March 8, 2017

DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT

High Mast Light Pole Foundations Highway 58 / Highway 406 and St. David's Road Interchange G.W.P. 2364-09-00

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DRAFT REPORT

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THOROLD, ONTARIO.
G.W.P. 2364-09-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by HDR Inc. (HDR) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for High Mast Light (HML) poles in support of the upgrades to the Interchange at Highway 58 / 406 – St David's Road in Thorold, Ontario. This report addresses the results of the foundation investigation carried out for the proposed three (3) HML poles at the interchange.

The Terms of Reference and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) dated December 17, 2010 and subsequent clarifications. The Scope of Work presented in Golder's revised scope change letter (Scope Change No. 1, Revision 2) dated November 9, 2016, has been concurred with and is addressed herein; it is comprised of a field investigation involving the advancement of three boreholes at the proposed HML pole locations.

2.0 SITE DESCRIPTION

The Highway 58 / 406 – St David's Road Interchange is comprised of nine overpass/underpass structures and upwards of 15 ramps. The ground surface topography in the Interchange area is generally flat with Highway 406 and Highway 58 generally cut into the landscape exposing the local bedrock in several locations; St. David's Road and the approaches to the overpass/underpass structures are raised fill embankments. The ground surface is generally vegetated with grasses and local thickets of brush and young trees.

The location of the site is shown on the Key Plan on Drawing 1 and the locations of the proposed HML poles, labelled P1, P2 and P3, are shown on Drawing 1. The HML poles are generally located on the flat plains adjacent to the highway cuts, with the exception of Pole P2 which is located near the transition from a cut to a filled approach embankment.

3.0 INVESTIGATION PROCEDURES

The field work for is investigation was carried out during the period between December 19 to 21, 2016, at which time a total of three boreholes (Boreholes P1 to P3) were advanced at the locations shown on Drawing 1.

The boreholes were advanced using a track-mounted drill rig (Geoprobe 7822 DT) supplied and operated by Determination Drilling of Hamilton, Ontario. The boreholes were augered to refusal using 150 mm diameter solid stem augers and were further advanced into the bedrock using HQ coring equipment. Soil samples were obtained in the boreholes at 0.75 m depth intervals using a 50 mm outer diameter split spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586).

The groundwater conditions in the open boreholes were observed during and immediately following the drilling operations. Subsequently, open borehole P2 was backfilled to ground surface with bentonite in accordance with Ontario Regulation 903 (as amended). A standpipe piezometer water installed in each of Boreholes P1 and P3 to allow future observation of the groundwater level at these locations. The standpipe piezometer consists of 50 mm diameter polyvinyl chloride (PVC) pipe with a 3 m long slotted screen section within the bedrock unit. The borehole and annulus surrounding the piezometer pipe above the screen (and sand pack) was backfilled to ground surface with bentonite. Piezometer installation details and water level readings are described on the Record of Borehole sheets presented in Appendix A. The water level in the piezometers was measured on February 3, 2017.

The field work was observed on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, arranged for the clearance of underground utilities, directed the drilling, sampling and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Cambridge for further examination and laboratory testing. Index and classification tests (water contents, Atterberg limits and grain size distributions) were carried out on selected soil samples. The bedrock core was placed in core boxes, photographed in the field and transported to our Mississauga laboratory for further visual examination and laboratory strength testing consisting of unconfined compression tests on selected specimens of the bedrock core. The results of the laboratory testing on the soil samples and bedrock core samples are shown on the Record of Borehole and Record of Drillhole sheets and are presented in the laboratory test sheets in Appendix B. All geotechnical laboratory testing was completed to ASTM and MTO LS standards, as applicable. Photographs of the bedrock core are also presented in Appendix B.

The borehole locations and ground surface elevations were measured on-site using a GPS with accuracy better than 0.1 m on the vertical and horizontal planes. The borehole locations are shown on Drawings 1 and 2 and are summarized below using MTM NAD83 (Zone 10) northing and easting coordinates. The ground surface elevations at the borehole locations are provided with reference to the Canadian Geodetic Vertical Datum of 1928.

The borehole locations, ground surface elevations and drilled depths are as follows:

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
P1	4,775,821.0	326,664.9	175.4	9.9
P2	4,775,568.9	326,618.1	176.5	9.5
P3	4,775,560.1	326,458.9	176.0	9.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The Highway 58 / 406 – St David's Road Interchange site is located within the physiographic region known as the Haldimand Clay Plain, according to The Physiography of Southern Ontario (Chapman and Putnam, 1984)[1].

The Haldimand Clay Plain is described as dissected into a series of sub-parallel belts of successive elevations separated by recessional moraines with the depressed areas between moraines in-filled with fine-grained lacustrine sediments. The upper belt located south of (above) the Niagara Escarpment is delineated by the Vinemount moraine that extends into New York state. The whole of this physiographic region was submerged by glacial Lake Warren.

[1] Chapman, L.J. and Putnam, D.F. 1984. The Physiography of Southern Ontario, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

The overburden soils are underlain by grey dolomite of the Lockport Formation. The overburden soil are underlain by grey Gasport Group dolomite of the Lockport Formation of the Middle Silurian time period. This dolomite formation is also comprised of vug-rich grainstones and argillaceous shales containing crinoidal fossils" (Menzies and Taylor, 1998)[2].

4.2 Subsurface Conditions

As part of the foundations investigation, three boreholes (Boreholes P1 to P3) were advanced, one at each of the proposed HML pole locations. The borehole locations, ground surface elevations and soil stratigraphy are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of this investigation, and the results of the laboratory testing, are provided on the borehole and drillhole records contained in Appendix A. The STP 'N' values in situ test results presented on the borehole records were obtained by driving the split spoon sampler using an automatic trip hammer and the results are uncorrected. The laboratory testing results are also presented on Figures B1 and B2 in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic cross-sections on Drawing 1 are interpreted from observations of drilling progress and from non-continuous sampling, and therefore represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of a surficial layer of topsoil underlain by a deposit of stiff to very stiff silty clay underlain by dolostone bedrock. A more detailed description of the subsurface conditions encountered in the current boreholes is provided in the following sections.

4.2.1 Topsoil

An approximately 0.15 m thick layer of topsoil was encountered at the ground surface of each borehole.

4.2.2 Clayey Silt Fill

A 0.9 m thick layer of fill comprised of clayey silt some sand, some gravel, was encountered underlying the topsoil in Borehole P1. Based on the middle drive, 150 mm penetration of the SPT, the 'N'-value obtained suggests that the fill is stiff in consistency. The natural water content of a sample of the fill is about 11 percent.

4.2.3 Silty Clay

A silty clay deposit was encountered underlying cohesive fill in Borehole P1 and underlying the topsoil in Boreholes P2 and P3 at depths between 0.2 m and 1.1 m below the ground surface, corresponding to between Elevations 176.3 m and 174.3 m and the thickness of the deposit ranges from 1.3 m to 3.4 m. The SPT 'N'-values measured in the silty clay deposit range between 11 and 25 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

The water content of seven sample of the silty clay deposit ranged between about 17 percent and 24 percent. Atterberg limit testing carried out on three samples of the cohesive deposit measured liquid limits ranging from about 39 percent and 50 percent, plastic limits ranging from about 18 percent and 21 percent, and plasticity indices

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[2] Menzies, J. Taylor, E.M. 1998. Urban Geology of St Catharines – Niagara Falls, Region Niagara. In Urban Geology of Canadian Cities. Geological Association of Canada, Special Paper 42. Ed: Karrow P.F., White, O.L.

ranging from 21 percent to 29 percent. These results, which are plotted on a Plasticity Chart on Figure B1 in Appendix B, indicate that the cohesive deposit is classified as a silty clay of intermediate plasticity. Grain size distribution tests carried out on three samples of the cohesive deposit are shown on Figure B2 in Appendix B.

4.2.4 Bedrock

The silty clay deposit is underlain by bedrock in all boreholes which was encountered at depths between about 2.4 m and 3.6 m below ground surface, corresponding to between Elevations 173.0 m and 172.9 m. The bedrock was cored for depths ranging from about 5.9 m to 7.5 m, and it is described as being comprised of slightly weathered, thin to medium bedded, fine-grained and moderately porous, very strong dolomite of the Lockport Formation. Silty clay seams were encountered within the bedrock in boreholes P2 and P3 at depths of about 5.2 m (Elevation 171.3 m) and 3.7 m (Elevation 172.3 m) below the ground surface. The Rock Quality Designation (RQD) of selected core samples ranges between about 60 percent and 100 percent, typically between about 80 percent and 100 percent, generally indicating a rock mass of good to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (2006)[3]. The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples of the bedrock range from 85 percent to 100 percent and 60 percent to 100 percent, respectively.

Unconfined compressive strength (UCS) tests were carried out on three selected samples of the bedrock core. The test specimens and the test results are summarized below and the test reports are included in Appendix B:

Borehole	Elevation	Depth	Bulk Unit Weight	UCS
P1	170.8 m to 170.6 m	4.6 m to 4.8 m	25.8 kN/m^3	90 MPa
P2	172.5 m to 172.2 m	4.1 m to 4.3 m	26.6 kN/m^3	70 MPa
P3	168.5 m to 168.3 m	7.5 m to 7.7 m	25.3 kN/m^3	71 MPa

Based on the unconfined compressive strengths presented above, in accordance with Table 3.5 in CFEM (2006)[4], the dolomite bedrock is classified as strong (R4, 50 MPa < UCS < 100 MPa).

4.2.5 Groundwater

The open boreholes were noted to be dry during drilling and prior to bedrock coring operations. Standpipe piezometers were installed in boreholes P1 and P3 with screens sealed within the bedrock to observe the local water level. Other details of each installation are shown on the Record of Borehole sheets in Appendix A. The groundwater levels observed in the piezometers are summarized below.

Borehole	Ground Surface Elevation	Screen Interval/Elevation	Observation Date	Water Level Elevation (depth below ground surface)
P1	175.4 m	6.9 m to 9.9 m 168.5 m to 165.5 m	December 19, 2017 February 3, 2017	Open Borehole - Dry 168.3 m (7.1 m)
P3	176.0 m	6.2 m to 9.3 m 169.8 m to 166.7 m	December 19, 2017 February 3, 2017	Open Borehole - Dry 169.7 m (6.3 m)

[3] Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

The groundwater level is subject to seasonal fluctuations and variations due to precipitation events and snow melt in the spring.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Andrew Van Dyk, P.Eng. Mr. Jorge M. A. Costa, P.Eng., a Designated MTO Foundations Contact and Senior Consultant with Golder, conducted an independent review and quality control audit of this report.

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PART B

FOUNDATION DESIGN REPORT
HIGH MAST LIGHT POLES
HIGHWAY 406 / HIGHWAY 58 – ST. DAVID’S ROAD INTERCHANGE
THOROLD, ONTARIO.
G.W.P. 2364-09-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation recommendations for the design of the high mast light (HML) poles at the Highway 58/406 – St. David's Road Interchange. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at the site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to carry out detail design of the HML pole foundations. The foundation investigation report and the discussion and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purpose or any other parties including the construction of design-build contractor. Contractors must make their own interpretation based on the factual data in Par A of the report. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary CAN/CSA-S6-14 (CHBDC 2014), the proposed HML poles (geotechnical system) may be classified as having potential effects of "typical consequence" associated with exceeding limit states design. In addition, given the level of foundation investigation completed at these locations, the level of confidence for design is considered to be "typical degree of site and prediction model understanding", per Section 6.5 of the CHBDC (2014). Accordingly, the appropriate corresponding consequence factor, ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.1 and Table 6.2 of the CHBDC (2014) should be used for design:

- ψ = 1.0
- ϕ_{gu} = 0.5 for lateral resistance of caissons at Ultimate Limit States (ULS) based on static analysis.
- ϕ_{gs} = 0.8 for lateral deflection of caissons at Serviceability Limit States (SLS) based on static analysis.

6.3 Design of High Mast Light Pole Foundations

6.3.1 General

Three new HML poles are required for this project to be supported on a single caisson foundation at each HML pole location. Caisson foundations for HML poles should be designed in accordance with the requirements in MTO's *Guidelines for the Design of High Mast Pole Foundations* (MTO, 2004). The recommended value of the various geotechnical parameters required for the design of the caisson foundations are given in Table 1 following the text of this report. These parameter values have been derived based on our interpretations of the subsurface stratigraphy and groundwater conditions and take into consideration the overburden thickness between about 2.4 m and 3.6 m and the underlying strong and slightly weathered dolomite bedrock at the three borehole locations.

The depth of frost penetration can be interpolated from Ontario Provincial Standards Drawing (OPSD) 3090.101 (*Foundation, Frost Penetration Depths for Southern Ontario)* which for the Niagara area can be taken to be about 1.2 m.

6.3.2 Foundations Embedded in Overburden Soil

Design of the caisson foundation within the cohesive overburden soil as encountered at this site should consider both undrained and drained (effective stress) strength parameters and the more conservative design approach should be adopted. To be consistent with the nomenclature and equations used in MTO (2004), the undrained strength of the cohesive soil given in Table 1 is described using the unconfined compressive strength (q_u), which is equivalent to twice the undrained shear strength (s_u).

Design of the caisson foundation should neglect resistance contributions from the soil that is within the depth of frost penetration. As the thickness of the overburden between the frost penetration depth and the top of bedrock is between about 1.2 m and 2.4 m at the borehole locations, this overburden zone is unlikely to be sufficient to develop the required resistances. As such, the HML pole foundations are expected to be socketed, embedded and/or anchored into rock. Recommendations for these conditions are provide in Sections 6.3.3, 6.3.4 and 6.3.5.

Recognizing that there is limited thickness of overburden at this site as noted above to provide for lateral resistance of the HML pole caisson foundation, nonetheless, the factored passive lateral earth pressure, P_p (kPa), distributed along the depth of the caisson foundation below the depth of frost penetration may be calculated using the following equations:

$P_p = K_p \cdot \gamma \cdot d$ above the groundwater level

$P_p = K_p \cdot \gamma \cdot d_w + K_p \cdot \gamma' \cdot (d - d_w)$ below the groundwater level

where: K_p is the passive earth pressure coefficient;

γ is the bulk unit weight (kN/m3);

γ' is the effective bulk unit weight below the groundwater level (kN/m3);

d is the depth below the ground surface (m); and

d_w is the depth below the groundwater level (m).

The lateral earth pressure provided by the overburden may be assumed to act over an equivalent width equal to three times the caisson diameter. The appropriate geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, as described in Section 6.2, should be applied to calculated nominal lateral resistance in order to obtain the factored lateral geotechnical resistance.

6.3.3 Foundations Socketed in Rock

The bedrock over the full depth of the borehole is visually characterized as slightly weathered. For the purpose of defining the parameter *W* (depth of weathering, per MTO, 2004), the weathering at the rock surface is slight such that *W* can be nominally assumed to be about 0.3 m. The depth of frost penetration at this site is less than the existing overburden thickness and therefore frost penetration will not influence the socket depth, provided that the ground surface elevation remains unchanged after construction. As such, the socket depth should be not less than the sum of *W* and one half of the caisson diameter (*D*) as per Section 7.1 of MTO (2004).

6.3.4 Foundations Embedded in Rock

Caisson foundations embedded in rock should penetrate below the weathered rock *W* (as defined in Section 6.3.3) by at least 2.5 m as per Section 6.1 of MTO (2004). The dolomite bedrock at the site is generally strong, and coring or churn drilling will be necessary to advance the socket into the bedrock. It is recommended that Non-Standard Special Provisions (NSSP) be included with the Contract Documents to warn the contractor of the bedrock strength which is expected to affect the installation of the HML pole caisson foundations; an example NSSP is included in Appendix C.

As the bedrock surface is present at shallow depth below the ground surface at the HML pole locations, consideration could be given to the use of foundations anchored to the rock. Recommendations for the rock anchors are provided in Section 6.3.5.

6.3.5 Foundations Anchored in Rock

Recommendations for uplift anchor design and construction are available in PTI (2014). The design and installation of rock anchors should be consistent with OPSS 942 (Prestressed Soil and Rock Anchors). If anchoring is adopted, it is recommended that the concrete foundations (either caissons or spread footings) be embedded a minimum of 0.3 m into the bedrock. As per Section 6.2 of MTO (2004), a minimum concrete foundation length of 1.75 m is required to allow sufficient length for the anchorage assembly. As the compressive strength of the caisson concrete is lower that the compressive strength of the bedrock at the site, the vertical bearing resistance should be taken as the compressive strength of the concrete in the caisson.

The horizontal resistance of the dowels is dependent on the strength of the bedock, grout and steel. At this site, the rock mass is stronger than concrete and so the design of the dowels in the rock should be handled in the same way as the dowel embedment into the concrete, assuming that the unconfined compressive strength of the grout is similar to that of the concrete. For uplift of the rock dowels, the anchor bond length in slightly weathered dolostone can be designed assuming average ultimate bond strength along the rock/grout interface is equal to 1.6 MPa (factored). A resistance factor of 0.4 has been used to calculate the factored bond strength. The structural strength of the dowel and the compressive strength of the grout should not be exceeded.

Consistent with recommended practice (PTI, 2014), the bond length as calculated above should be greater than 3.0 m. A free-stressing length of at least 3.0 m is recommended between the underside of the foundation and the start of the bonded length. As such, the minimum total anchor length would be 6.0 m. Proposed anchor designs that vary from the PTI recommendations should be reviewed and approved by the structural designer prior to their use in the project.

The uplift resistance of the rock mass should be verified by confirming that the weight of the rock volume mobilized by the anchor will resist the applied anchor load. For this purpose, designers can assume that a single anchor will mobilize a cone-shaped rock mass with an apex at mid-depth of the anchor bond length and an apex angle of 90°. No allowance is made in this calculation for the strength of the rock along the surface of the mobilized rock mass which can be considered to provide an adequate factor of safety against applied uplift forces. Group effects should be considered in the calculation if more than one anchor is required to support the applied load.

The rock/grout bond strength should be confirmed in the field with pre-production tests as discussed in Section 6.5.2.

Adequate corrosion protection should be provided for the steel anchors with consideration given to the design life of the structure.

6.4 Construction Considerations

6.4.1 Control of Soil and Groundwater

Water-bearing granular soil lenses or interlayers within the cohesive deposits were not encountered in drilling but are known to exist in glaciolacustrine deposits. Depending on the season and weather patterns leading to and during and during construction, “perched” groundwater could be encountered in the cohesive deposits.

The cohesive deposits encountered in the boreholes are stiff to very stiff. Provided wet granular seams or layers are not penetrated, the caisson sidewalls are expected to remain self-supporting over the estimated 2.4 m to 3.6 m depth of the caisson length in overburden soil. Wet granular soil lenses or interlayers (if encountered) should be expected to run or flow into the drilled hole during or after augering for the caisson. If these are encountered, provisions should be considered for using temporary or permanent caisson liners are recommended to minimize ground loss during drilling cleaning of the caisson and to allow for concrete placement fully to the bottom of the caisson.

Concrete placement in the caisson excavation could occur in the wet, depending on the final caisson design and the contractor’s methodology. Dewatering should be considered and carried out prior to caisson concrete placement. Alternatively, the concrete should be placed using the tremie method.

6.4.2 Cobbles and Boulders

While cobbles, boulders and other obstructions were not encountered in the boreholes, erratic clasts can sometimes occur in glaciolacustrine sediments. Appropriate equipment and procedures should be used to penetrate cobbles and boulders during the caisson advancement.

6.5 Uplift Anchor Installation

Design and construction of rock anchors should comply with the requirements of OPSS 942. PTI (2014) provides additional comment and recommendations for design and construction of uplift ground anchors.

6.5.1 Potential for Grout Loss

Water return during rock coring was variable, generally better than about 80 percent within the upper 3 m of the bedrock, decreasing to less than about 50 percent below about 3.5 m to 4.5 m below the rock surface. These conditions suggest that there is potential for grout loss during anchor installation in the more fractured zones of the bedrock. Grout loss can cause decreased anchor load capacities and reduced corrosion protection of the anchor, as well as possible environmental issues associated with the migration of the cement grout. Improved grout retention can be achieved by adjusting the rheology of the grout.

6.5.2 Anchor Load Testing

It is recommended that the rock/grout bond strength be verified in the field prior to production anchor installation with pre-production performance tests on two anchors, on at each of two HML locations. Proof testing should be carried out in accordance with OPSS 942 on all production anchors to confirm that the anchors are capable of providing consistent performance.

Performance testing should be conducted by incrementally loading and unloading the anchor in accordance with OPSS 942. In the case of the pre-production anchors, it is recommended that the anchors be loaded up to 2.5

times the design load (but not more that 80 percent of the steel tendon yield stress), to provide an opportunity to confirm the assumed bond stress has been achieved.

This Foundation Investigation Report was prepared by Andrew Van Dyk, P.Eng. Mr. Jorge M. A. Costa, P.Eng., a Designated MTO Foundations Contact and Senior Consultant with Golder, conducted an independent review and quality control audit of this report.

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Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors

TABLE 1
GEOTECHNICAL DESIGN PARAMETERS FOR HIGH MAST LIGHT POLE FOUNDATIONS
HIGHWAY 406/58 - St. DAVID'S ROAD INTERCHANGE

Pole No.	Reference Borehole (s)	Ground Surface Elevation at Reference Borehole(s) (m)	Generalized Stratigraphy: Strata Descriptions	Generalized Stratigraphy: Depth Below Existing Ground Surface at Proposed HML Pole Location (m)	Generalized Stratigraphy: Elevation (m)	Groundwater Elevation (m)	Design Parameters [1,2]: q_u (kPa)	ϕ'	γ (kN/m^3)	γ' (kN/m^3)	K_p	f_{horiz} (MPa)	f_{vert} (MPa)
P1	P1	175.4	Topsoil	0.0 to 0.2	175.4 to 175.2	168.3	-	-	-	-	-	-	-
			Stiff CLAYEY SILT, some sand (FILL)	0.2 to 1.1	175.2 to 174.3		150	28	23.0	13.0	2.8	-	-
			Stiff to Very stiff SILTY CLAY	1.1 to 2.4	174.3 to 173.0		200	23	21.5	11.5	2.3	-	-
			Slightly weathered, strong DOLOMITE (BEDROCK)	2.4 to 9.9	173.0 to 165.5		-	-	26.0	16.0	-	30	30
P2	P2	176.5	Topsoil	0.0 to 0.2	176.5 to 176.3	-	-	-	-	-	-	-	-
			Stiff to Very stiff SILTY CLAY	0.2 to 3.6	176.3 to 172.9		200	23	20.5	10.5	2.3	-	-
			Slightly weathered, strong DOLIMITE (BEDROCK)	3.6 to 9.5	172.9 to 167.0		-	-	26.5	16.5	-	30	30
P3	P3	176.0	Topsoil	0.0 to 0.2	176.0 to 175.8	169.7	-	-	-	-	-	-	-
			Stiff to Very stiff SILTY CLAY	0.2 to 3.2	175.8 to 172.8		200	23	21.0	11.0	2.3	-	-
			Slightly weathered, strong DOLIMITE (BEDROCK)	3.2 to 9.3	172.8 to 166.8		-	-	25.5	15.5	-	-	-

NOTES:

1. q_u = unconfined compressive strength = 2 x undrained shear strength (kPa);
   ϕ' = effective friction angle (degrees);
   γ = bulk unit weight (kN/m^3);
   γ' = effective unit weight below the groundwater level (kN/m^3);
   K_p = passive earth pressure coefficient
   $f_{horiz,}$ = unfactored lateral (horizontal) geotechnical resistance
   f_{vert} = unfactored vertical (bearing) geotechnical resistance

2. Passibe Resistance should be neglected for the upper 1.2 m of the subsurface soil to account for frost penetration.

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

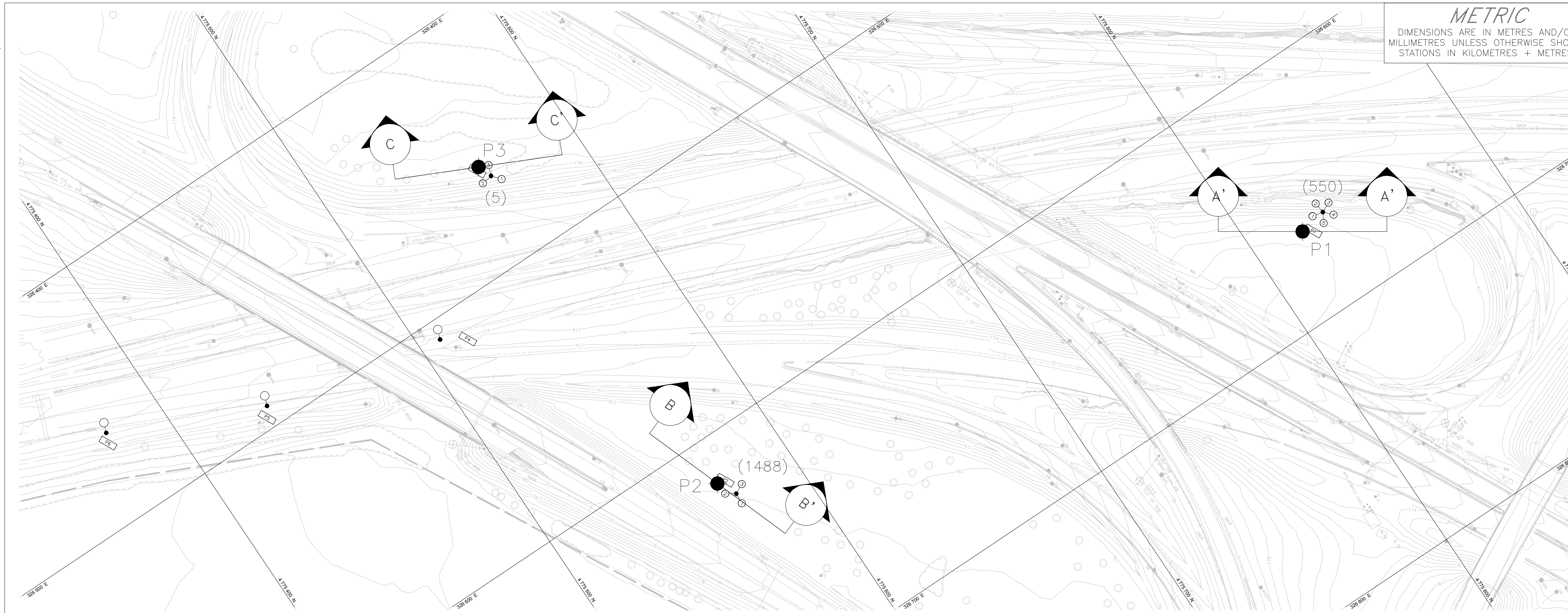
CONT No.
WP No.

SHEET

HIGHWAY 406– ST. DAVID'S ROAD/HWY 58
HIGH MAST LIGHT POLES

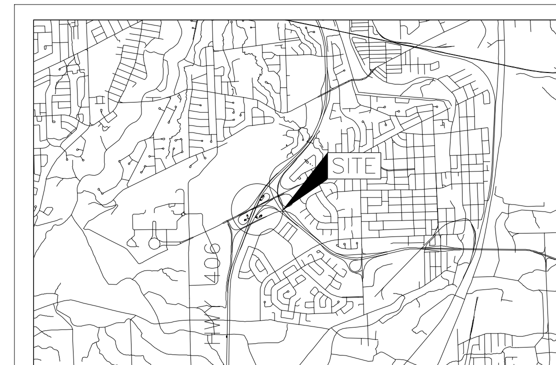
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates



PLAN

SCALE 20 0 20 40 m

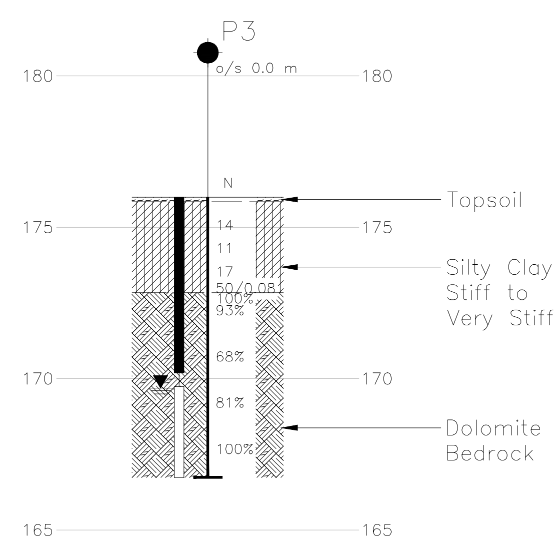


KEY PLAN

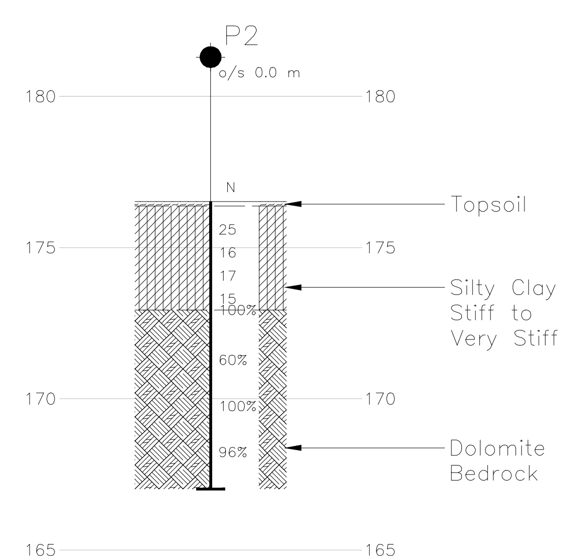
SCALE 1 0 1 2 km

LEGEND

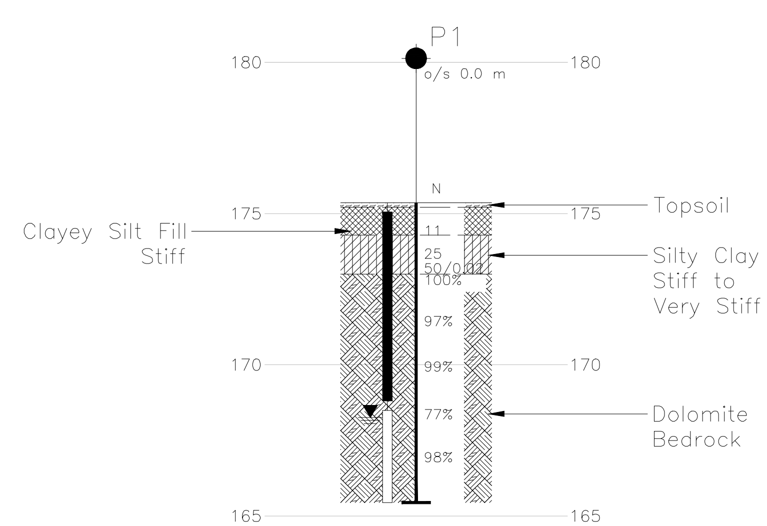
●	Borehole – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on February 3, 2017



CROSS SECTION C – C'



CROSS SECTION B – B'



CROSS SECTION A – A'

HORIZONTAL SCALE 5 0 5 10 m

2.5 0 2.5 5 m VERTICAL SCALE

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
P1	175.4	4775820.9	326664.9
P2	176.5	4775569.0	326618.1
P3	176.0	4775560.1	326458.9

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

Base plans provided in digital format by HDR, drawing file nos. StDavids_Hwy406&58_Base.dwg received December 8, 2016 and 4796 HM–HDR.dwg, received December 16, 2016.

NO.	DATE	BY	REVISION

Geocres No.

HWY.		PROJECT NO. 11–1111–0067	DIST.
SUBM'D. AVD	CHKD. AVD	DATE: 3/10/2017	SITE:
DRAWN: DD	CHKD.	APPD. JMAC	DWG. 1

DRAFT

APPENDIX A

List of Symbols
List of Abbreviations
Record of Boreholes: P1, P2 and P3
Record of Drillholes: P1, P2 and P3

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index / **Relative Density**	**N** / **Blows/300 mm or Blows/ft**
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u **kPa**	c_u, s_u **psf**
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^{o} angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Golder Associates

RECORD OF BOREHOLE No P1

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0067

W.P. --

LOCATION N 4775821.0; E 326664.9 MTM ZONE 10 (LAT. 43.04693817; LONG. -75.33826278)

ORIGINATED BY MC

DIST -- HWY 406

BOREHOLE TYPE 150 mm Diameter Solid Stem Auger

COMPILED BY SZ

DATUM Geodetic

DATE December 19, 2016

CHECKED BY AVD

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
175.4	GROUND SURFACE							
0.0	TOPSOIL				175			
0.2	Clayey silt, some sand, some gravel, trace organics (FILL) Stiff Brown	1A						
174.3 / 1.1	SILTY CLAY, trace to some sand, trace gravel Stiff to very stiff Brown to grey	1B	SS	11	174	50		0 7 33 60
		2	SS	25				
173.0		3	SS	50/0.02	173			
2.4	DOLOMITE (BEDROCK) Bedrock cored from depths of 2.4 m and 9.9 m. For bedrock coring details refer to Record of Drillhole P1.	1	RC	REC 100%	172			RQD = 100%
		2	RC	REC 97%	171, 170		25.8	RQD = 97%
		3	RC	REC 99%	169			RQD = 99%
		4	RC	REC 89%	168, 167			RQD = 77%
		5	RC	REC 98%	166			RQD = 98%
165.5 / 9.9	END OF BOREHOLE							

NOTES:

1. Borehole was dry prior to coring.

2. Water level measured in piezometer at a depth of 7.1 m below ground surface (Elev. 168.3 m) on February 3, 2017.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: P1

PROJECT: 11-1111-006711-1111-0067 | SHEET 1 OF 1

LOCATION: N 4775821.0 ;E 326664.9 | DRILLING DATE: December 19, 2016 | DATUM: Geodetic

INCLINATION: -90° AZIMUTH: --- | DRILL RIG: Geoprobe 7822 DT

DRILLING CONTRACTOR: Determination Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole P1		172.99		
	HQ Rock Core, December 19, 2017	Slightly weathered, thin to medium bedded, grey, fine grained, moderately porous, very strong DOLOMITE (LOCKPORT FORMATION)		2.36		
3					1	
4					2	BENTONITE UCS = 90 MPa
5						
6					3	
7						Feb. 3, 2017
8					4	SCREEN AND SAND
9					5	
				165.44 9.91		
10		END OF DRILLHOLE				
11						
12						

Other column headings: PENETRATION RATE min/(m); COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 m 5 10 15 20; DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja; Jn); ROCK STRENGTH INDEX R4 R3 R2 R1; WEATHERING INDEX W1 W2 W3 W4; Q AVG.

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: MC

CHECKED: AVD

GTA-RCK 004 T:\PROJECTS\2011\11-1111-0067 (HDR HIGH MAST LIGHTS, NIAGARA REGION)\LOG\11-1111-0067.GPJ GAL-MISS.GDT 03/10/17

Golder Associates

RECORD OF BOREHOLE No P2

SHEET 1 OF 1

METRIC

PROJECT 11-1111-0067

W.P. --

DIST -- HWY 406

DATUM Geodetic

LOCATION N 4775568.9; E 326618.1 MTM ZONE 10 (LAT. 43.04468726; LONG. -75.3389343)

BOREHOLE TYPE 150 mm Diameter Solid Stem Auger

DATE December 21, 2016

ORIGINATED BY MC

COMPILED BY SZ

CHECKED BY AVD

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.5	GROUND SURFACE					
0.0	TOPSOIL					
0.2	SILTY CLAY, trace sand, trace gravel Stiff to very stiff					
		1	SS	25		
		2	SS	16		
		3	SS	17		0 3 46 51
		4	SS	15		
172.9 3.6	DOLOMITE (BEDROCK) Bedrock cored from depths of 3.6 m and 9.5 m. For bedrock coring details refer to Record of Drillhole P2.					
		1	RC	REC 100%	26.6	RQD = 100%
		2	RC	REC 98%		RQD = 60%
		3	RC	REC 100%		RQD = 100%
		4	RC	REC 96%		RQD = 96%
167.0 9.5	END OF BOREHOLE NOTE: 1. Borehole was dry prior to coring.					

Elevation scale: 176, 175, 174, 173, 172, 171, 170, 169, 168

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30

+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\2011\11-1111-0067 (HDR HIGH MAST LIGHTS, NIAGARA REGION)\LOG\11-1111-0067.GPJ GAL-GTA.GDT 03/10/17

RECORD OF DRILLHOLE: P2

PROJECT: 11-1111-006711-1111-0067

SHEET 1 OF 1

LOCATION: N 4775568.9 ;E 326618.1

DRILLING DATE: December 21, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Geoprobe 7822 DT

DRILLING CONTRACTOR: Determination Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate; BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage; PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular; PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break; BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 m (5 10 15 20)	DISCONTINUITY DATA B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole P2		172.94																	
4	HQ Rock Core, December 21, 2017	Slightly weathered, thin to medium bedded, grey, fine grained, moderately porous, very strong DOLOMITE (LOCKPORT FORMATION)		3.58	1																UCS = 70 MPa
5–6		- Silty clay, some sand at 5.2 m depth			2																
7–8					3																
9				167.02 / 9.50	4																
		END OF DRILLHOLE																			

Depth scale: 4, 5, 6, 7, 8, 9, 10, 11, 12, 13

GTA-RCK 004 T:\PROJECTS\2011\11-1111-0067 (HDR HIGH MAST LIGHTS, NIAGARA REGION)\LOG\11-1111-0067.GPJ GAL-MISS.GDT 03/10/17

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MC

CHECKED: AVD

RECORD OF BOREHOLE No P3

SHEET 1 OF 1 **METRIC**

PROJECT 11-1111-0067

W.P. --

LOCATION N 4775560.1; E 326458.9 MTM ZONE 10 (LAT. 43.04465309; LONG. -75.34088781)

ORIGINATED BY MC

DIST -- HWY 406

BOREHOLE TYPE 150 mm Diameter Solid Stem Auger

COMPILED BY SZ

DATUM Geodetic

DATE December 20, 2016

CHECKED BY AVD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.0	GROUND SURFACE							
0.0	TOPSOIL							
0.2	SILTY CLAY, trace sand, trace gravel Stiff to very stiff Brown/grey	1	SS	14	175			
		2	SS	11	174	47		0 3 29 68
		3	SS	17				
172.9		4	SS	50/0.08	173			
3.2	DOLOMITE (BEDROCK) Bedrock cored from depths of 3.2 m and 9.3 m. For bedrock coring details refer to Record of Drillhole P1.	1	RC	REC 100%				RQD = 100%
		2	RC	REC 93%	172, 171			RQD = 93%
		3	RC	REC 100%	170			RQD = 68%
		4	RC	REC 85%	169, 168		25.3	RQD = 81%
166.8		5	RC	REC 100%	167			RQD = 100%
9.3	END OF BOREHOLE							

NOTES:

1. Borehole was dry prior to coring.

2. Water level measured in piezometer at a depth of 6.3 m below ground surface (Elev. 169.7 m) on February 3, 2017.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

WATER CONTENT (%): 10 20 30

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 11-1111-006711-1111-0067

RECORD OF DRILLHOLE: P3

LOCATION: N 4775560.1 ;E 326458.9

DRILLING DATE: December 20, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Geoprobe 7822 DT

DRILLING CONTRACTOR: Determination Drilling

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Borehole P3			
	HQ Rock Core, December 20, 2017	Slightly weathered, thin to medium bedded, grey, fine grained, moderately porous, very strong DOLOMITE (LOCKPORT FORMATION)	172.84 3.15	1	
4		- Clay seam at 3.7 m depth		2	BENTONITE
5					
6				3	Feb. 3, 2017
7				4	UCS = 71 MPa SCREEN AND SAND
8					
9				5	
		END OF DRILLHOLE	166.74 9.25		
10					
11					
12					
13					

Other columns: SYMBOLIC LOG; PENETRATION RATE min/(m); FLUSH; COLOUR % RETURN; RECOVERY (TOTAL CORE %, SOLID CORE %) 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX PER 0.3 m 5 10 15 20; DISCONTINUITY DATA (B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr Ja Jn); HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Diametral Point Load Index (MPa) 2 4 6; RMC -Q' AVG.

DRAFT

DEPTH SCALE

1 : 50

Golder Associates

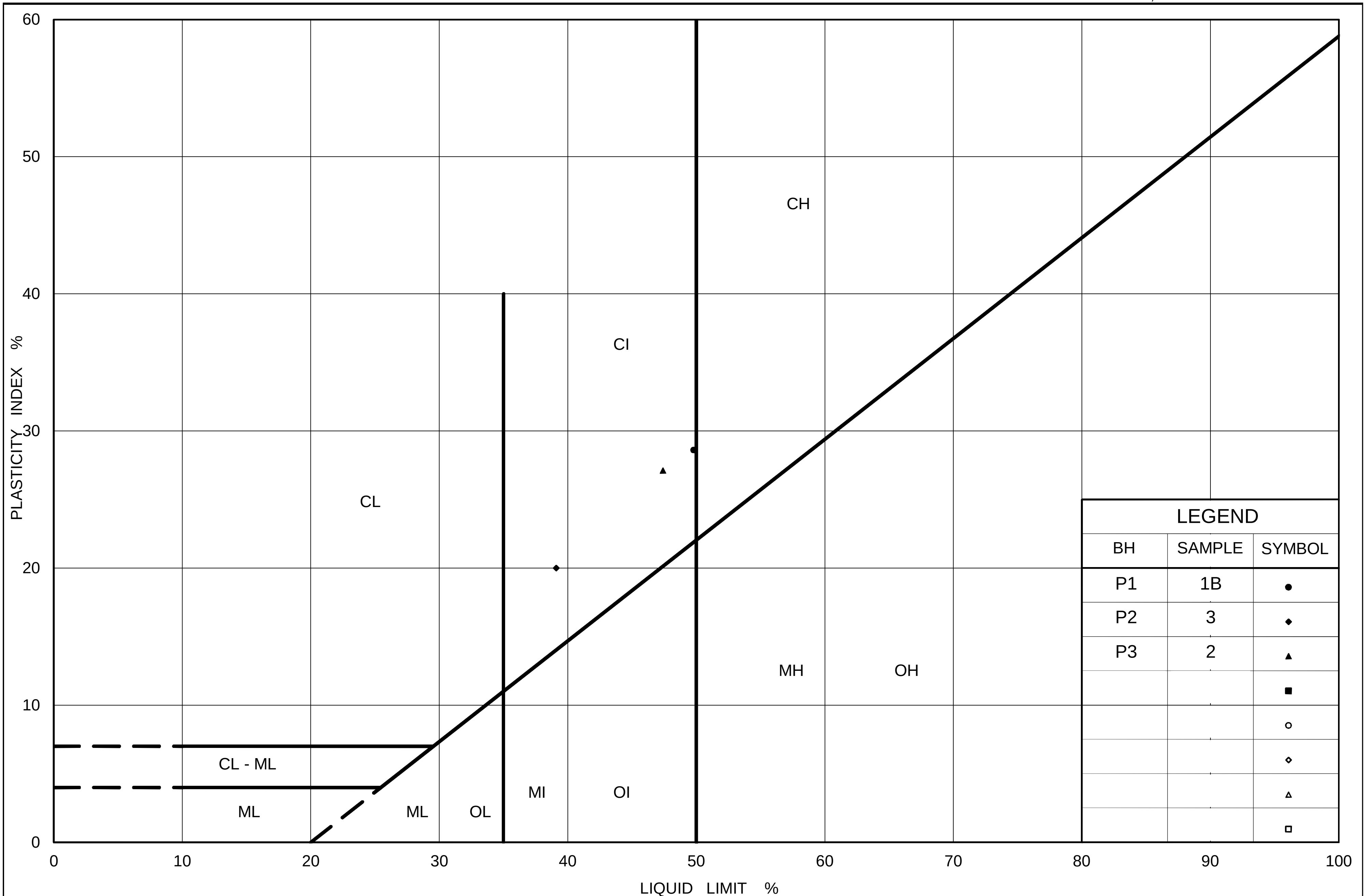
LOGGED: MC

CHECKED: AVD

APPENDIX B

Laboratory Test Results: Figures B1 and B2
Rock Core Photographs: Figures B3 to B11
Unconfined Compression Test Results

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
SILTY CLAY

Figure No. B1

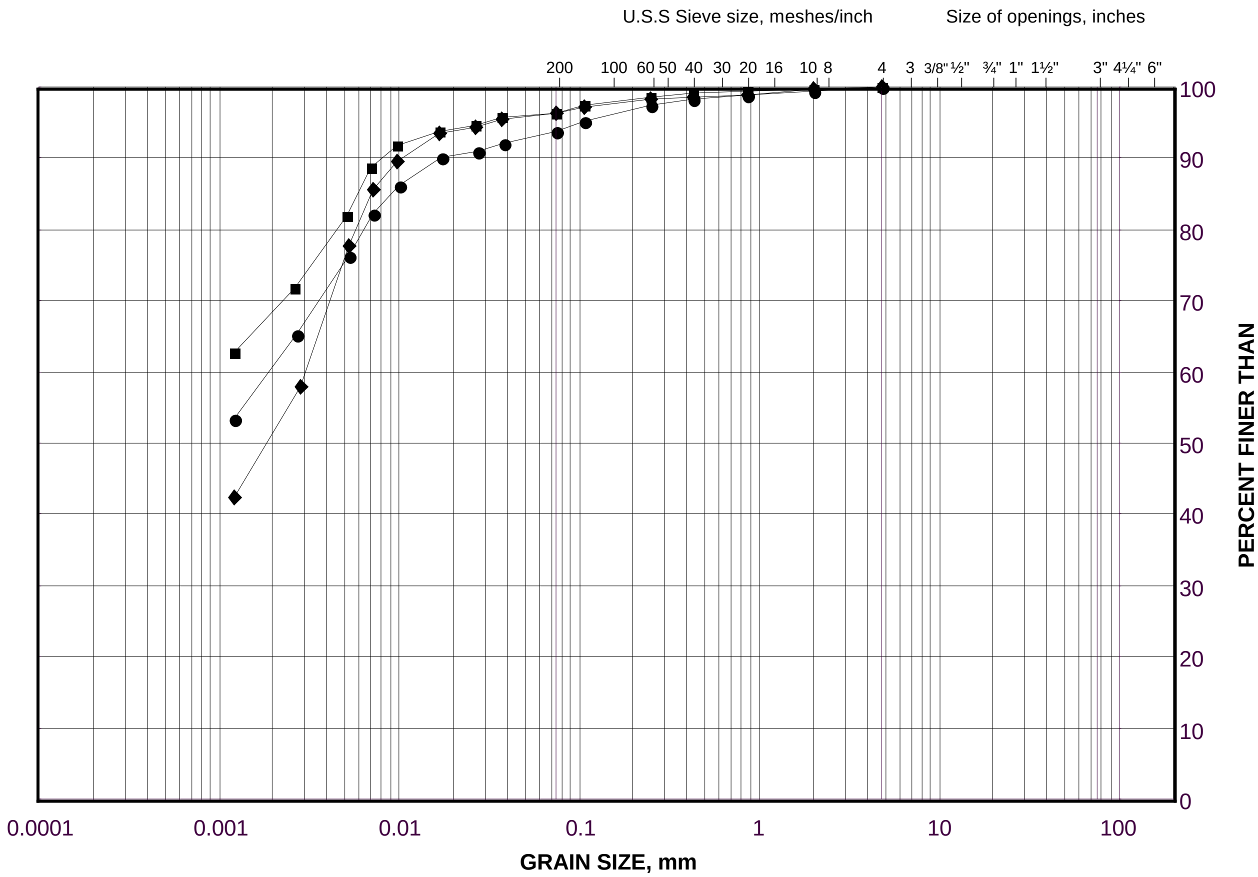
Project No. 11-1111-0067

Checked By: AVD

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE B2



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION
●	P1	1B	174.2
■	P3	2	174.2
◆	P2	3	173.9

Project Number: 11-1111-0067

Checked By: AVD

Golder Associates

Date: 07-Feb-17

DRAFT

CLIENT

HDR, Inc.

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-03-09
PREPARED	SLM
DESIGN	AVD
REVIEW	AVD
APPROVED	JMAC

PROJECT

HIGH MAST LIGHT POLE FOUNDATIONS
HWY 58/HWY 406 AND ST. DAVID'S ROAD INTERCHANGE
G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P1 – RUNS 1 & 2

PROJECT No.	Rev	FIGURE
11-1111-0067		**B3**

Path: 11-1111-0067 rock cores 09Mar2017.pptx | N:\Active\2011\1111\11-1111-0067 HDR- High Mast Lights - Niagara Region\reporting\Draft\Appendix B

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G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P1 – RUNS 1 & 2

PROJECT No.	Rev	FIGURE
11-1111-0067		**B4**

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G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P1 – RUNS 3 & 4

PROJECT No.	Rev	FIGURE
11-1111-0067		**B5**

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G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P2 – RUNS 1 & 2

PROJECT No.	Rev	FIGURE
11-1111-0067		**B6**

Path: 11-1111-0067 rock cores 09Mar2017.pptx | N:\Active\2011\1111\11-1111-0067 HDR- High Mast Lights - Niagara Region\reporting\Draft\Appendix B

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REVIEW	AVD
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PROJECT

HIGH MAST LIGHT POLE FOUNDATIONS
HWY 58/HWY 406 AND ST. DAVID'S ROAD INTERCHANGE
G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P2 – RUN 3

PROJECT No.	Rev	FIGURE
11-1111-0067		**B7**

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HWY 58/HWY 406 AND ST. DAVID'S ROAD INTERCHANGE
G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P2 – RUN 4

PROJECT No.	Rev	FIGURE
11-1111-0067		**B8**

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PROJECT

HIGH MAST LIGHT POLE FOUNDATIONS
HWY 58/HWY 406 AND ST. DAVID'S ROAD INTERCHANGE
G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P3 – RUNS 1 & 2

PROJECT No.	Rev	FIGURE
11-1111-0067		**B9**

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PREPARED	SLM
DESIGN	AVD
REVIEW	AVD
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PROJECT

HIGH MAST LIGHT POLE FOUNDATIONS
HWY 58/HWY 406 AND ST. DAVID'S ROAD INTERCHANGE
G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P3 – RUNS 3 & 4

PROJECT No.	Rev	FIGURE
11-1111-0067		**B10**

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PREPARED	SLM
DESIGN	AVD
REVIEW	AVD
APPROVED	JMAC

PROJECT

HIGH MAST LIGHT POLE FOUNDATIONS
HWY 58/HWY 406 AND ST. DAVID'S ROAD INTERCHANGE
G.W.P. 2364-09-00

TITLE

ROCK CORE PHOTOGRAPHS
BOREHOLE P3 – RUN 5

PROJECT No.	Rev	FIGURE
11-1111-0067		**B11**

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0067	SAMPLE NUMBER	P1-01
PROJECT NAME	HDR HIGH MAST LIGHTS/NIAGARA REGION	SAMPLE DEPTH, m	4.58-4.79
BOREHOLE NUMBER	P1	DATE:	Jan.18, 2017

TEST CONDITIONS

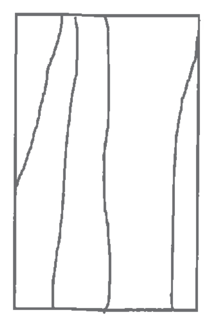
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.33

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	14.86	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	6.38	UNIT WEIGHT, kN/m^3	25.84
SAMPLE AREA, cm^2	31.98	DRY UNIT WT., kN/m^3	25.81
SAMPLE VOLUME, cm^3	475.34	SPECIFIC GRAVITY	-
WET WEIGHT, g	1252.75	VOID RATIO	-
DRY WEIGHT, g	1251.50		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	90.4

REMARKS:

Checked By: LM

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0067	SAMPLE NUMBER	P2-01
PROJECT NAME	HDR HIGH MAST LIGHTS/NIAGARA REGION	SAMPLE DEPTH, m	4.10-4.34
BOREHOLE NUMBER	P2	DATE:	Jan.18, 2017

TEST CONDITIONS

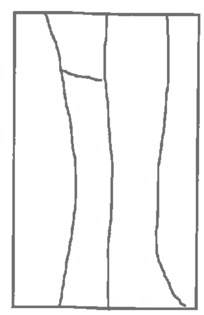
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.34

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	15.02	WATER CONTENT, (specimen) %	0.10
SAMPLE DIAMETER, cm	6.41	UNIT WEIGHT, kN/m^3	26.63
SAMPLE AREA, cm^2	32.31	DRY UNIT WT., kN/m^3	26.60
SAMPLE VOLUME, cm^3	485.41	SPECIFIC GRAVITY	-
WET WEIGHT, g	1318.66	VOID RATIO	-
DRY WEIGHT, g	1317.34		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	69.5

REMARKS:

Checked By: LU

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0067	SAMPLE NUMBER	P3-01
PROJECT NAME	HDR HIGH MAST LIGHTS/NIAGARA REGION	SAMPLE DEPTH, m	7.52-7.72
BOREHOLE NUMBER	P3	DATE:	Jan.18, 2017

TEST CONDITIONS

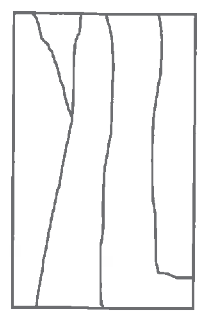
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.39

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	15.29	WATER CONTENT, (specimen) %	0.20
SAMPLE DIAMETER, cm	6.40	UNIT WEIGHT, kN/m^3	25.28
SAMPLE AREA, cm^2	32.17	DRY UNIT WT., kN/m^3	25.23
SAMPLE VOLUME, cm^3	491.94	SPECIFIC GRAVITY	-
WET WEIGHT, g	1268.74	VOID RATIO	-
DRY WEIGHT, g	1266.21		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	70.5

REMARKS:

Checked By: *[signature]*

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APPENDIX C

Non-standard Special Provision: Bedrock Strength

Socket into Bedrock – Item No.

Non-Standard Special Provision

The High Mast Light Pole foundations will require sockets to be formed within the bedrock, which is strong, based on an Unconfined Compressive Strength test result on a bedrock core sample at each HML Pole location (i.e., uniaxial compressive strengths ranging from 70 MPa to 90 MPa). It is anticipated that is will be necessary to use the rock coring or churn drilling techniques to advance the caisson holes into the bedrock.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

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