



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for
WOODLAWN ROAD OVERPASS
HIGHWAY 406 FOUR-LANING
GWP 280-99-00
CITY OF WELLAND, ONTARIO

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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 proposed Woodlawn Road Overpasses over 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 bridges 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 new overpass consists of two structures and will carry the Highway 406 northbound and southbound lanes over the realigned Woodlawn Road approximately 50 m north of the existing at-grade intersection.

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

2. SITE DESCRIPTION AND GEOLOGY

The contemplated structures are proposed about 50 m north of the existing Woodlawn Road and Highway 406 at-grade crossing. The site is about 3,200 m north of the existing East Main Street intersection at Highway 406.



Land use in the vicinity of the site comprises the existing Woodlawn Road and Highway 406 at-grade intersection. A golf course and residential areas are present north east of Woodlawn Road (along Daimler Parkway which adjoins Woodlawn Road) and about 200 m north of Highway 406, respectively. At the proposed structure location, Highway 406 runs roughly north to south.

The local topography of the structure site is relatively flat. 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 and undulating. The soil cover in the region typically comprises lacustrine silts and clays. Dolostone bedrock of the Salina Formation is anticipated at an approximate depth of 25 m.

3. INVESTIGATION PROCEDURES

The field work was carried out on November 7 and 15, 2001. The interchange configuration being used for preliminary design purposes at that time was generally consistent with the current preferred configuration. Two sampled boreholes were put down at the site. The boreholes were drilled to refusal at depths of 25.0 and 25.9 m at the locations shown on Drawing WL-1.

Borehole 1 located in the general area of the proposed northbound overpass was extended below refusal by coring 6.1 m into bedrock to a total depth of 31.1 m. Borehole 2 located in the southbound bridge area was advanced an additional 4.6 m into bedrock by power augering into the weak/shattered bedrock.

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

BENCHMARK	ELEVATION (*)
Cut cross in northwest corner of concrete culvert, west side of Highway 406, north of Trillium Railway (previous CNR) 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 a truck-mounted CME-75 drill rig (borehole 1) and track-mounted CME-75 Bombardier (borehole 2) drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

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

In borehole 1, casing was extended to the bedrock surface and an approximate 6.1 m length of rock core was recovered using NXL 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 (28)
- Grain Size analyses (5)
- 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-WL-1 to GS-WL-3. The Atterberg limits results are presented on Figures PC-WL-1 to PC-WL-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 Woodlawn Road overpass are presented on the attached Foundation Drawing WL-1.

The subsurface stratigraphy revealed in the two boreholes generally consisted 3.7 and 5.5 m thick discontinuous deposits of clayey silt and silt (overlain by 300 mm thick Highway 406 shoulder granular fill in borehole 1) overlying 20.4 and 21.0 m thick glacial deposits which included clayey silt till interbedded/underlain by sandy silt till units. The soil cover had a typically stiff/very stiff consistency or dense to very dense relative density. Dolostone bedrock was contacted below the soil cover at depths of 25.0 and 25.9 m (elevations 156.9 and 157.2). The strata encountered are summarized below.



4.2 Fill

A 25 mm thick layer of tar and chip over 250 mm of Granular 'A' crushed limestone fill was encountered surficially in borehole 1 drilled on the existing Highway 406 shoulder. The unit extended to an approximate depth of 0.3 m (elevation 181.6).

4.3 Clayey Silt

Cohesive deposits of clayey silt were present below the fill in borehole 1 at a depth of 0.3 m (elevation 181.6) and surficially in borehole 2.

The stratum was 3.7 m thick extending to the underlying clayey silt till and cohesionless silt at depths of 4.0 and 3.7 m (elevations 177.9 and 179.4) in boreholes 1 and 2, respectively. The clayey deposits typically exhibited very stiff consistency with hard local layers in borehole 1. N values ranged from 20 to 32.

The grain size distribution chart of a representative sample of the clayey silt is shown on Figure GS-WL-1 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-WL-1. The liquid limits of the clayey silt were 26 and 32 and the plastic limits 19 and 20, giving the plasticity index values of 6 and 13. The water content of the clayey silt varied from 19 to 21%.

4.4 Silt

A discontinuous 1.8 m thick layer of non-cohesive silt was found directly beneath the clayey silt at 3.7 m depth (elevation 179.4) in borehole 2 extending to 5.5 m depth (elevation 177.6). The silt had a compact relative density with N values of 20 and 27. The laboratory moisture content of the silt was about 20%.

4.5 Clayey Silt Till

A deposit of cohesive clayey silt till was encountered below the clayey silt and silt deposits at depths of 4.0 and 5.5 m (elevations 177.9 and 177.6) in boreholes 1 and 2, respectively.



This unit was 13.1 and 11.5 m in thickness and extended to underlying sandy till at depths of 17.1 and 17.0 m (elevations 164.8 and 166.1).

A discontinuous lower layer of 3.0 m thick clayey silt till was found in borehole 2 at depths between 20.0 and 23.0 m (elevations 163.1 and 160.1).

The consistency of the clayey silt till was typically stiff in borehole 1 becoming very stiff to hard with depth. The results of vane shear testing conducted in this stratum at depths of 7.0 to 11.5 m indicate that the undisturbed shear strength values ranged from 85 to 160 kPa respectively (soil sensitivity is about 2). Penetrometer tests values on three samples were 75 and 150 kPa. N values ranged from 6 to 43, typically in the range of 8 to 15.

The grain size distribution chart of a representative sample of the clayey silt till is shown on Figure GS-WL-2 and the Atterberg plasticity limits of two samples on the Plasticity Chart Figure PC-WL-2. The liquid limits of the clayey silt till were 23 and 28 and the plastic limits 15 and 18, giving the plasticity index values of 8 and 10. The moisture content of this till deposit ranged from 13 to 21%, typically 18 to 21%, indicating soils with low compressibility characteristics.

4.6 Sandy Silt Till

A cohesionless deposit of glacial till comprising dense to very dense sandy silt till was encountered in both boreholes below the clayey silt till deposit at depths of 17.1 and 17.0 m (elevations 164.8 and 166.1). The N values in this till deposit typically ranged from 42 to 67 for 10 cm penetration of the sampler.

The cohesionless till contained variable gravel content. Scattered cobbles were encountered. Buried boulders are also anticipated in this deposit although not encountered in the boreholes.

The thickness of this till deposit was 7.9 and 5.9 m extending to underlying bedrock at depths of 25.0 and 25.9 m (elevations 156.9 and 157.2) in boreholes 1 and 2, respectively. The unit was interbedded by a 3.0 m thick layer of very stiff to hard clayey silt till between 20.0 and 23.0 m depths (elevations 163.1 and 160.1) in borehole 2.



The grain size distribution charts of representative samples of the till are shown on Figure GS-WL-3 and the Atterberg plasticity limits on the Plasticity Chart Figure PC-WL-3. The liquid limits of this till were 14 and 16 and the uniform plastic limit 13, giving the plasticity index values of 1 and 3 indicating very low plasticity characteristics. The water content of the till was between 6 and 7%.

4.7 **Bedrock**

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

BOREHOLE No.	DEPTH (m)	ELEVATION	ROCK CORE LENGTH (m) (*)
1	25.0	156.9	6.1
2	25.9	157.2	(**)

(*) NXL diamond rock cores obtained.

(**) Augered 5.4 m with difficulty into bedrock.

The core recovery was 83 and 100% for the core samples 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% for all core samples). Loss of drilling water was experienced during drilling and an about 23 cm deep void was encountered at 25.4 m depth. The detailed rock core descriptions are provided on Table A.

It is noted that borehole 2 was advanced through bedrock by power augering. The rock was found to be very weak and shattered. Loss of drilling water was also experienced in borehole 2 and a possible void was detected at 26.8 m depth during the augering.



4.8 Groundwater

Upon completion of drilling groundwater was measured in borehole 2 at a depth of 13.5 m (elevation 169.6). The groundwater table was not determined upon completion of drilling in borehole 1 (NBL overpass) because the borehole was charged with water for rock coring purposes.

The groundwater levels at the site 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.

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

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
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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 Woodlawn Road Overpass at the Highway 406 in the City of Welland, Ontario. The recommendations are preliminary and based on the results of the limited subsurface investigation that was outlined in the Part A of this report.

Based on the preliminary lay-out drawing provided by MRC, the proposed overpass will comprise two single-span bridges of 20.0 (NB) and 12.7 m (SB) width, with a total length of approximately 22.8 m between abutments. Approach embankments will be about 8 m high at the abutments of the northbound and southbound structures.

In summary, the subsurface stratigraphy revealed in the two boreholes generally consisted 3.7 and 5.5 m thick discontinuous deposits of clayey silt and silt (overlain by 300 mm thick Highway 406 shoulder granular fill in borehole 1) overlying 20.4 and 21.0 m thick glacial deposits which included clayey silt till interbedded/underlain by sandy silt till units. The soil cover had a typically stiff/very stiff consistency or dense to very dense relative density. Dolostone bedrock was contacted below the soil cover at depths of 25.0 and 25.9 m (elevations 156.9 and 157.2).

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

The pile lengths will vary in view of the variable relative density of the dense to very dense and over 5.9 m thick sandy silt till stratum which mantles the bedrock. It is likely that some of the piles will be driven through the glacial till and to the bedrock and other piles will find refusal in the denser glacial till. The geotechnical resistance of the deep foundations should be designed for



bearing on the glacial till layer rather than on the bedrock for preliminary design purposes. The piles should be provided with rock points due to potentially heavy driving into the till deposit which likely contains cobbles and boulders.

In addition, the clayey soils that will support the approach embankments may experience consolidation settlement in the order of 60 mm due to 8 to 10 m high approach embankment fill loading. Further 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 overpass location during the Detail Design phase of the project. The foregoing "red-flag" issues and the interpretation and recommendations in this report are only provided for planning purposes and feasibility studies.

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

6.2 Foundations

6.2.1 General

Based on the preliminary data, founding the proposed structures on pile foundations driven to practical refusal on the dense to very dense sandy silt till or bedrock is considered feasible. Lightly loaded footings placed on the native soils or on engineered fill may be used for semi-integral or conventional abutment design.

Drilled caissons bearing on the glacial till or on the bedrock to support the structures are not considered to be practical due to the presence of cobbles and boulders in the till, cohesionless silt layers as well as a relatively high groundwater table at the anticipated founding level.

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



6.2.2 Deep Foundations

6.2.2.1 General

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

The preliminary pile foundation design recommendations for conventional and semi-integral abutments are provided on the following section. Additional recommendations for integral abutment foundations are provided in Section 6.2.2.3.

6.2.2.2 Conventional Abutment Considerations

Piles for the abutments should be driven to refusal into very dense silt till. Estimated reference levels of sandy silt till are provided in the following table:

FOUNDATION ELEMENT	BOREHOLE No.	FOUNDING DEPTH (m)	FOUNDING ELEVATION	ESTIMATED PILE TIP ELEVATION
NBL Overpass North Abutment	1	21.5	160.4	158.4
SBL Overpass South Abutment	2	23.0	160.1	158.1

Note: For preliminary design purposes, the founding levels for driven piles at the south abutment of the NBL overpass and north abutment of the SBL overpass should be taken as those of the adjacent founding element since the bedrock surface and soil stratigraphy are relatively consistent in the boreholes.

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

The piles will have to be driven through native soils containing compressible clayey soils at the abutment locations. The existing grade at the abutments will be raised about 8 to 10 m above the existing ground levels. Consequently, the development of negative skin friction on the piles should be considered to the axial resistance 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.

Based on the anticipated very dense silt till at the pile tips (and low to medium strength bedrock with voids for piles reaching the bedrock level), the preliminary factored axial resistance at ultimate limit states (ULS) for a steel HP 310x110 pile is 1,600 kN. The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the maximum 25 to 28 m pile length required, the design is not expected to be governed by settlement since the required load causing that magnitude of deformation of the pile is larger than the ULS factored capacity.

The capacity of the HP 310x110 piles should be reduced to allow for negative skin friction of 200 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 hard soils and into very dense glacial till soils or may set on the bedrock found under the abutments. Consequently, the piles should be equipped with steel H-pile driving shoes (OPSD-3000.100) or the Titus H Bearing Pile Point Standard Model (SP 903S01) to minimize the potential for damage when driving through the glacial till soils containing cobbles and boulders. All piles should be re-tapped to ensure adequate seating into the glacial till or on bedrock in view of the potential presence of voids in 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 the integral abutment design, the H-piles should be driven to very dense silt till or bedrock anticipated at the depths/elevations and axial resistance are indicated in the previous section. The minimum 5.0 m long pile length below the abutment stem 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 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 assessed lateral resistance for the HP 310x110 pile section noted previously is as follows:

	CLAYEY SILT	CLAYEY SILT TILL	SANDY SILT TILL	GRANULAR BACKFILL 'A' OR 'B' TYPE II
Factored Lateral Resistance at ULS, kN	200	160	120	120
Lateral Resistance at SLS, kN	110	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/m³) should be computed using the following equation:

Cohesionless Soils (Terzaghi, 1955)

$$\begin{aligned} k_s &= n_h z/b \\ \text{where } n_h &= \text{coefficient related to soil density} \\ &= 10.0 \text{ MN/m}^3 \text{ for granular backfill} \\ &= 1.3 \text{ MN/m}^3 \text{ for sandy silt till (below water table)} \\ z &= \text{depth, m} \\ b &= \text{pile width, m} \end{aligned}$$

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

The coefficient of horizontal subgrade reaction, k_s , for the native clayey silt/clayey silt till units should be taken as 28,000 kN/m³ for preliminary purposes.

6.2.3 Shallow Foundations

6.2.3.1 Spread Footings on Native Soil

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

Spread footings should be constructed on the typically very stiff clayey silt soils at or 1.2 m below elevation 181.9 for the south abutments (based on borehole 2), elevation 180.7 for the north abutments (based on borehole 1). The recommended bearing resistance for minimum 2.0 m wide footings constructed on the native soils is as follows:

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

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m and groundwater level below founding depth was assumed for computation of the ULS resistance.



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

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

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

6.2.3.2 Spread Footings On Structural Fill

Construction of the abutment footings on structural fill placed in the approach embankment could also be employed to support the foundation loads. The structural fill should comprise Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D698 (standard Proctor) maximum dry density. A standard engineered fill construction is attached in Figure 1.

Footings should not be constructed on rockfill. However, rockfill may be placed adjacent to the Granular 'A' core. The recommended bearing resistance for 2.5 m wide footings constructed on structural fill is as follows:

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

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

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



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

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

6.3 Lateral Earth Pressures

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

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

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/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 ϕ = angle of internal friction of retained soil (35° for Granular A or Granular B Type II or Type III)

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



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

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

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

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

A subdrain system (SP 405F03) or weep holes (OPSD-3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the wall. 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 recommended previously for spread footings constructed on the native soil or 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 embankment fill heights are about 8 m at the northbound and southbound structures, respectively.

The scope of work for this preliminary study did not require that boreholes be carried out for the approach embankments to the Woodlawn Road Overpass. In view of the very stiff cohesive soils underlain by typically stiff to very stiff clayey silt till followed by dense to very dense sandy silt till at the abutments encountered in the two drilled boreholes, the approach embankments are likely to be founded on competent native soils. Consequently, slope stability issues are not anticipated at this site.

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

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

Based on limited laboratory test data, including the plasticity characteristics of the native soils, it is estimated that some 60 mm of consolidation settlement of the clayey subgrade soils will occur at the approach embankments. This estimated settlement is likely to take up to 6 months to occur to about 80% completion after which the remaining 10 to 20 mm of settlement should be tolerable for the approaches.



For pile foundations, it is recommended that the areas of the approach embankment fill over the abutments be placed at least 3 to 4 months prior to driving the abutments piles. This partial pre-loading would eliminate or reduce the negative skin friction on the abutment piles.

For spread footing foundations, the footing area and including a zone of influence equal to the height of the fill should be pre-loaded for at least 3 to 4 months before constructing the footing to reduce the post-construction settlements of the foundations.

Further subsurface investigation and laboratory tests should be carried out during detail design for this purpose.

The structure should be backfilled using granular materials 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.

Since the native soils in the area are known to be susceptible to surface erosion and surficial slope instability, local protection measures of earth fill embankments should be implemented during detail design. Measures such as rip-rap, vegetation cover, slope flattening and intercepting ditches may be considered.

6.5 Construction Considerations

6.5.1 Excavation

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



6.5.2 Groundwater Control

Groundwater was observed during the course of the field work at a depth of 13.7 m (elevation 169.4) in borehole 2 drilled near the SBL overpass south abutment. It is anticipated that groundwater will not be an issue for this site.

It is considered that seepage from soil and surface water run-off that enters the excavation should be readily handled by conventional sump pumping techniques. Groundwater conditions should be further assessed during detail design.

7. ADDITIONAL STUDIES

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

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

1. Carry out the complete scope of detailed field investigations at the structure site. Incorporate the data from the previously drilled boreholes included in this report for the Detail Design.
2. Determine/evaluate the slope of the bedrock founding surface to evaluate the need for steel pile rock points and the reduction of axial bearing resistance related to the extent of cobbles and boulders in the very dense sandy silt till mantling the dolostone bedrock.



8. CLOSURE

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

Yours very truly,

Peto MacCallum Ltd.

**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



8. CLOSURE

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

Yours very truly,

Peto MacCallum Ltd.



C. M. P. Nascimento, P.Eng.
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	25.0 – 26.5	85	0	25.0 – 31.1	DOLOSTONE: Buff to grey, fine grained, low to medium strength; unweathered; with occ. layers of grey to black shale (up to 700 mm thick), with black shale partings and occ. seams of gypsum and calcite, very close to close spaced flat bedding layers, smooth planar, tight; very poor quality. (Salina Formation)
	16	26.5 – 28.1	98	0		
	17	28.1 – 29.6	100	0		
	18	29.6 – 31.1	83	0		

RQD: Rock Quality Designation

Originated: MR
 Compiled: JFW
 Checked: MRA



TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

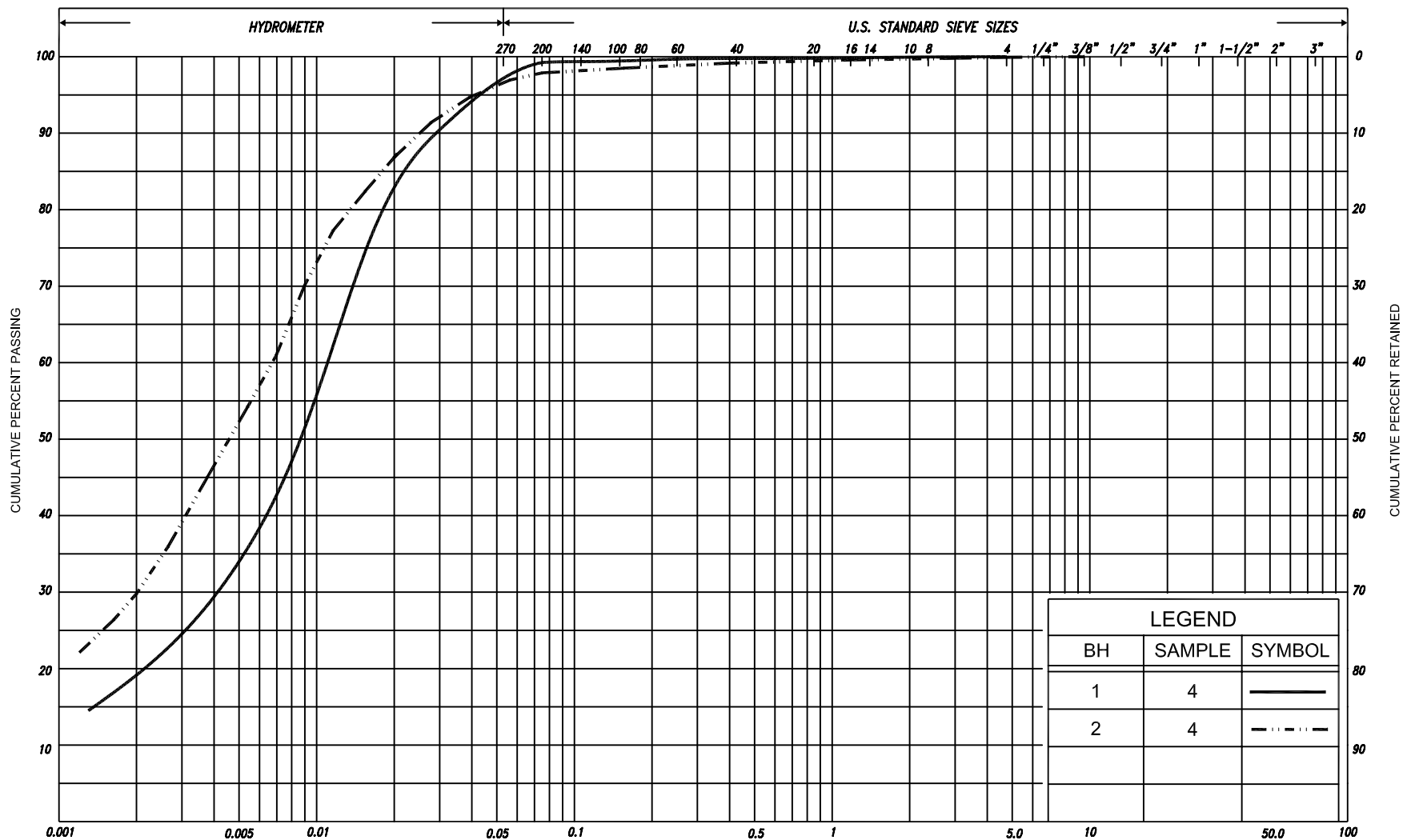
DOCUMENT	TITLE
OPSS 501	Construction Specification for Compacting
OPSS 571	Construction Specification for Sodding
OPSS 902	Excavation and Backfilling of Structures
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 599S22	Requirements for The Design,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



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

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

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



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												

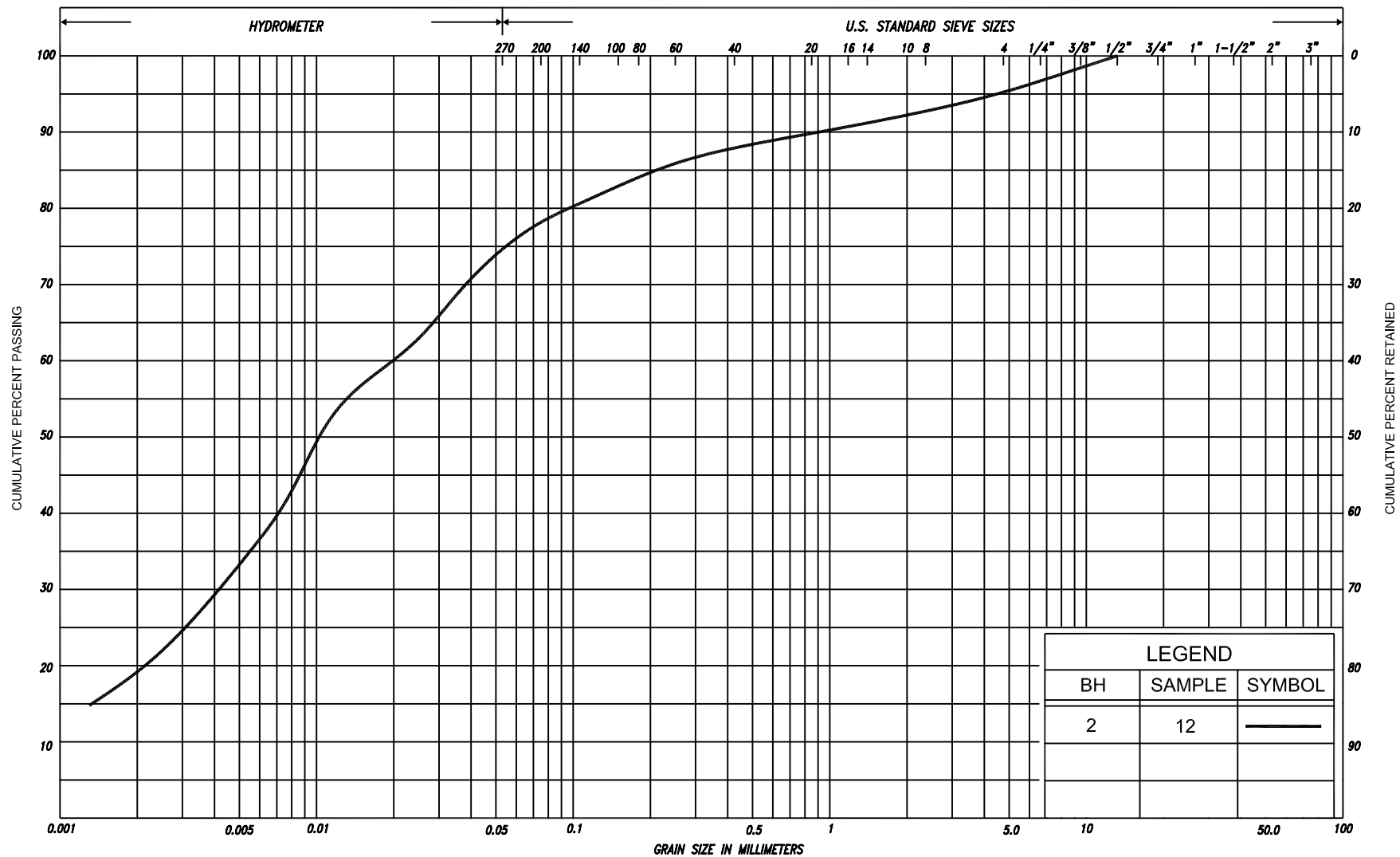
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

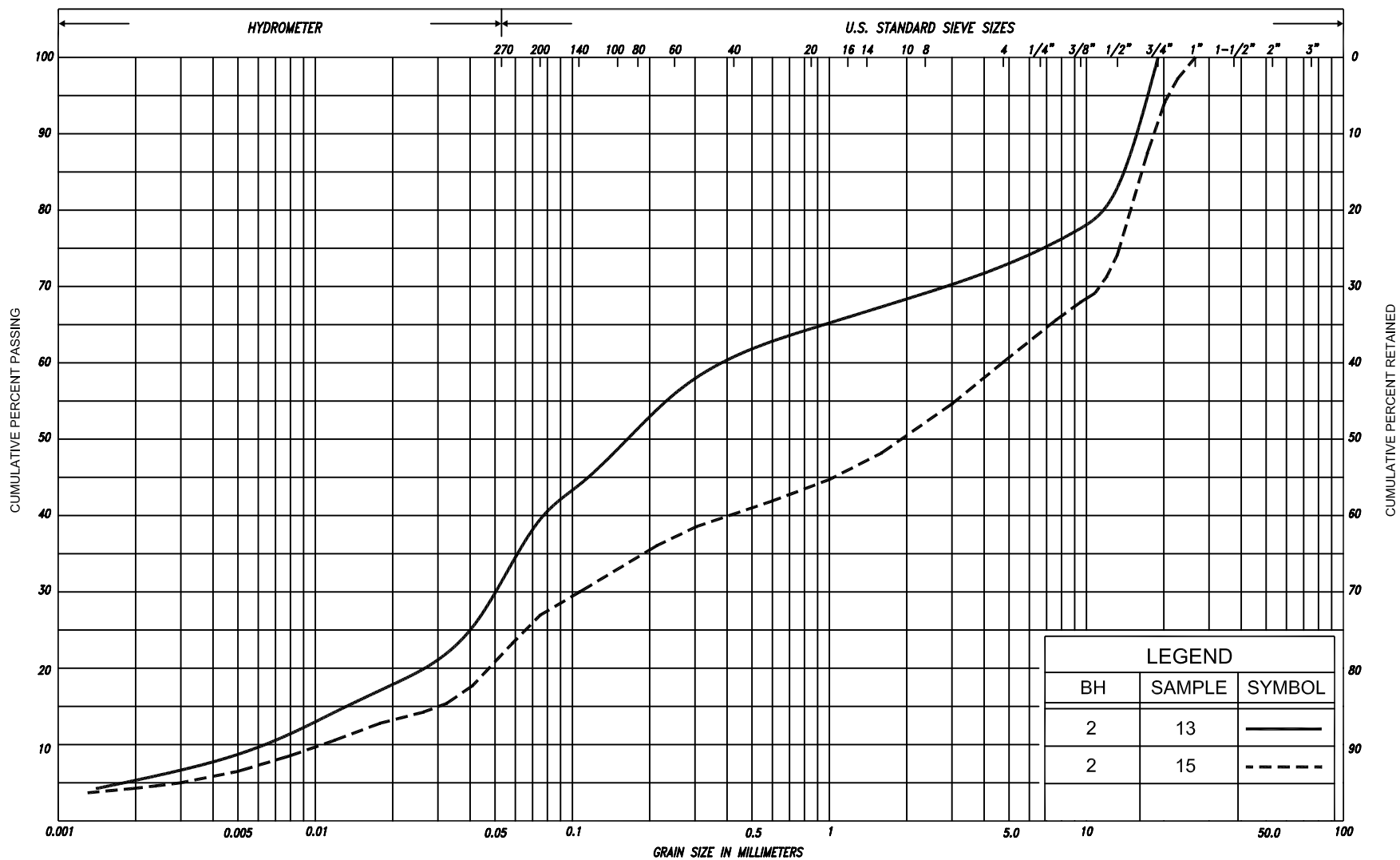
FIG No. GS-WL-1

HWY: 406

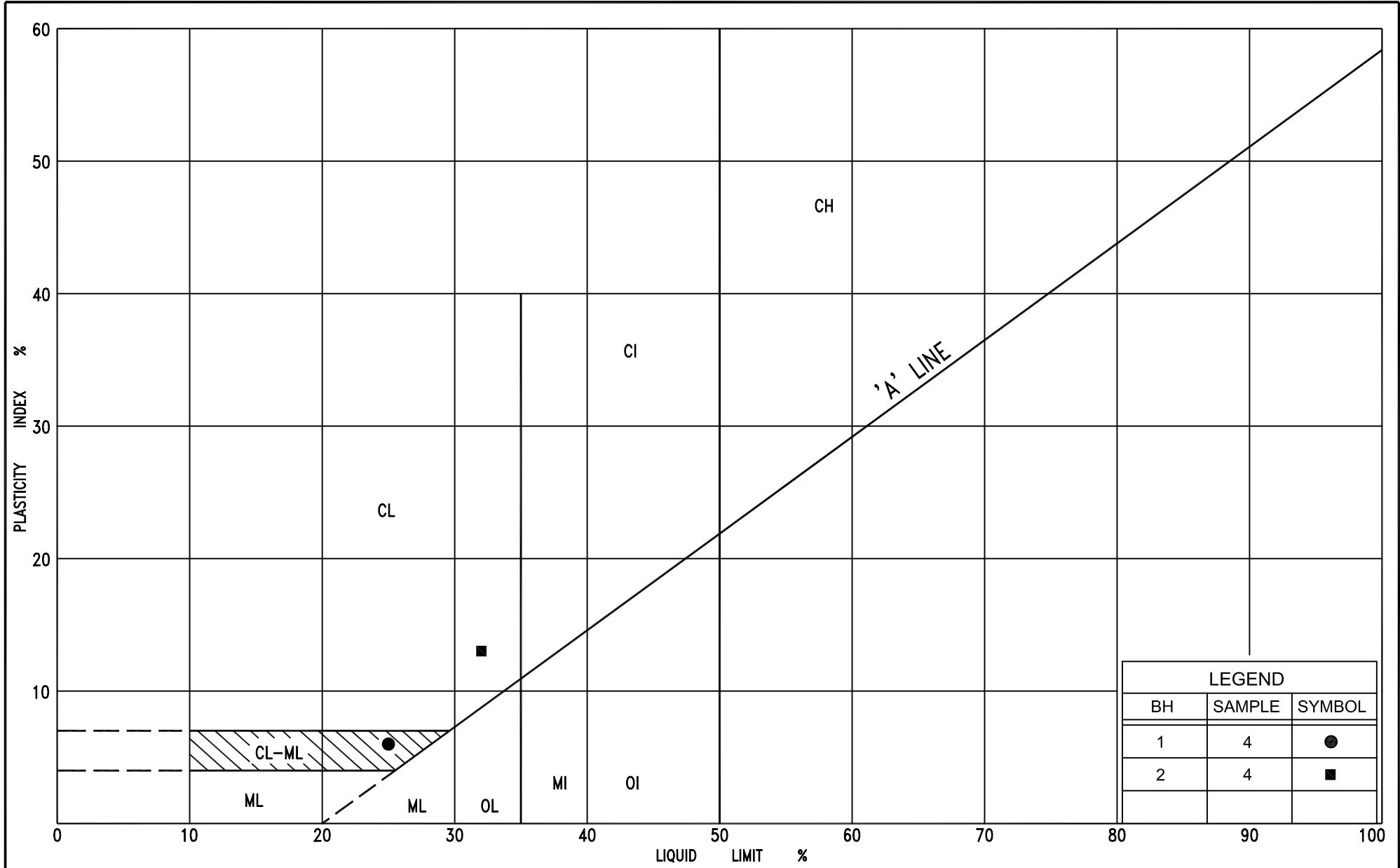
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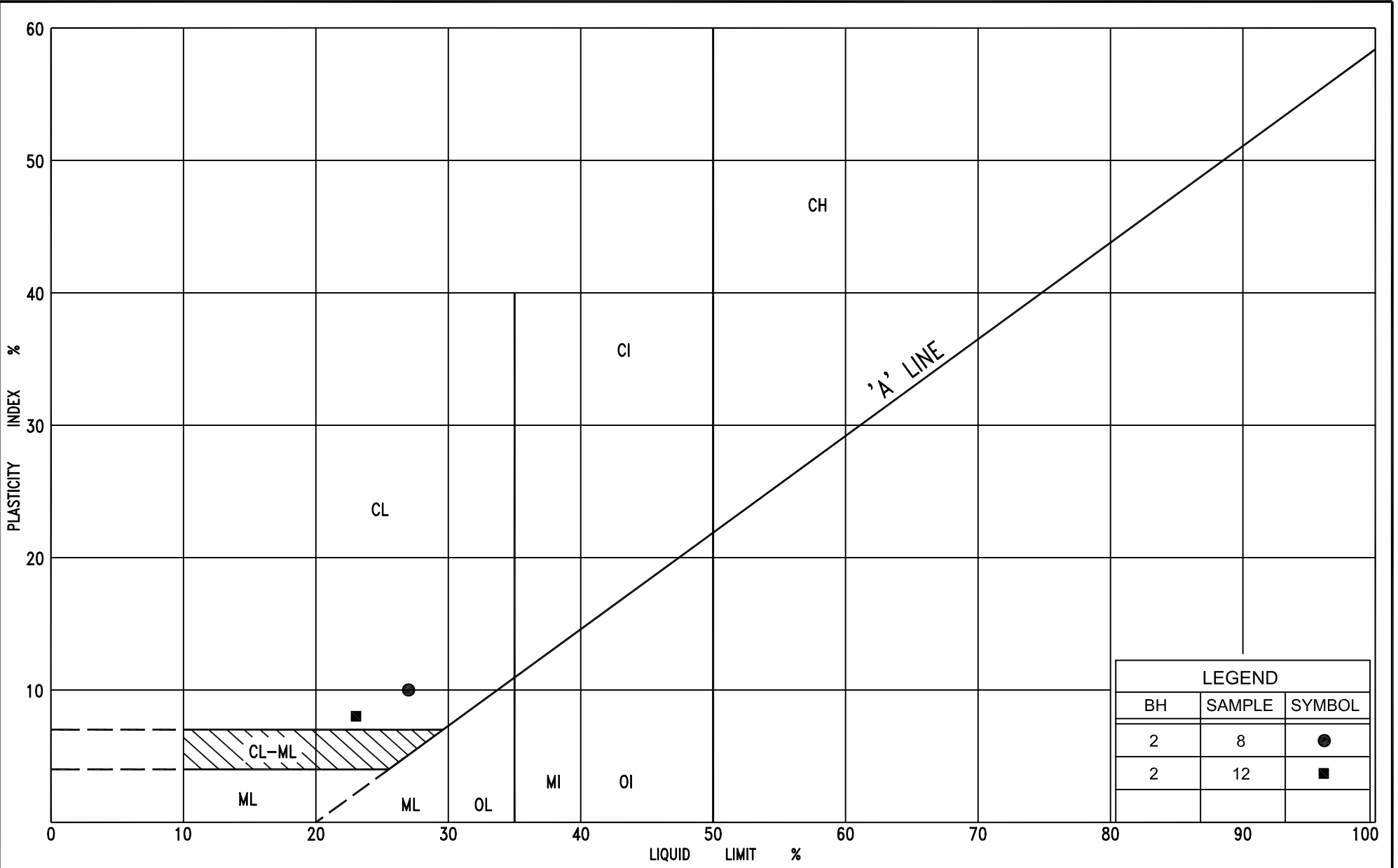


SILT & CLAY			FINE			MEDIUM			COARSE			GRAVEL			COBBLES	UNIFIED
						SAND										M.I.T.
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE	GRAVEL			COBBLES		U.S. BUREAU
		SILT				V. FINE	FINE	MED.	COARSE	GRAVEL						
CLAY		SILT				SAND										

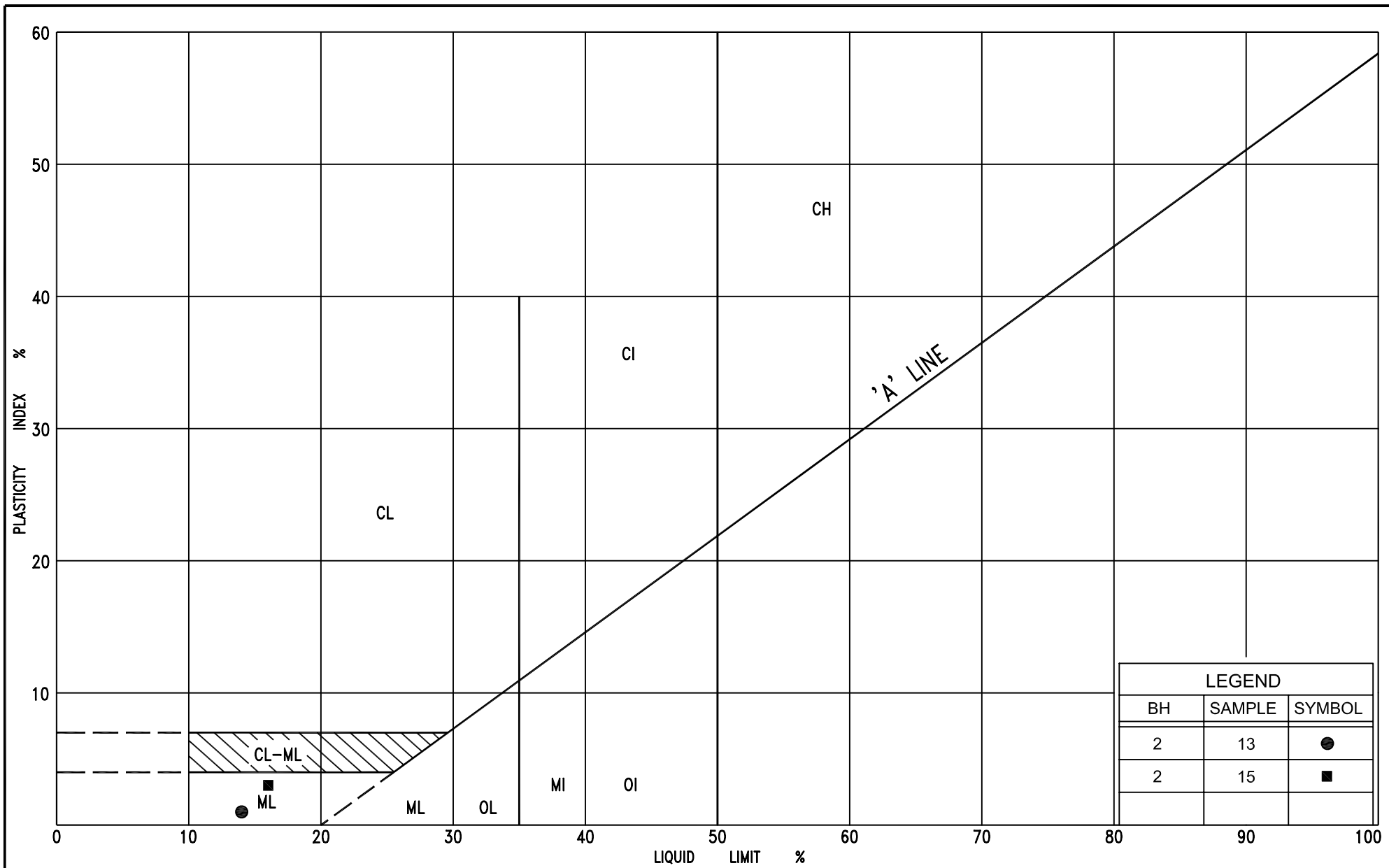


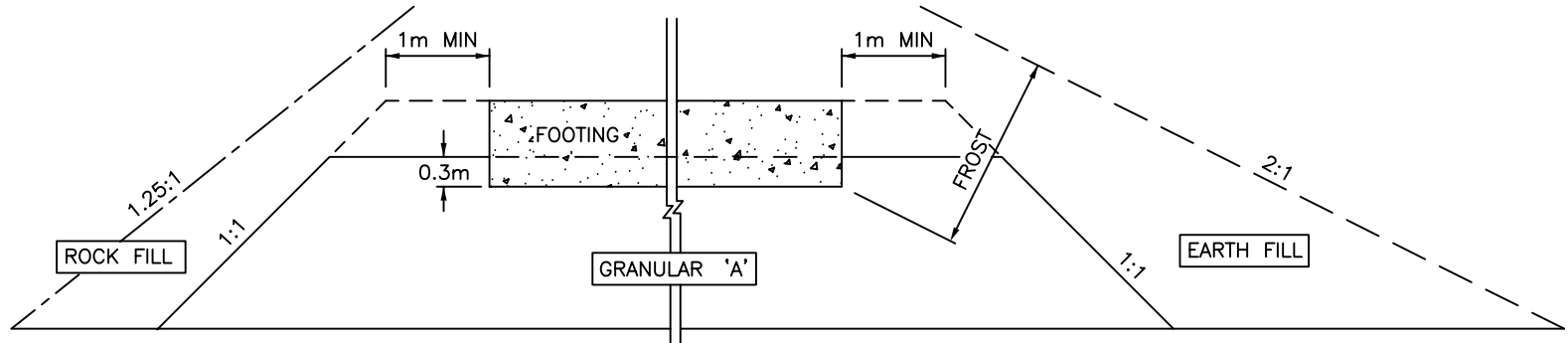
SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED
CLAY	SAND			FINE		MEDIUM		COARSE	GRAVEL		COBBLES	M.I.T.
	SILT			Y. FINE		FINE		MED.	COARSE	GRAVEL		U.S. BUREAU





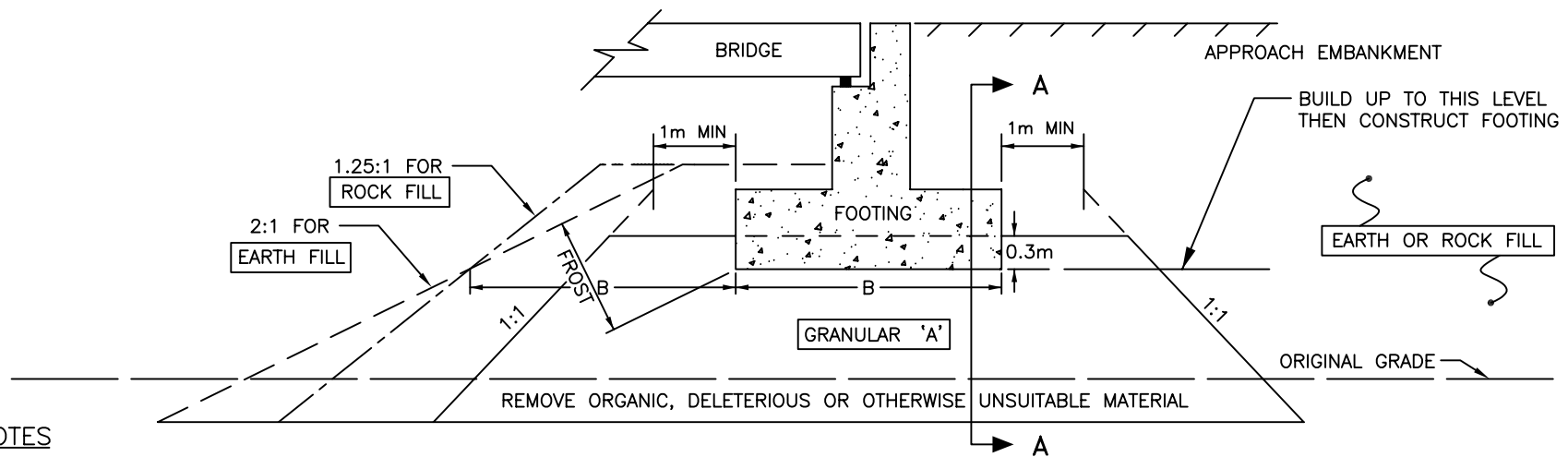
LEGEND		
BH	SAMPLE	SYMBOL
2	8	●
2	12	■





CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

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

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

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 1

1 of 3

METRIC

G.W.P. 280-99-00 LOCATION

Co-ords: 4 764 229 N; 327 334 E
Highway 406 at Woodlawn Road

ORIGINATED BY M.R.

DIST CR HWY 406 BOREHOLE TYPE

C.F.S.S.A. + NW Wash Boring + NXL Rock Coring

COMPILED BY G.D.

DATUM Geodetic DATE

November 07, 2001

CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL					
WATER CONTENT (%)															
181.9	Ground Surface						40 80 120 160 200								
0.0	Shoulder structure, 25mm of tar and chip over 250mm of granular 'A' crushed limestone						20 40 60 80 100								
181.6	(FILL)														
0.3	Clayey silt, trace sand		1	SS	26										
	Very stiff Brown Moist														
	Hard		2	SS	32										
	oxidized stains and thin partings of silt		3	SS	21										
	Very stiff														
	lenses of silty clay		4	SS	29										
177.9															
4.0	Clayey silt trace sand, trace gravel														
	Stiff Reddish Moist brown		5	SS	11										
	(TILL)														
			6	SS	10										
	lenses of silt		7	SS	6										
	Firm to Brown			FV											
	stiff														
	Stiff to Reddish		8	SS	8										
	hard brown			FV											
			9	SS	8										
				FV											
			10	SS	15										
														</	

RECORD OF BOREHOLE No 1 2 of 3 METRIC																
G.W.P. 280-99-00			LOCATION Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road			ORIGINATED BY M.R.										
DIST CR HWY 406			BOREHOLE TYPE C.F.S.S.A. + NW Wash Boring + NXL Rock Coring			COMPILED BY G.D.										
DATUM Geodetic			DATE November 07, 2001			CHECKED BY M.R.A.										
SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			40	80	120	160					
166.9 15.0			11	SS	43											
164.8 17.1	Sandy silt, trace clay some gravel, cobbles Dense to Reddish Moist very dense brown to wet 10cm layer of sand (TILL)		12	SS	42											
			13	SS	52											
			14	SS	50/5cm											
156.9 25.0	Dolostone Bedrock 23 cm deep void at 25.4m Buff to grey Unweathered Shale layers Low to medium strength Very poor quality		15	RC NXL	REC 85%											
			16	RC NXL	REC 98%											
			17	RC NXL	REC 100%											
151.9																

RECORD OF BOREHOLE No 1										3 of 3		METRIC					
G.W.P. 280-99-00			LOCATION Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road			ORIGINATED BY M.R.											
DIST CR HWY 406			BOREHOLE TYPE C.F.S.S.A. + NW Wash Boring + NXL Rock Coring			COMPILED BY G.D.											
DATUM Geodetic			DATE November 07, 2001			CHECKED BY M.R.A.											
SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
151.9 30.0			18	RC NXL	REC 83%												
150.8 31.1	End of borehole						151										
	* Borehole charged with drilling water C.F.S.S.A denotes: Continuous Flight Solid Stem Augers																

RECORD OF BOREHOLE No 2

1 of 3

METRIC

G.W.P. 280-99-00 LOCATION Co-ords: 4 764 164 N; 327 311 E
DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers
DATUM Geodetic DATE November 15, 2001

ORIGINATED BY M.R.

COMPILED BY G.D.

CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							WATER CONTENT (%)										
183.1	Ground Surface						40	80	120	160	200	20	40	60			
0.0	Clayey silt, trace sand fissured																
	Very stiff Brown Moist		1	SS	23												
	thin layers of silt		2	SS	29												
			3	SS	21												
			4	SS	20												
179.4	Silt, trace gravel layers of grey silty clay		5	SS	20												
3.7	Compact Reddish Moist brown		6	SS	27												
177.6	Clayey silt, trace sand specks of shale																
5.5	Stiff to Reddish Moist very stiff brown		7	SS	14												
	(TILL)																
	trace gravel		8	SS	8												
	faintly layered																
	thin silt layers		9	SS	8												
	layers of silt, trace gravel up to 20mm diameter		10	SS	9												
			11	SS	13												
	some sand																
168.1	Cont'd																

RECORD OF BOREHOLE No 2										2 of 3	METRIC				
G.W.P. 280-99-00			LOCATION			Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road				ORIGINATED BY M.R.					
DIST CR HWY 406			BOREHOLE TYPE			Continuous Flight Solid Stem Augers				COMPILED BY G.D.					
DATUM Geodetic			DATE			November 15, 2001				CHECKED BY M.R.A.					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
168.1 15.0			12	SS	19										5 17 59 19
166.1 17.0	Sandy silt with gravel, trace clay Very dense Reddish Moist brown (TILL)		13	SS	61										27 33 35 5
163.1 20.0	Clayey silt some sand, trace gravel Very stiff Reddish Moist to hard brown (TILL)		14	SS	30										
160.1 23.0	Sandy gravelly silt trace clay, cobbles Very dense Reddish Moist brown (TILL)		15	SS	67/10cm										40 32 24 4
157.2 25.9	Bedrock - Possible Dolostone shale layers Augered with difficulty Shattered / Very weak possible void was encountered at 26.8m														
153.1															

RECORD OF BOREHOLE No 2 3 of 3 METRIC																	
G.W.P. 280-99-00		LOCATION		Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road				ORIGINATED BY M.R.									
DIST CR HWY 406		BOREHOLE TYPE		Continuous Flight Solid Stem Augers				COMPILED BY G.D.									
DATUM Geodetic		DATE		November 15, 2001				CHECKED BY M.R.A.									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
153.1								40	80	120	160	200					
30.0						153											
152.6																	
30.5	End of borehole																
<div style="display: flex; justify-content: space-between;"> <div> <p>* 2001 11 15</p> <p>▼ Water level measured after drilling</p> <p>■ Penetrometer Test</p> </div> <div> <p>20 15—○—5 10</p> <p>(%) STRAIN AT FAILURE</p> </div> </div>																	

RECORD OF BOREHOLE No 1

1 of 3

METRIC

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

ORIGINATED BY M.R.
COMPILED BY G.D.
CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
181.9	Ground Surface													
0.0	Shoulder structure, 25mm of tar and chip over 250mm of granular 'A' crushed limestone													
0.3	(FILL)													
	Clayey silt, trace sand		1	SS	26		181							
	Very stiff Brown Moist													
	Hard		2	SS	32		180							
	oxidized stains and thin partings of silt		3	SS	21		179							
	Very stiff		4	SS	29		178							
	lenses of silty clay													
177.9	Clayey silt trace sand, trace gravel						177							
4.0	Stiff Reddish Moist brown		5	SS	11		176							
	(TILL)		6	SS	10		175							
	lenses of silt		7	SS	6		174							
	Firm to Brown stiff			FV			173							
	Stiff to Reddish hard brown		8	SS	8		172							
				FV			171							
			9	SS	8		170							
				FV			169							
			10	SS	15		168							
166.9							167							

RECORD OF BOREHOLE No 1

2 of 3

METRIC

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

ORIGINATED BY M.R.
COMPILED BY G.D.
CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
166.9 15.0			11	SS	43												
							166										
							165										
164.8 17.1	Sandy silt, trace clay some gravel, cobbles Dense to Reddish Moist very dense brown to wet						164										
			12	SS	42		163										
	10cm layer of sand						162										
	(TILL)						161										
			13	SS	52		160										
							159										
							158										
			14	SS	50/5cm		157										
156.9 25.0	Dolostone Bedrock						156										
	23 cm deep void at 25.4m		15	RC NQ	REC 85%		155										RQD 0%
	Buff to grey Unweathered Shale layers Low to medium strength Very poor quality		16	RC NQ	REC 98%		154										RQD 0%
			17	RC NQ	REC 100%		153										RQD 0%
151.9							152										

RECORD OF BOREHOLE No 1										3 of 3		METRIC					
G.W.P. 280-99-00			LOCATION			Co-ords: 4 764 229 N; 327 334 E Highway 406 at Woodlawn Road			ORIGINATED BY M.R.								
DIST CR HWY 406			BOREHOLE TYPE			C.F.S.S.A.+ NW Wash Boring + Rotary Diamond Drilling			COMPILED BY G.D.								
DATUM Geodetic			DATE			November 07, 2001			CHECKED BY M.R.A.								
SOIL PROFILE			SAMPLES			* GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
151.9			18	RC NQ	REC 83%												
30.0							151										RQD 0%
150.8																	
31.1	End of borehole																
	Sample 14: Sampler bouncing																
	* Borehole charged with drilling water																
	C.F.S.S.A denotes: Continuous Flight Solid Stem Augers																

RECORD OF BOREHOLE No 2

1 of 3



METRIC

G.W.P. 280-99-00 LOCATION Co-ords: 4 764 164 N; 327 311 E
DIST CR HWY 406 BOREHOLE TYPE Continuous Flight Solid Stem Augers
DATUM Geodetic DATE November 15, 2001

ORIGINATED BY M.R.
COMPILED BY G.D.
CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
183.1	Ground Surface						183										
0.0	Clayey silt, trace sand fissured																
	Very stiff Brown Moist		1	SS	23												
	thin layers of silt		2	SS	29												
			3	SS	21												
			4	SS	20												
179.4	Silt, trace gravel layers of grey silty clay		5	SS	20												
3.7	Compact Reddish Moist brown		6	SS	27												
177.6	Clayey silt, trace sand specks of shale		7	SS	14												
5.5	Stiff to Reddish Moist very stiff brown																
	(TILL)																
	trace gravel		8	SS	8												
	faintly layered		9	SS	8												
	thin silt layers		10	SS	9												
	layers of silt, trace gravel up to 20mm diameter		11	SS	13												
	some sand																
168.1																	

RECORD OF BOREHOLE No 2										2 of 3		METRIC				
G.W.P. 280-99-00			LOCATION Highway 406 at Woodlawn Road			Co-ords: 4 764 164 N; 327 311 E			ORIGINATED BY M.R.							
DIST CR HWY 406			BOREHOLE TYPE Continuous Flight Solid Stem Augers						COMPILED BY G.D.							
DATUM Geodetic			DATE November 15, 2001						CHECKED BY M.R.A.							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa									
168.1 15.0			12	SS	19		20	40	60	80	100					
166.1 17.0	Sandy silt with gravel, trace clay Very dense Reddish Moist brown (TILL)		13	SS	61											
163.1 20.0	Clayey silt some sand, trace gravel Very stiff Reddish Moist brown (TILL)		14	SS	30											
160.1 23.0	Gravelly sandy silt trace clay, cobbles Very dense Reddish Moist brown (TILL)		15	SS	67/10cm											
157.2 25.9	Bedrock - Possible Dolostone shale layers Augered with difficulty Shattered / Very weak possible void was encountered at 26.8m															
153.1																

RECORD OF BOREHOLE No 2 3 of 3 METRIC																	
G.W.P. 280-99-00		LOCATION		Co-ords: 4 764 164 N; 327 311 E Highway 406 at Woodlawn Road				ORIGINATED BY M.R.									
DIST CR HWY 406		BOREHOLE TYPE		Continuous Flight Solid Stem Augers				COMPILED BY G.D.									
DATUM Geodetic		DATE		November 15, 2001				CHECKED BY M.R.A.									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION kPa RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
153.1																	
30.0																	
152.6																	
30.5	End of borehole Sample 15: Sampler bouncing * 2001 11 15  Water level measured after drilling  Penetrometer Test																

METRIC

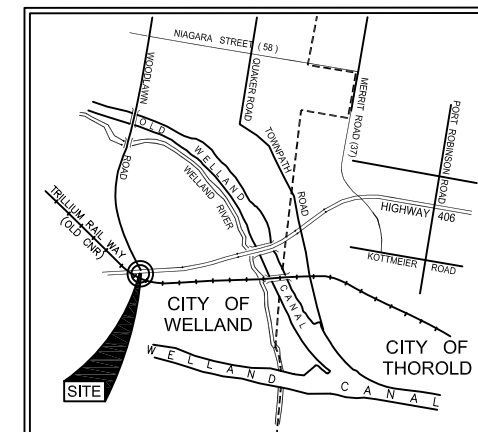
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

CONT No
GWP No 280-99-00

WOODLAWN ROAD UNDERPASS
HIGHWAY 406
BOREHOLE LOCATIONS



SHEET



KEY PLAN
0.5 0 0.5 1km

LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation Nov 2001
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

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

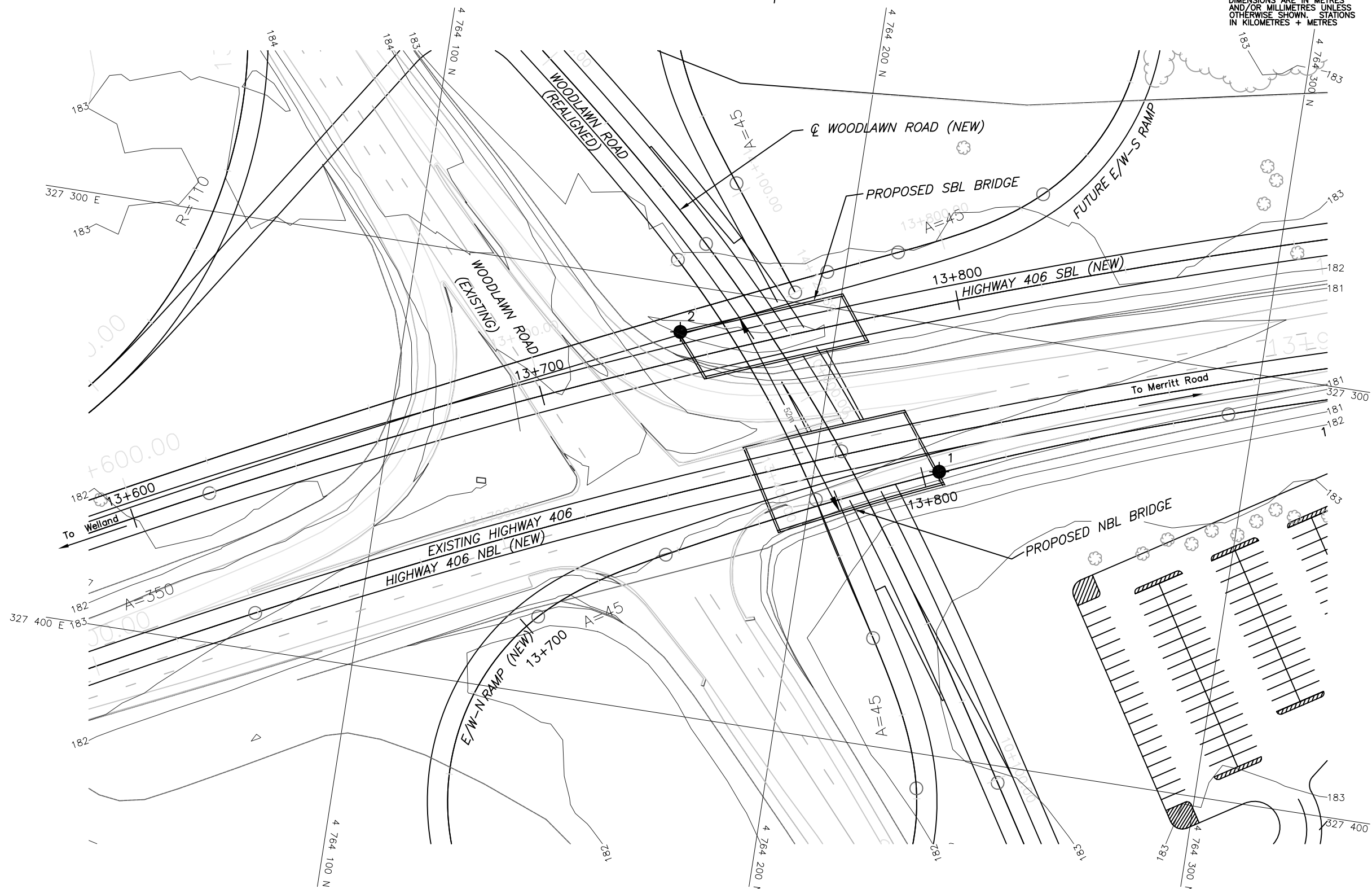
NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 30M03-236

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



PLAN

SCALE



NOTE:

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



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