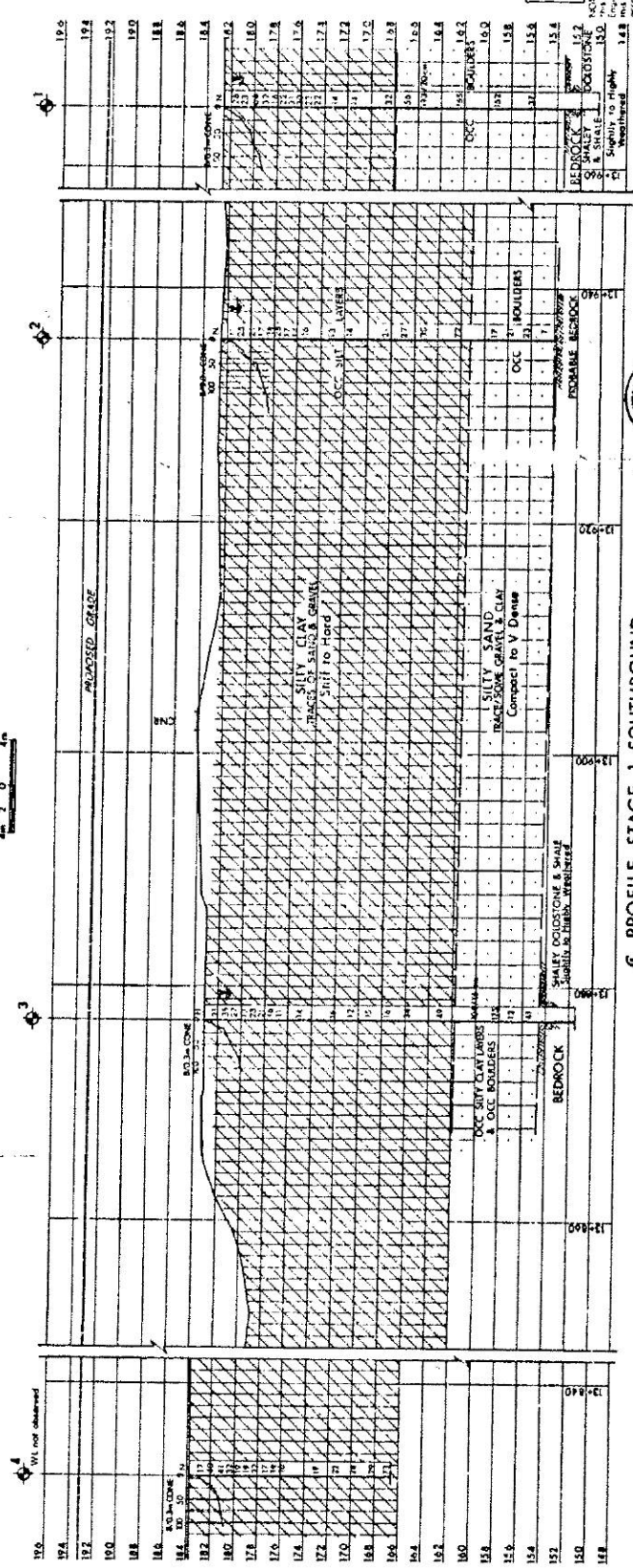
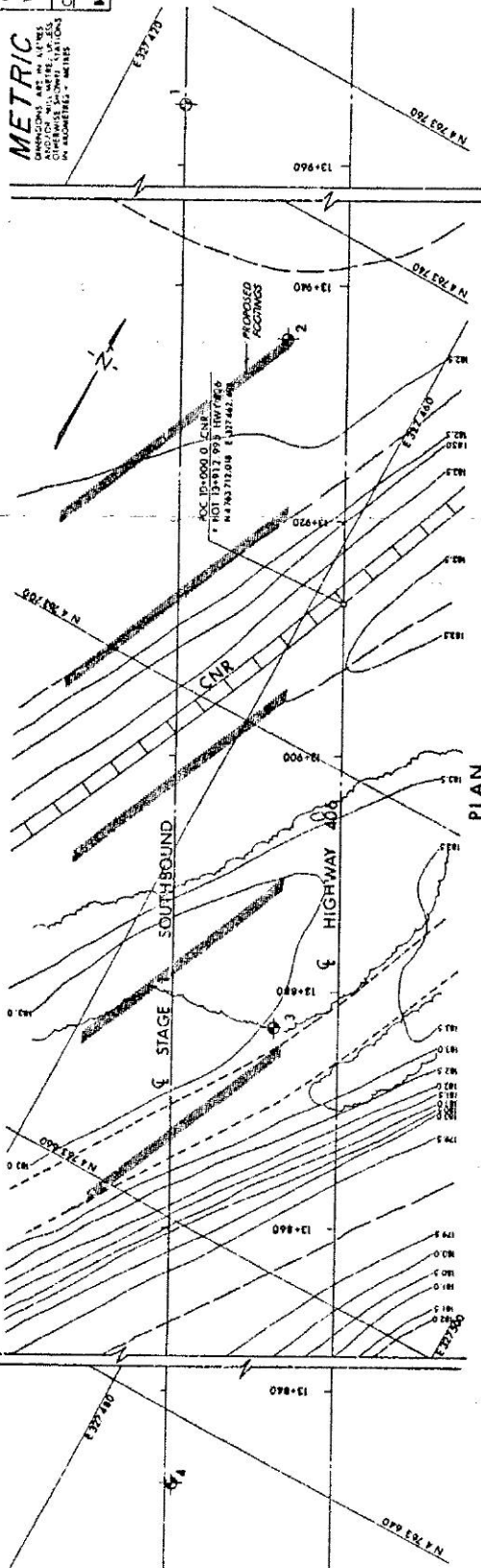
NOTE: The boundaries between and strata have been established only at the bore hole locations. Between bore holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and sample must be made at the time of investigation. The investigation must be made at the time of investigation. The investigation must be made at the time of investigation.

GENERAL REF 3043-181

DATE 1971

DESCRIPTION



## PROFILE SCALE 1 SOUTHBOUND

SCALE 1:2000

REF NO E-6059-1 83 08

