January 2016

REPORT ON

Foundation Investigation and Design
Tri-Chord Overhead Signs
Highway 416 (North) and Highway 417 (West)
Ottawa, Ontario
W.P. 4184-15-01

Submitted to:
Ontario Ministry of Transportation
1355 John Counter Boulevard
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REPORT

Report Number: 1413191-1020-001 Rev-001

GEOCRES Number: 31G5-270

Distribution:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the design and construction of proposed replacement overhead sign structures on Highway 416 and Highway 417 in Ottawa, Ontario.

The original assignment included foundation investigation at a total of 26 proposed replacement tri-chord overhead signs. This report presents the results of a foundation assessment conducted for five of the overhead signs: four located on northbound Highway 416 between Hunt Club Road and Highway 417, and one located on westbound Highway 417 between Richmond Road and Holly Acres Road. The approximate sign locations are shown on Drawings 1 to 3. The purpose of the investigation was to assess the subsurface conditions at the locations of the existing signs by borehole drilling and carrying out in-situ and laboratory testing on selected samples.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Work Item Order Form for Assignment No. 3 as part of Agreement No. 4014-E-0012 received on September 22, 2015, and in Golder's Work Item Quote Form and Understanding of Scope documents submitted on September 22, 2015 and approved by MTO on September 29, 2015.

2.0 SITE DESCRIPTION

2.1 General

The locations of the existing tri-chord overhead signs that are to be replaced are along northbound Highway 416 between Hunt Club Road and Highway 417, and on westbound Highway 417 between Richmond Road and Holly Acres Road in Ottawa, Ontario. The proposed sign locations are shown on Drawings 1 to 3 and are summarized in the following table:

Sign Number		Highway	Sign Description
Golder	MTO		
01	416-0073.2	416	417 East Ottawa / 417 West Kanata Pembroke (Advanced)
02	416-0073.3	416	417 East Ottawa / 417 West Kanata Pembroke (Exit)
03	416-0074.0	416	417 East Ottawa / Holly Acres Rd. Richmond Rd. (Advanced)
04	416-0074.4	416	417 East Ottawa (Exit) / Holly Acres Rd. Richmond Rd.
05	417-0131.0	417	416 South (Exit) / 417 West

In the area of interest, Highway 416 is generally in cut between the Bruin Road underpass and Highway 417. The natural ground surface elevation varies in height above the highway grade and is separated from the highway alignment with sloped ground (from the Bruin Road underpass to about 200 m south of the rail underpass), retaining walls several metres high (from about 200 m south of the rail underpass to about 150 m south of the Baseline Road underpass), and near-vertical rock cuts (from about 150 m south of the Baseline Road underpass to the connection with Highway 417).

Near sign 417-0131.0, Highway 417 is constructed on embankments that are up to about 3 m above the natural ground surface, and increase in height to the west at the Highway 416 interchange.

2.2 Regional Geology

This area of Highway 416 and 417 lies within the physiographic region known as the Ottawa Valley Clay Flats adjacent to the Ottawa River, as delineated in The Physiography of Southern Ontario[1].

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silty clay and silt that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.[1]

[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*. Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed tri-chord overhead sign replacements was carried out between September 30 and October 28, 2015. During that time, a total of 15 boreholes were advanced at the locations of the proposed sign foundations as part of the overall assignment. This report addresses six of the boreholes put down at the five proposed sign locations listed in Section 2.1.

The boreholes were advanced adjacent to or within the shoulder or median lane using 108 mm inside diameter continuous-flight hollow-stem augers with a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario or George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec. The boreholes were advanced to depths of up to about 10.4 m below the existing pavement/ground surface in the overburden. Where encountered within the upper 7 m, the boreholes were then cored about 3 m into the bedrock using NQ-size coring equipment.

Soil samples in the boreholes were obtained at vertical intervals of about 0.76 to 1.52 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing (using an MTO N-size vane) was carried out within the cohesive deposits where possible.

The boreholes were backfilled with bentonite pellets mixed with native soils in the overburden, bentonite pellets in the bedrock, and compacted cold-patch asphalt at surface. The site conditions were restored following completion of work.

The field work was supervised by members of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory facility in Ottawa for further examination. Index and classification tests consisting of grain size distributions, Atterberg limits, and water contents were carried out on selected soil samples at Golder's Ottawa laboratory. Unconfined compressive strength tests were carried out on selected rock core samples in Golder's Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations were determined by Golder in relation to the existing signs, based on information on the proposed sign locations provided by MTO. The plan location and ground surface elevation at each borehole was surveyed by Golder using a precision GPS survey unit. The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawings 1 to 3.

Borehole Number	Sign Number		Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Golder	MTO	Northing (m)	Easting (m)		
15-01A	01	416-0073.2	5021373.5	359001.5	81.9	9.5
15-02A	02	416-0073.3	5021670.3	358866.2	75.4	4.5
15-03A	03	416-0074.0	5021970.9	358744.7	68.9	4.4
15-04A	04	416-0074.4	5022278.7	358600.0	67.8	4.3
15-04B			5022285.5	358610.9	67.7	5.3
15-05A	05	417-0131.0	5023005.2	358834.4	68.6	10.4

Notes: 1) Northing and Easting coordinates shown are relative to the MTM NAD83 (Zone 9) coordinate system.
2) Ground surface elevations shown are relative to Geodetic Datum.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes advanced as part of this investigation, together with the results of related in-situ and laboratory testing, are given on the Record of Borehole and Drillhole sheets contained in Appendix A. The results of geotechnical laboratory testing carried out as part of this investigation are also included in Figures B1 to B6, in Appendix B.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred 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 pavement structure and fill overlying either firm to very stiff silty clay to clay and silty sand, or bedrock consisting of dolostone or sandstone.

The following table summarizes the subsurface conditions encountered at the borehole locations, and a more detailed description of the soils and bedrock is provided in the subsections that follow.

Sign Number		Borehole Number(s)	Summary of Subsurface Conditions Encountered in Boreholes
Golder	MTO		
01	416-0073.2	15-01A	Asphaltic concrete is underlain by 1.0 m of granular base and subbase consisting of gravelly sand which is, in turn, underlain by 6.7 m of silty clay to clay. The silty clay to clay is typically firm to stiff, grading to stiff with depth, and is underlain by about 1.6 m of silty sand to sand till. Drilling met auger refusal at a depth of about 9.5 m (Elevation 72.4 m) in the median area below the existing Highway 416 grade.
02	416-0073.3	15-02A	Asphaltic concrete is underlain by 0.9 m of granular base and subbase consisting of gravelly sand which is, in turn, underlain by dolostone bedrock. The bedrock is at about Elevation 74.5 m in the median area. Bedrock was proven to a depth of about 4.5 m (Elevation 70.9 m) below the existing Highway 416 grade.
03	416-0074.0	15-03A	Asphaltic concrete is underlain by 1.0 m of granular base and subbase consisting of gravelly sand which is, in turn, underlain by sandstone bedrock. The bedrock is at about Elevation 67.5 m near the northbound shoulder. Bedrock was proven to a depth of about 4.4 m (Elevation 64.5 m) below the existing Highway 416 grade.
04	416-0074.4	15-04A, 15-04B	Asphaltic concrete is underlain by 1.0 to 1.4 m of granular base and subbase consisting of gravelly sand and silty sand which is, in turn, underlain by sandstone bedrock. The bedrock is at about Elevation 66.8 m near the northbound median and Elevation 66.3 m at the northbound shoulder. Bedrock was proven to depths of about 4.2 to 5.3 m (Elevations 62.4 to 63.5 m) below the existing Highway 416 grade.

Sign Number		Borehole Number(s)	Summary of Subsurface Conditions Encountered in Boreholes
Golder	MTO		
05	417-0131.0	15-05A	Near the Highway 417 median, asphaltic concrete is underlain by 0.8 m of granular base and subbase consisting of gravelly sand which is, in turn, underlain by about 2.0 m of sandy embankment fill. Silty clay was encountered below the fill to the termination of the borehole at a depth of 10.4 m (Elevation 58.2 m) below the existing Highway 417 grade. The lower portion of the silty clay is firm to stiff and overlain by about 2.5 m of silty clay that is weathered to a stiff to very stiff crust.

4.1.1 Pavement Structure and Embankment Fill

Boreholes 15-01A, 15-02A, 15-03A, and 15-04A were advanced through the pavement structure of the northbound median lane of Highway 416. The pavement structure generally consists of 300 mm of asphaltic concrete overlying up to about 1.0 m of gravelly sand fill.

Borehole 15-04B was advanced through the pavement structure of the northbound shoulder of Highway 416. The pavement structure consists of 100 mm of asphaltic concrete over about 700 mm of gravelly sand fill. The pavement structure is underlain by about 600 mm of gravel and sand fill, containing some silt.

Borehole 15-05A was advanced through the pavement structure of the westbound median lane of Highway 417. The pavement structure consists of 100 mm of asphaltic concrete overlying 300 mm of gravelly sand fill and is underlain by about 400 mm of sandy gravel fill. The pavement structure is underlain by a layer of embankment fill to a depth of about 2.8 m below the existing roadway surface. The embankment fill generally consists of silty sand with some gravel. A layer of asphaltic concrete was encountered at a depth of about 1.5 m below the existing roadway surface.

At Boreholes 15-03A and 15-04A, a layer of cobbles and boulders was encountered beneath the pavement structure/grade fill at depths of about 1.0 m and 0.9 m below the existing ground surface, respectively. The layer of cobbles and boulders is up to about 0.4 m thick and was encountered to depths of about 1.4 and 1.0 m below the existing ground surface at boreholes 15-03A and 15-04A, respectively.

The Standard Penetration Test (SPT) “N” values measured within the fill encountered at the Boreholes 15-02A, 15-03A, 15-04A, and 15-04B range from 67 to greater than 100 blows per 0.3 m of penetration, indicating very dense relative density. At Boreholes 15-01A and 15-05A, the SPT “N” values measured in the fill typically range between 18 and 20 blows per 0.3 m of penetration, indicating a compact relative density (the higher blow count recorded in the fill at Borehole 15-05A likely reflects the presence of the buried asphaltic concrete layer rather than the relative density of the soil matrix).

The results of grain size distribution testing carried out on two samples of fill are shown on Figures B1 and B2 in Appendix B. The measured water contents of two samples of the granular fill were 2 and 10 percent.

4.1.2 Silty Clay to Clay

A deposit of silty clay to clay (hereafter referred to as silty clay) was encountered beneath the pavement structure and embankment fill at Boreholes 15-01A and 15-05A. The silty clay deposit was proven to depths ranging from about 7.9 m to 10.4 m below the existing Highway grades (Elevations 58.2 m to 74.0 m).

The upper portion of the silty clay in Borehole 15-05A has been weathered to a grey brown crust. The weathered crust has a thickness of about 2.5 m and extends to a depth of about 5.3 m below the existing ground surface. The weathered deposit contains trace to some sand and silty sand seams.

The SPT "N" values measured within the weathered silty clay crust range from 4 to 16 blows per 0.3 m of penetration indicating a stiff to very stiff consistency.

The measured water content of one sample of the weathered silty clay was about 25 percent.

The full depth of the silty clay in Borehole 15-01A and the silty clay below the depth of weathering in Borehole 15-05A is grey in colour. The grey silty clay extends to depths of about 7.9 and 10.4 m below the existing ground surface. The silty clay deposit contains clayey silt interbeds and silty sand layers.

The SPT "N" values measured within the silty clay range from 'weight of hammer' to 3 blows per 0.3 m of penetration. In situ vane testing carried out within the silty clay gave undrained shear strength ranging from 42 to greater than 96 kilopascals indicating a firm to stiff consistency.

The results of grain size distribution testing carried out on a sample of silty clay to clay recovered from Borehole 15-01A are shown on Figure B3 in Appendix B. The results of grain size distribution testing carried out on two samples of the silty clay portion of the deposit recovered from Borehole 15-05A are shown on Figure B4. The results of grain size distribution testing carried out on one sample of a silty sand layer of the deposit recovered from Borehole 15-05A are shown on Figure B5.

Atterberg limit determination testing carried out on four samples of the silty clay deposit gave plasticity index values ranging from about 9 to 33 percent and liquid limit values ranging from about 23 to 51 percent, indicating that the tested samples consist of silty clay of low to high plasticity. The measured water content of the deposit ranged from approximately 29 to 43 percent.

4.1.3 Sand and Silt

A deposit of sand and silt, exists beneath the silty clay in Borehole 15-01A at a depth of about 7.9 m and was proven to a depth of about 9.5 m (Elevation 72.4 m) below the existing ground surface.

The SPT "N" value of one test completed within the sand and silt was 1 blow per 0.3 m of penetration, indicating a very loose relative density. The SPT "N" value of greater than 50 blows per 0.3 m of penetration measured at the bottom of the deposit was encountered at effective refusal of the sampler on the underlying soil/rock deposit.

The measured water content of one sample of the sand and silt was about 17 percent.

4.1.4 Auger Refusal and Bedrock

SPT sampler and auger refusal was encountered in Borehole 15-01A at a depth of about 9.5 m below the existing ground surface. Bedrock was encountered beneath the pavement structure at Boreholes 15-02A, 15-03A, 15-04A, and 15-04B where it was cored between about 3.0 m and 3.9 m in NQ-size. Borehole 15-05A was terminated within the overburden at the target depth.

The following table summarizes the bedrock surface depths and elevations as encountered at the borehole locations. The bedrock surface elevation varies at each proposed foundation location.

Borehole Number	Sign Number		Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
	Golder	MTO			
15-01A	01	416-0073.2	81.9	9.5(1)	72.4(1)
15-02A	02	416-0073.3	75.4	0.9	74.5
15-03A	03	416-0074.0	68.9	1.4	67.5
15-04A	04	416-0074.4	67.8	1.0	66.8
15-04B	04		67.7	1.4	66.3
15-05A	05	417-0131.0	68.6	N/A(2)	N/A(2)

Notes: 1) Bedrock surface inferred from auger refusal encountered within the borehole.
2) Bedrock not encountered within the advancement depth.

The bedrock encountered in Borehole 15-02A consists of dark grey dolostone bedrock. The bedrock is fresh, thinly to thickly bedded, fine to medium grained, non-porous, and strong.

The bedrock encountered in Boreholes 15-03A, 15-04A, and 15-04B consists of light brown to dark grey sandstone bedrock. The bedrock is fresh, medium to thickly bedded, fine to medium grained, non-porous, slightly calcareous to calcareous, and strong with occasional nodular sections and thin laminations of shale.

The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples typically ranged from about 33 to 100 percent, indicating poor to excellent quality rock. The discontinuities observed in the rock core were associated with the joints, veins, faults and fractures of the bedrock.

Laboratory unconfined compressive strength testing was carried out on selected specimens of the bedrock core from Boreholes 15-02A, 15-03A, and 15-04B. The results of the testing carried out on samples of the dolostone and sandstone bedrock indicate unconfined compressive strengths ranging from about 72 to 86 MPa, which correspond to strong rock (Canadian Foundation Engineering Manual, 2006). The results of the unconfined compressive strength tests are provided on Figure B6 in Appendix B.

4.1.5 Groundwater Conditions

Where visible, the groundwater levels were measured in the open boreholes during drilling. The estimated groundwater levels based on observation during drilling and/or condition of recovered samples are summarized in the table below:

Borehole Number	Sign Number		Existing Ground Surface Elevation (m)	Estimated Water Level Depth (m)	Estimated Water Level Elevation (m)
	Golder	MTO			
15-01A	01	416-0073.2	81.9	1.3	80.6
15-02A	02	416-0073.3	75.4	0.9	74.5
15-03A	03	416-0074.0	68.9	1.4	67.5
15-04A	04	416-0074.4	67.8	1.0	66.8
15-04B	04		67.7	1.4	66.3
15-05A	05	417-0131.0	68.6	4.6	64.0

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Matt Kennedy, P.Eng., a geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent review of this report.

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PART B

FOUNDATION DESIGN REPORT

TRI-CHORD OVERHEAD SIGNS

HIGHWAY 416 (NORTH) AND HIGHWAY 417 (WEST)

OTTAWA, ONTARIO

W.P. 4184-15-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed tri-chord overhead sign replacements on northbound Highway 416, between Hunt Club Road and Highway 417, and on westbound Highway 417, east of Holly Acres Road in Ottawa, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes and drillholes advanced as close as practicable to the proposed replacement sign locations. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives to carry out the detail design of the sign foundations.

Where comments are made on construction, they are provided to highlight those aspects that 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, and scheduling.

A comparison of the foundation alternatives for all sign types is provided in Table 1.

6.2 Design of Sign Foundations

Caisson foundations for trichord overhead signs should be designed in accordance with the standard design methods for Tri-Chord Static Sign Supports, contained in Section 4 and Standard Drawings SS118-3, SS118-4 and SS118-5 of MTO's *Sign Support Manual* (2015).

The standard sign foundation designs presented on the Standard Drawings have been developed based on the minimum soil conditions given below:

- **Case 1 (Cohesionless Soils):** Sand with a friction angle of 28 degrees surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and sand with a friction angle of 30 degrees surrounding the lower third of the portion of the caisson below the design frost depth.

- **Case 2 (Cohesive Soils):** Soft clay with an undrained shear strength of 25 kPa surrounding the upper two-thirds of the portion of the caisson foundation below the frost depth, and "soft" clay with an undrained shear strength of 50 kPa surrounding the lower third of the portion of the caisson below the design frost depth.

In the standard design, caissons are extended 5 m below the design frost depth, unless bedrock is encountered within this depth. For sign foundation design, the frost depth in the Ottawa area may be taken as 1.8 m. The typical caisson founding level would therefore be 6.8 m below the ground surface, except where bedrock is encountered within this depth.

The following table summarizes the depth to bedrock and the bedrock surface elevation in the boreholes at each of the proposed sign support locations where bedrock was encountered within 6.8 metres depth, as determined by bedrock coring:

Sign Number		Borehole(s)	Bedrock Depth [(1)] (m)	Bedrock Surface Elevation (m)	Bedrock Type
Golder	MTO				
01	416-0073.2	15-01A	> 9.5	N/A[(2)]	N/A[(2)]
02	416-0073.3	15-02A	0.9	74.5	Dolostone
03	416-0074.0	15-03A	1.4	67.5	Sandstone
04	416-0074.4	15-04A, 15-4B	1.0 – 1.4	66.3 – 66.8	Sandstone
05	417-0131.0	15-05A	> 10.4	N/A[(2)]	N/A[(2)]

Notes: 1) Depth below existing ground surface at borehole location.
2) Bedrock not encountered within the advancement depth.

The overburden at the sign locations consists of both cohesive and granular materials within the caisson length below the frost depth. The granular and cohesive materials at these locations have friction angles and shear strengths, respectively, which meet or exceed the input parameters used in modelling the standard caisson foundations for both Cases 1 and 2 of the *Sign Support Manual (2015)*. Therefore, where the bedrock depth is greater than 6.8 m (as is the case at 416-0073.2 and 417-0131.0) the foundations for the sign supports may be designed using the standard caisson foundation designs. The standard foundation designs may be checked and optimized if desired by the structural designer using the recommendations provided in Section 6.2.1 for site-specific design and the geotechnical design parameters provided in Table 2.

The depth to bedrock at the remaining sign locations (i.e., 416-0073.3, 416-0074.0, and 416-0074.4) is less than 5 m below the design frost depth (i.e., between 0.9 and 1.4 m depth below the existing ground surface). At these three sites, the overburden soils will not be of sufficient depth to provide the required lateral resistance, and a socket into the rock may be required, as discussed in Section 6.2.2.

6.2.1 Caisson Foundations in Soil

The stratigraphy and design parameters for the subsurface conditions encountered in the boreholes at the sign support locations are given in Table 2, for use in site-specific design.

6.2.1.1 Cohesive Soils

For cohesive soils, the lateral resistance should be checked under drained and undrained conditions to determine which case will govern.

For drained conditions, the unfactored passive lateral earth pressure, P_p (kPa), distributed along the caisson may be calculated using the following equation, based on the stratigraphy and design parameters given in Table 2 for the sign locations:

$P_p = K_p \gamma z + 2 c' \sqrt{K_p}$ Above the groundwater table; and,

$P_p = K_p \gamma z_w - K_p (z - z_w) \gamma' + 2 c' \sqrt{K_p}$ Below the groundwater table.

Where:
- K_p Is the passive earth pressure coefficient;
- γ Is the bulk unit weight (kN/m^3);
- γ' Is the effective unit weight below the groundwater level (kN/m^3);
- z Is the depth below the ground surface (m);
- z_w Is the depth to the groundwater level (m); and,
- c' Is the cohesion (kPa).

In the design of the sign foundations, the passive resistance within the upper 1.8 m below ground surface should be neglected to account for frost action, as allowance for the frost penetration depth for the Ottawa area, as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Depth for Southern Ontario).

For the undrained case, the lateral resistance for the length of the caisson within the cohesive soil should be calculated assuming an internal angle of friction, $\Phi' = 0$ degrees, and an unfactored passive lateral pressure distribution varying from $2 C_u$ at 1.8 m below ground surface (i.e., frost depth) to $9 C_u$ at and below a depth equivalent to three caisson diameters, acting over the actual width of the caisson.

For the drained case, the unfactored lateral resistance should be calculated assuming an equivalent width equal to three times the caisson diameter.

For both the drained and undrained cases, a resistance factor of 0.5 should be applied to the calculated lateral resistance in order to obtain the factored lateral geotechnical resistance at Ultimate Limit States (ULS).

6.2.1.2 Cohesionless Soils

For cohesionless soils, the unfactored passive lateral earth pressure, P_p (kPa), distributed along the caisson may be calculated using the following equation, based on the stratigraphy and parameters given in Table 2 for the sign locations:

$P_p = K_p \gamma z$ Above the groundwater table; and,

$P_p = K_p \gamma z_w + K_p (z - z_w) \gamma'$ Below the groundwater table.

Where:
- K_p Is the passive earth pressure coefficient;
- γ Is the bulk unit weight (kN/m^3);
- γ' Is the effective unit weight below the groundwater level (kN/m^3);
- z Is the depth below the ground surface (m); and,
- z_w is the depth to the groundwater level (m).

The lateral earth pressure may be assumed to act over an equivalent width equal to three times the caisson diameter. In the design of the foundations, the passive resistance within the upper 1.8 m below the ground surface should be neglected to account for frost action. A resistance factor of 0.5 should be applied to this calculated lateral resistance in order to obtain the factored lateral geotechnical resistance at ULS.

6.2.2 Caisson Foundations Embedded or Socketted in Rock

In accordance with Standard Drawing SS118-3 of MTO's Sign Support Manual (2015), where bedrock is encountered at a depth, Y (in m), of less than 5 m below the bottom of the frost layer, the required depth of the foundation below the frost layer can be reduced to:

$$Y + (5\ \text{m} - Y)/2$$

For signs 416-0073.3, 416-0074.0, and 416-0074.4, the depth to the surface of bedrock is less than the frost depth of 1.8 m. Based on the above equation, the caissons for support of this sign will be socketted 2.5 m into the bedrock. At these locations where shallow bedrock sockets are required, the factored passive lateral resistance for caissons in the dolostone and sandstone bedrock may be taken as the follows:

- 15 MPa for the upper 0.5 m of the rock socket (to account for fracturing in the upper portion of the rock mass, based on the lower RQD values in some of the boreholes); and,
- 30 MPa below a depth of 0.5 m below the surface of the bedrock.

From a design/analysis perspective, the lateral resistance would be assumed to act against the projected vertical planar area of the rock face against the side of the sign support.

From a geotechnical perspective, the rock sockets could have a diameter less than 1200 mm (the standard caisson diameter); in general, smaller diameter rock sockets are more readily constructible and more cost effective than larger diameter rock sockets. However the actual size should be determined by the structural designers if optimization of the caisson size is desired.

The bedrock at the proposed sign locations is classified as strong, corresponding to uniaxial compressive strengths for intact rock samples of about 70 to 90 MPa. As such, appropriate equipment and construction procedures (such as rock coring techniques) would be required to advance the sockets into the bedrock. In order to minimize coring within the strong bedrock, consideration could be given to the use of spread footing or caisson foundations anchored to the rock at these locations. Recommendations for the shallow foundations anchored to rock are provided in the following section.

6.2.3 Shallow Foundations Anchored to Rock

A shallow spread footing placed directly on the dolostone or sandstone bedrock at signs 416-0073.3, 416-0074.0, and 416-0074.4 may be designed using a factored vertical geotechnical resistance of the sandstone and dolostone at ULS of 15 MPa, to account for potential weathering/fracturing of the bedrock at the sign support location. The geotechnical reaction at Serviceability Limit States (SLS) will be greater than the ULS resistance.

The calculated geotechnical resistance/reaction for the spread footings do not account for eccentric loading. Eccentric loading should be taken into account by the designer in accordance with Clauses 6.7.4 and C6.7.4 of the Canadian Highway Bridge Design Code (CHBDC, 2006) and the related Commentary.

The horizontal resistance of the footings may be achieved with dowels installed into the underlying bedrock, and would be dependent on the strength of the bedrock, grout and steel. The dowels may be designed based on a factored passive lateral resistance at ULS for the rock mass of 15 MPa in the upper 0.5 m of the bedrock, and 30 MPa below this depth. The rock dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and the compressive strength of the grout should not be exceeded.

For uplift of dowels, an unfactored value of 1,200 kPa may be assumed for the cement grout-to-rock bond stress, based on typical average values in dolostone and sandstone. The upper 0.3 m of the bond length should be ignored in the calculation of required bond length as the rock near surface may be disturbed due to excavation. The actual bond stress along the rock-grout interface may vary from the design value given and it should, therefore, be verified in the field by pull-out testing; in this case, a NSSP will have to be included in the Contract Documents to cover this testing.

A sample NSSP which addresses the supply, installation and testing of rock dowels is included in Appendix C.

6.3 Construction Considerations

All excavation should be inspected prior to placing concrete to ensure that the base has been adequately cleaned and that the subsurface conditions as exposed at the founding level are consistent with the design assumptions. Wherever possible, the foundations should be excavated to provide a flat bearing surface.

At signs 416-0073.3, 416-0074.0, and 416-0073.4, if anchored shallow foundations are adopted, rock protrusions or cavities should be avoided to provide a uniform bearing pressure across the full area of the footing. All loose and/or highly fractured rock within the foundation footprint at the founding level should be removed and replaced with mass concrete or a working slab. The footing foundation should be cleaned of deleterious material using high pressure air and water and inspected by the Quality Verification Engineer prior to placing any concrete for the footings.

Construction of the foundations for the sign support structures should be in accordance with OPSS.PROV 915 (Sign Support Structures) and OPSS.PROV 904 (Concrete Structures).

The overburden soils at the sign locations include granular fill. These soils should be expected to be unstable if the groundwater level, and should be expected to run or flow into the caisson holes during or after drilling for the foundations. Therefore, temporary or permanent caisson liners will be required to minimize ground loss during drilling and concrete placement.

Some of the sign foundations may require sockets to be formed within the bedrock. The bedrock at the site includes strong dolostone and sandstone. It should be anticipated that it will be necessary to use rock coring techniques to advance the caisson holes into the strong bedrock, and that the rate of progress in forming the socket will be slow.

For the construction of concrete caissons, the performance of the rock socket will depend to a large degree on the condition of the bedrock, or other material, at the base of the shaft. The base must be cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the founding stratum. The caisson should be measured for depth to verify/confirm that the entire drilled length is open to the base of the rock socket.

The drilling and construction of the caisson foundations should be observed throughout by the Quality Verification Engineer (QVE) to confirm that the conditions encountered are consistent with the information obtained from the borehole and that the required tip elevation and base cleanliness has been achieved.

It is recommended that the NSSPs presented in Appendix C be included in the Contract Documents to warn the Contractor of the items above which are expected to affect the installation of the sign foundations.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Kennedy, P.Eng., a geotechnical engineer with Golder, with technical input from Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.

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WAM/MJK/ob

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Table 1 – Comparison of Foundation Alternatives

Sign Number		Sign Type	Foundation Location	Caisson Foundation in Soil	Caisson Foundation in Soil with Rock Socket	Caisson Foundation in Soil with Rock Dowels	Spread Footings on Rock with Rock Dowels
Golder	MTO						
01	416-0073.2	Tri-Chord Overhead	Median	Preferred Alternative	Not practical due to bedrock depth	Not practical due to bedrock depth	Not practical due to bedrock depth
			Right Shoulder (1)	Preferred Alternative (1)	Not practical due to bedrock depth (1)	Not practical due to bedrock depth (1)	Not practical due to bedrock depth (1)
02	416-0073.3	Tri-Chord Overhead	Median	N/A – Rock too shallow	Feasible, but not economic	Feasible, but not preferred	Preferred Alternative
			Right Shoulder (1)	N/A – Rock too shallow (1)	Feasible, but not economic (1)	Feasible, but not preferred (1)	Preferred Alternative (1)
03	416-0074.0	Tri-Chord Overhead	Median (1)	N/A – Rock too shallow (1)	Feasible, but not economic (1)	Feasible, but not preferred (1)	Preferred Alternative (1)
			Right Shoulder	N/A – Rock too shallow	Feasible, but not economic	Feasible, but not preferred	Preferred Alternative
04	416-0074.4	Tri-Chord Overhead	Median	N/A – Rock too shallow	Feasible, but not economic	Feasible, but not preferred	Preferred Alternative
			Right Shoulder	N/A – Rock too shallow	Feasible, but not economic	Feasible, but not preferred	Preferred Alternative
05	417-0131.0	Tri-Chord Overhead	Median	Preferred Alternative	Not practical due to bedrock depth	Not practical due to bedrock depth	Not practical due to bedrock depth
			Right Shoulder (1)	Preferred Alternative (1)	Not practical due to bedrock depth (1)	Not practical due to bedrock depth (1)	Not practical due to bedrock depth (1)

Notes: N/A – Not an applicable/appropriate design option.

1. Preferred caisson alternative specified based on borehole information collected at adjacent foundation location.

Table 2 – Geotechnical Design Parameters for Tri-Chord Overhead Sign Foundations

Sign Number		BH No.	Borehole Location (MTM)	Stratum	Depth[1] (m)	Elevation (m)	Ground Water Elev. (m)	Design Parameters [2]					
Golder	MTO							C_u	c'	φ'	γ	γ'	K_p
01	416-0073.2	15-01A	N 5021373.5 E 359001.5	Compact Gravelly Sand (Fill)[3] Firm to Stiff Silty Clay Stiff Silty Clay to Clay Very Loose Silty Sand Very Dense Sand (Glacial Till)	0.0 – 1.3 1.3 – 5.2 5.2 – 7.9 7.9 – 9.0 Below 9.0	81.9 – 80.6 80.6 – 76.7 76.7 – 74.0 74.0 – 72.9 Below 72.9	80.6 [4]	- 40 50 - -	- 7.5 7.5 - -	30 28 28 28 35	19 17 17 18 20	9 7 7 8 10	3.0 2.8 2.8 2.8 3.7
02	416-0073.3	15-02A	N 5021670.3 E 358866.2	Gravelly Sand (Fill)[3] Dolostone Bedrock	0.0 – 0.9 Below 0.9	75.4 – 74.5 Below 74.5	74.5 [4]	- -	- -	30 -	19 -	9 -	3.0 -
03	416-0074.0	15-03A	N 5021970.9 E 358744.7	Gravelly Sand (Fill)[3] Cobbles and Boulders (Rock Fill)[3] Sandstone Bedrock	0.0 – 1.0 1.0 – 1.4 Below 1.4	68.9 – 67.9 67.9 – 67.5 Below 67.5	67.5 [4]	- - -	- - -	30 35 -	19 22 -	9 12 -	3.0 3.7 -
04	416-0074.4	15-04A	N 5022278.7 E 358600.0	Gravelly Sand (Fill)[3] Cobbles and Boulders (Rock Fill)[3] Sandstone Bedrock	0.0 – 0.9 0.9 – 1.0 Below 1.0	67.8 – 66.9 66.9 – 66.8 Below 66.8	66.8 [4]	- - -	- - -	30 35 -	19 22 -	9 12 -	3.0 3.7 -
		15-04B	N 5022285.5 E 358610.9	Gravelly Sand (Fill)[3] Sandstone Bedrock	0.0 – 1.4 Below 1.4	67.7 – 66.3 Below 66.3	66.3 [4]	- -	- -	30 -	19 -	9 -	3.0 -
05	417-0131.0	15-05A	N 5023005.2 E 358834.4	Gravel and Sand (Fill)[3] Compact to Dense Silty Sand (Fill) Very Stiff to Stiff Silty Clay Firm to Stiff Silty Clay/Silty Sand	0.0 – 0.8 0.8 – 2.8 2.8 – 5.3 Below 5.3	68.6 – 67.8 67.8 – 65.8 65.8 – 63.3 Below 63.3	64.0 [4]	- - 75 50	- - 7.5 7.5	30 32 28 28	19 19 17 17	9 9 7 7	3.0 3.2 2.8 2.8

Notes: [See Next]

Notes:

1. Depth to bedrock is given for the borehole location; the ground surface elevation at the borehole location should be compared to the ground surface elevation at the actual sign location, and the depth to “sound” bedrock adjusted accordingly.
2. Design parameters: C_u = Undrained shear strength (kPa);
   c' = Cohesion (kPa);
   φ' = Effective friction angle (degrees);
   γ = Bulk unit weight (kN/m^3);
   γ' = Effective unit weight below the groundwater level (kN/m^3); and,
   K_p = Passive earth pressure coefficient.
3. Although the passive resistance in the upper 1.8 m is neglected to account for frost action, C_u, c', φ' and K_p parameters are given for the soil, in the event that the ground surface elevation varies significantly between the borehole and sign support locations.
4. Groundwater elevation inferred in the open borehole at the time of drilling and based on sample conditions.

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

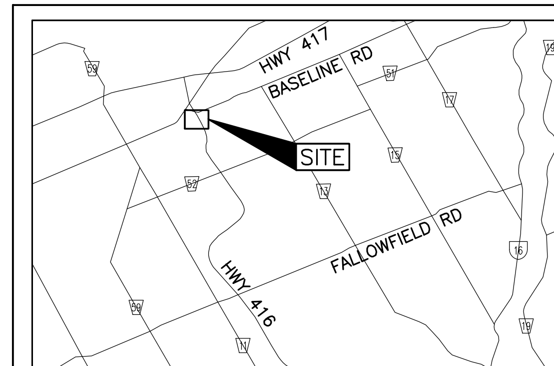
CONT No.
WP No. 4184-15-01

SHEET

TRI-CHORD OVERHEAD SIGNS

HIGHWAY 416 (NORTH) AND HIGHWAY 417 (WEST)
BOREHOLE LOCATIONS
1413191-1020-001-001

Golder Associates



KEY PLAN

SCALE 2 0 2 4 km

LEGEND

Borehole – Current Investigation

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
15-01A	81.9	5021373.5	359001.5
15-02A	75.4	5021670.3	358866.2

LICENSED PROFESSIONAL ENGINEER M. J. KENNEDY 100188946 Jan 28, 2016 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Jan 28, 2016 PROVINCE OF ONTARIO

NOTES

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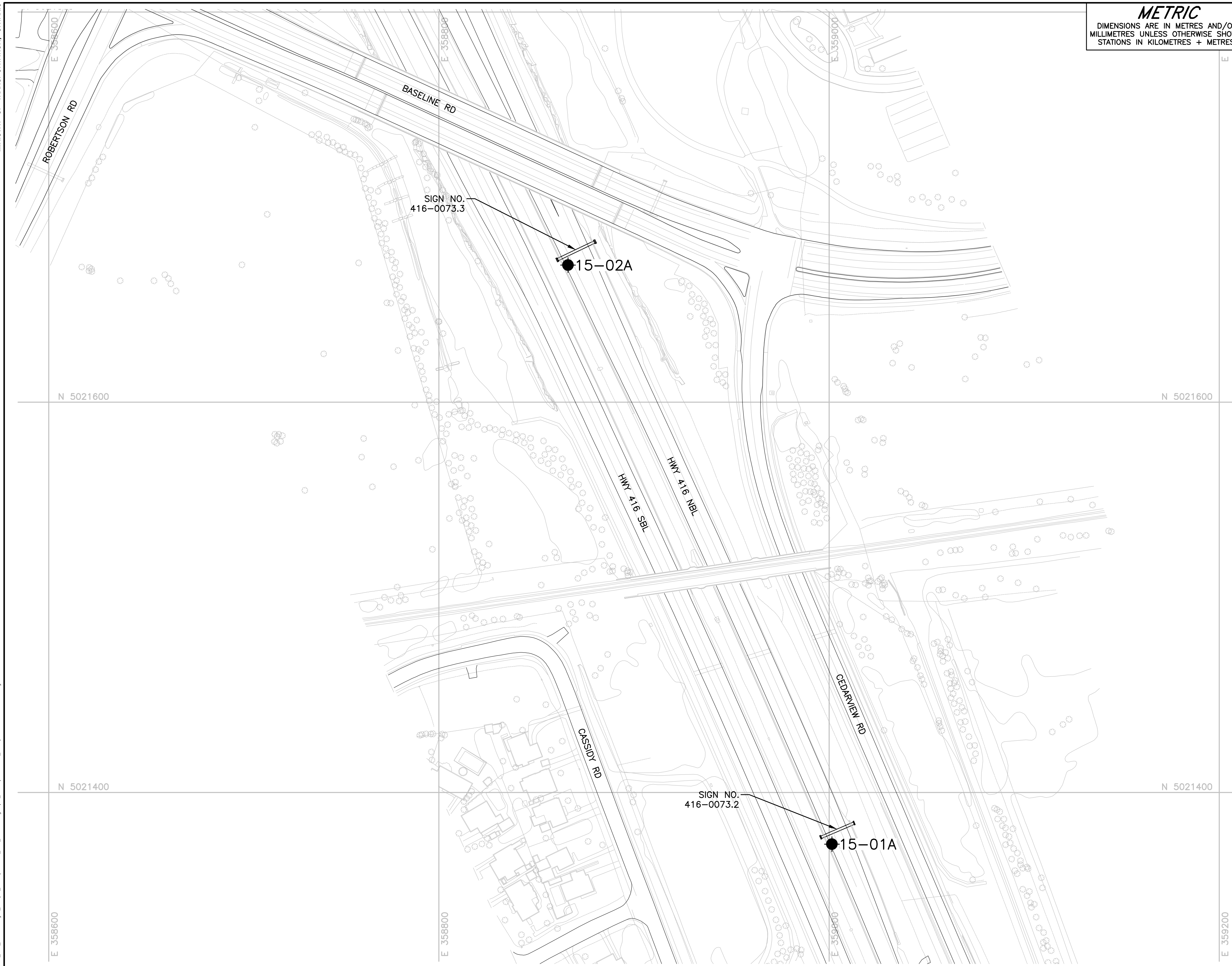
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by MTO, drawing file nos. 139929.dwg, 139930.dwg, 139931.dwg and 139932.dwg, dated Sept. 17, 2014, received Dec. 07, 2015.



PLAN

SCALE 20 0 20 40 m

NO.	DATE	BY	REVISION

Geocres No. 31G5-270

HWY. 416	PROJECT NO. 1413191		DIST. EASTERN
SUBM'D. MJK	CHKD. MJK	DATE: 01/28/2016	SITE:
DRAWN: JM	CHKD. FJH	APPD. FJH	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

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

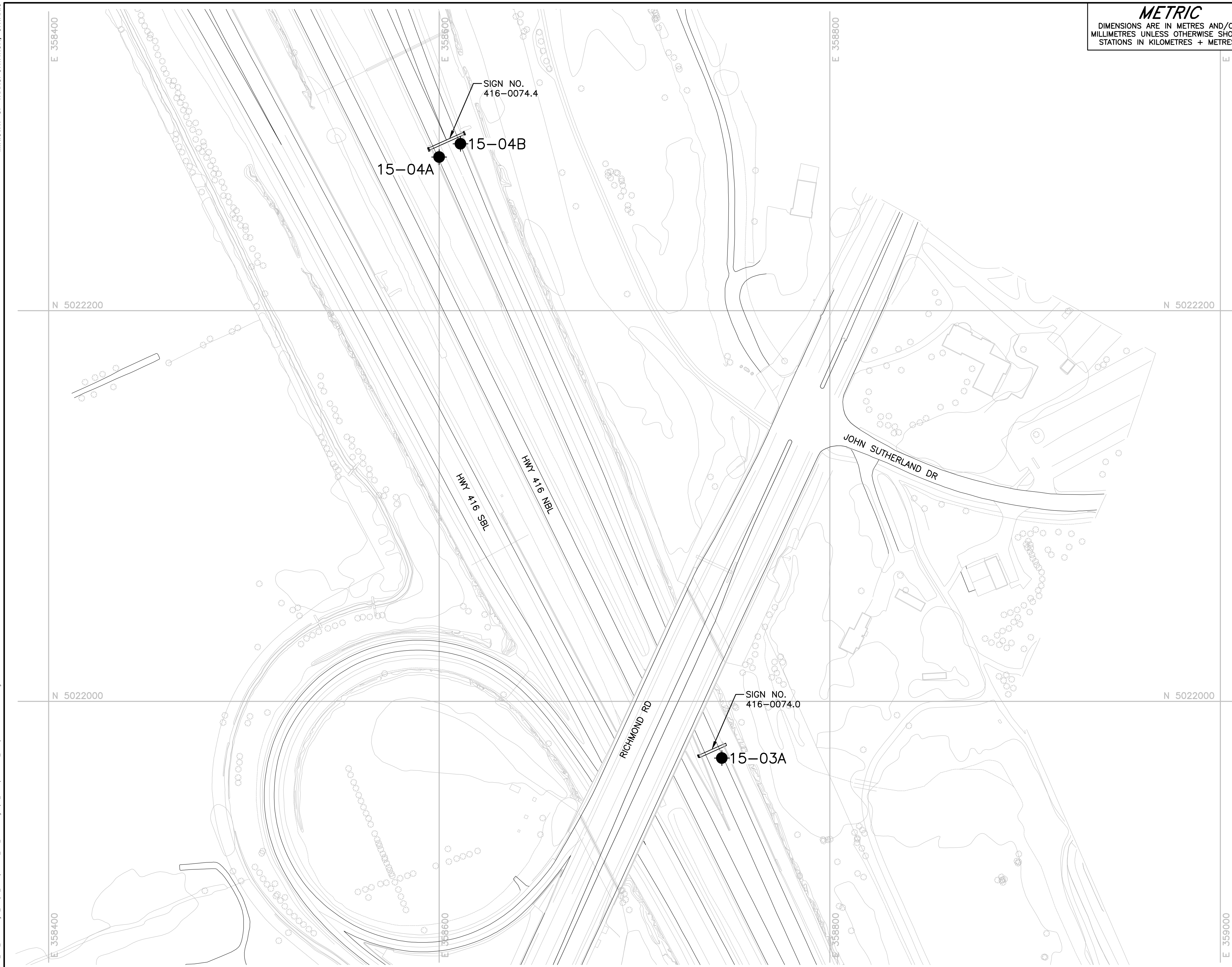
CONT No.
WP No. 4184-15-01

SHEET

TRI-CHORD OVERHEAD SIGNS

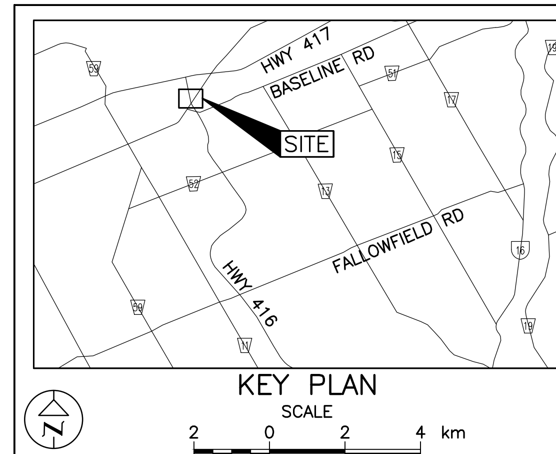
HIGHWAY 416 (NORTH) AND HIGHWAY 417 (WEST)
BOREHOLE LOCATIONS
1413191-1020-001-002

Golder Associates



PLAN

SCALE
20 0 20 40 m



KEY PLAN

SCALE
2 0 2 4 km

LEGEND

● Borehole – Current Investigation

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
15-03A	68.9	5021970.9	358744.7
15-04A	67.8	5022278.6	358600.0
15-04B	67.7	5022285.5	358610.9

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NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31G5-270

HWY. 416	PROJECT NO. 1413191		DIST. EASTERN
SUBM'D. MJK	CHKD. MJK	DATE: 01/28/2016	SITE:
DRAWN: JM	CHKD. FJH	APPD. FJH	DWG. 2

PLOT DATE: January 28, 2016

MINISTRY OF TRANSPORTATION, ONTARIO

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

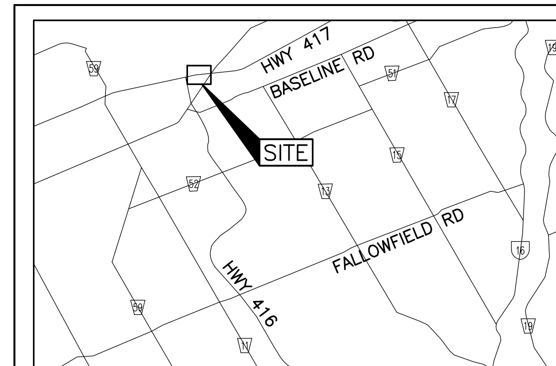
CONT No.
WP No. 4184-15-01

TRI-CHORD OVERHEAD SIGNS
HIGHWAY 416 (NORTH) AND HIGHWAY 417 (WEST)
BOREHOLE LOCATIONS
1413191-1020-001-003

SHEET

Golder Associates

PLAN
SCALE
20 0 20 40 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

Borehole – Current Investigation

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
15-05A	68.6	5023005.2	358834.4

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.	.	.	.

Geocres No. 31G5-270

HWY. 417	PROJECT NO. 1413191		DIST. EASTERN
SUBM'D. MJK	CHKD. MJK	DATE: 01/28/2016	SITE:
DRAWN: JM	CHKD. FJH	APPD. FJH	DWG. 3

PLOT DATE: January 28, 2016
FILENAME: N:\Active\Spatial_IM\MTO\416_417_OverheadSigns_TOA\99_PROJ\1413191_MTO_OverheadSigns\40_PROD\Phase1020_West\1413191-1020-003.dwg

APPENDIX A

Borehole and Drillhole Records

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
DO or DP	Seamless open-ended, driven or pushed tube samplers
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual tube sample
DD	Diamond drilling

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.) split spoon sampler for a distance of 300 mm (12 in.).

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60^0 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

Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected 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 (u) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) 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 or S_u** kPa	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 or PL	Plastic limited
w_1 or LL	Liquid limit
C	Consolidaiton (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
DS	Direct shear test
Gs	Specific gravity
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 test (LV-laboratory vane test)
γ	Unit weight

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

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
V	volume
W	weight

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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial vertical effective overburden stress
$\sigma_1 \sigma_2 \sigma_3$	principal stresses (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_1 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity Index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_1 - w) / I_p$
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 (overconsolidated range)
C_s	swelling index
C_α	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	overconsolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p or 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 or 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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of rock material 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 texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very Thickly Bedded	> 2 m
Thickly Bedded	0.6 m to 2m
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	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very Wide	> 3 m
Wide	1 – 3 m
Moderately Close	0.3 – 1 m
Close	50 – 300 mm
Very Close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

Total Core Recovery
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 varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^0 angle is horizontal.

Description and Notes
An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

BD -	Bedding	PY -	Pyrite
FO -	Foliation/Schistosity	Ca -	Calcite
CL -	Clean	PO -	Polished
SH -	Shear Plane/Zone	K -	Slickensided
VN -	Vein	SM -	Smooth
FLT -	Fault	RO -	Ridged/Rough
CO -	Contact	ST -	Stepped
JN -	Joint	PL -	Planar
FR -	Fracture	IR -	Irregular
MB -	Mechanical Break	UN -	Undulating
BR -	Broken Rock	CU -	Curved
BL -	Blast Induced	TCA -	To Core Axis
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

Golder Associates

RECORD OF BOREHOLE No 15-01A

SHEET 1 OF 1 **METRIC**

PROJECT 1413191
G.W.P. 4184-15-01
LOCATION N 5021373.5 ;E 359001.5
ORIGINATED BY HEC
DIST Eastern HWY 416
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
COMPILED BY JM
DATUM Geodetic
DATE September 30, 2015
CHECKED BY MJK

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
81.9	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
81.7 / 0.3	Gravelly sand (FILL) Grey Moist	1	SS	20	
80.6 / 1.3	SILTY CLAY Firm to stiff Grey Moist	2	SS	2	
		3	SS	2	
		4	SS	3	
		5	SS	WH	
76.7 / 5.2	SILTY CLAY to CLAY, with clayey silt interbeds Stiff Grey Moist	6	SS	1	0 10 47 43
74.0 / 7.9	Silty SAND, some gravel, trace clay (TILL) Very loose Grey Wet	7	SS	3	
		8	SS	1	
72.9 / 9.0	SAND, trace silt (TILL) Very loose Grey Wet	9	SS	55/0.20	
72.4 / 9.5	END OF BOREHOLE AUGER REFUSAL				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; 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 (%) 25 50 75; UNIT WEIGHT γ kN/m³.

Elevation scale: 81, 80, 79, 78, 77, 76, 75, 74, 73

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

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Golder Associates

RECORD OF BOREHOLE No 15-02A

SHEET 1 OF 2 **METRIC**

PROJECT 1413191
G.W.P. 4184-15-01
LOCATION N 5021670.3 ;E 358866.2
ORIGINATED BY HEC
DIST Eastern HWY 416
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core
COMPILED BY JM
DATUM Geodetic
DATE October 1, 2015
CHECKED BY MJK

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
75.4	GROUND SURFACE				
0.0	ASPHALTIC CONCRETE				
75.1 / 0.3	Gravelly SAND (FILL) Grey Moist	1	SS	50/0.13	
74.5 / 0.9	Dolostone (BEDROCK) Bedrock cored from depths of 0.9 m to 4.5 m. For bedrock coring details refer to Record of Drillhole 15-02A	1	RC	REC 100%	RQD = 84%
		2	RC	REC 92%	RQD = 87%
		3	RC	REC 100%	RQD = 59%
70.9 / 4.5	END OF BOREHOLE				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (75, 74, 73, 72, 71); 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 (%) 25, 50, 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^{3}, \times^{3}$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 1413191

RECORD OF DRILLHOLE: 15-02A

LOCATION: N 5021670.3 ;E 358866.2

DRILLING DATE: October 1, 2015

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES
		Continued from Record of Borehole 15-02A		74.49													
1	Rotary Drill NQ Core	Dolostone (BEDROCK) Fresh Thinly to thickly bedded Dark grey Fine to medium grained Non-porous Strong		0.89	1	100											
2					2	100-0											
3					3	0											
4				70.91 4.47													
		END OF DRILLHOLE															

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: MJK

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Golder Associates

RECORD OF BOREHOLE No 15-03A

SHEET 1 OF 2

METRIC

PROJECT 1413191			
G.W.P. 4184-15-01	LOCATION N 5021970.9 ;E 358744.7	ORIGINATED BY HEC
DIST Eastern HWY 416	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core	COMPILED BY JM
DATUM Geodetic	DATE October 7, 2015	CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
68.9	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE					
68.6 0.3	Gravelly sand (FILL) Grey Moist	1	SS	91/0.28	68	
67.9 1.0	Cobbles and boulders (FILL)					
67.5 1.4	Sandstone (BEDROCK) Bedrock cored from depths of 1.4 m to 4.4 m For bedrock coring details refer to Record of Drillhole 15-03A	1	RC	REC 100%	67	RQD = 77%
		2	RC	REC 100%	66	RQD = 84%
		3	RC	REC 100%	65	RQD = 33%
64.5 4.4	END OF BOREHOLE					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH 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 (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

PROJECT: 1413191

RECORD OF DRILLHOLE: 15-03A

LOCATION: N 5021970.9 ;E 358744.7

DRILLING DATE: October 7, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN
		Continued from Record of Borehole 15-03A		67.54		
2 3 4	Rotary Drill NQ Core	Sandstone (BEDROCK), with occasional nodular sections Fresh Medium to thickly bedded White to light brown Crystalline Fine to medium grained Non-porous Slightly calcareous Strong - Occasional thin (1-4 mm) calcite vein		1.40	1 2 3	100-0 0 0
				64.50		
5 6 7 8 9 10 11		END OF DRILLHOLE		4.44		

Other column headings: RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20), R.Q.D. % (80 60 40 20), FRACT. INDEX PER (5 10 15 20), DISCONTINUITY DATA (DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja), HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}), WEATH-ERING INDEX (W1 W2 W3 W4 W5 W6), FEATURES

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: HEC
CHECKED: MJK

Golder Associates

RECORD OF BOREHOLE No 15-04A

SHEET 1 OF 2 — METRIC

PROJECT 1413191 | G.W.P. 4184-15-01 | DIST Eastern | HWY 416 | DATUM Geodetic

LOCATION N 5022278.7 ;E 358600.0

BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core

DATE October 7, 2015

ORIGINATED BY HEC | COMPILED BY JM | CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
67.8	GROUND SURFACE										
0.0	ASPHALTIC CONCRETE										
67.6 0.3	Gravelly sand (FILL) Grey										
66.9	Cobbles and boulders (FILL)		1	SS	55/0.13		67				
1.0	Sandstone (BEDROCK) Bedrock cored from depths of 1.0 m to 4.3 m For bedrock coring details refer to Record of Drillhole 15-04A		1	RC	REC 92%		66				RQD = 53%
			2	RC	REC 100%		65 64				RQD = 98%
63.5			3	RC	REC 100%						RQD = 100%
4.3	END OF BOREHOLE										

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 (%) 25 50 75

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

PROJECT: 1413191

RECORD OF DRILLHOLE: 15-04A

SHEET 2 OF 2

LOCATION: N 5022278.7 ;E 358600.0

DRILLING DATE: October 7, 2015

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES
		Continued from Record of Borehole 15-04A		66.76													
	Rotary Drill NQ Core	Sandstone (BEDROCK), with occasional nodular sections and thin laminations of shale Fresh Thinly to thickly bedded Light brown to dark grey Fine to medium grained Slightly calcareous Non-porous Strong		1.04	1	100-0											
					2	0											
				63.51 4.29	3	0											
		END OF DRILLHOLE															

Depth scale marks: 2, 3, 4, 5, 6, 7, 8, 9, 10, 11

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DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: MJK

RECORD OF BOREHOLE No 15-04B

SHEET 1 OF 2 **METRIC**

PROJECT 1413191

G.W.P. 4184-15-01 LOCATION N 5022285.5 ;E 358610.9 ORIGINATED BY RI

DIST Eastern HWY 416 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core COMPILED BY JM

DATUM Geodetic DATE October 27-28, 2015 CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
67.7	GROUND SURFACE						
0.0	ASPHALTIC CONCRETE						
0.1	Gravelly sand (FILL) Grey						
66.9 / 0.8	Gravel and sand, some silt (FILL) Very dense Brown Moist	1	SS	67		67	53 34 10 3
66.3 / 1.4	Sandstone (BEDROCK) Bedrock cored from depths of 1.4 m to 5.3 m. For bedrock coring details refer to Record of Drillhole 15-04B	1	RC	REC 82%	∇	66, 65	RQD = 53%
		2	RC	REC 98%		64	RQD = 93%
		3	RC	REC 99%		63	RQD = 98%
62.4 / 5.3	END OF BOREHOLE						

NOTES:

1. Water level in open borehole at a depth of 1.9 m below ground surface (Elev. m), measured during drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; 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 (%) 25 50 75; UNIT WEIGHT γ kN/m^3

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

PROJECT: 1413191

RECORD OF DRILLHOLE: 15-04B

LOCATION: N 5022285.5 ;E 358610.9

DRILLING DATE: October 27-28, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES
		Continued from Record of Borehole 15-04B		66.31													
2, 3, 4, 5	Rotary Drill NQ Core	Sandstone (BEDROCK), with thin laminations of grey to black shale Fresh Thinly to thickly bedded Light to dark grey Fine to medium grained Non-porous Calcareous Strong - Calcareous content increasing with depth		1.37	1	100-0											
					2	0											
				62.42 5.26	3	0											
6, 7, 8, 9, 10, 11		END OF DRILLHOLE															

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: RI

CHECKED: MJK

RECORD OF BOREHOLE No 15-05A

SHEET 1 OF 2 **METRIC**

PROJECT 1413191			
G.W.P. 4184-15-01	LOCATION N 5023005.2 ;E 358834.4		ORIGINATED BY RI
DIST Eastern HWY 417	BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY JM
DATUM Geodetic	DATE October 28, 2015		CHECKED BY MJK

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
68.6	GROUND SURFACE					
0.0	ASPHALTIC CONCRETE					
0.1	Gravelly sand (FILL) Grey					
68.2 / 0.4	Sandy gravel (FILL) Grey				68	
67.8 / 0.8	Silty sand, some gravel, contains asphalt layer at 1.5 m (FILL) Compact Brown Moist	1	SS	18		11 67 (22)
		2	SS	50/0.10	67	
		3	SS	25	66	
65.8 / 2.8	SILTY CLAY to CLAY, trace to some sand, with silty sand seams Very stiff to stiff Grey-brown Moist	4	SS	16	65	
		5	SS	8		0 31 41 28
		6	SS	4	64	
63.3 / 5.3	Layered SILTY CLAY and Silty SAND Firm to stiff Grey Moist	7	SS	WH	63	0 27 42 31 / 0 59 28 13
		8	SS	WH	62	
					61	
		9	SS	PM	60	
		10	SS	1	59	

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 15-05A

SHEET 2 OF 2

METRIC

PROJECT 1413191

G.W.P. 4184-15-01 LOCATION N 5023005.2 ;E 358834.4 ORIGINATED BY RI

DIST Eastern HWY 417 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) COMPILED BY JM

DATUM Geodetic DATE October 28, 2015 CHECKED BY MJK

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
58.2 10.4	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 6.1 m below ground surface (Elev. 62.5 m), measured during drilling.							>96			

Legend: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED. DCPT / shear strength scale: 20 40 60 80 100. Water content: PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; scale 25 50 75.

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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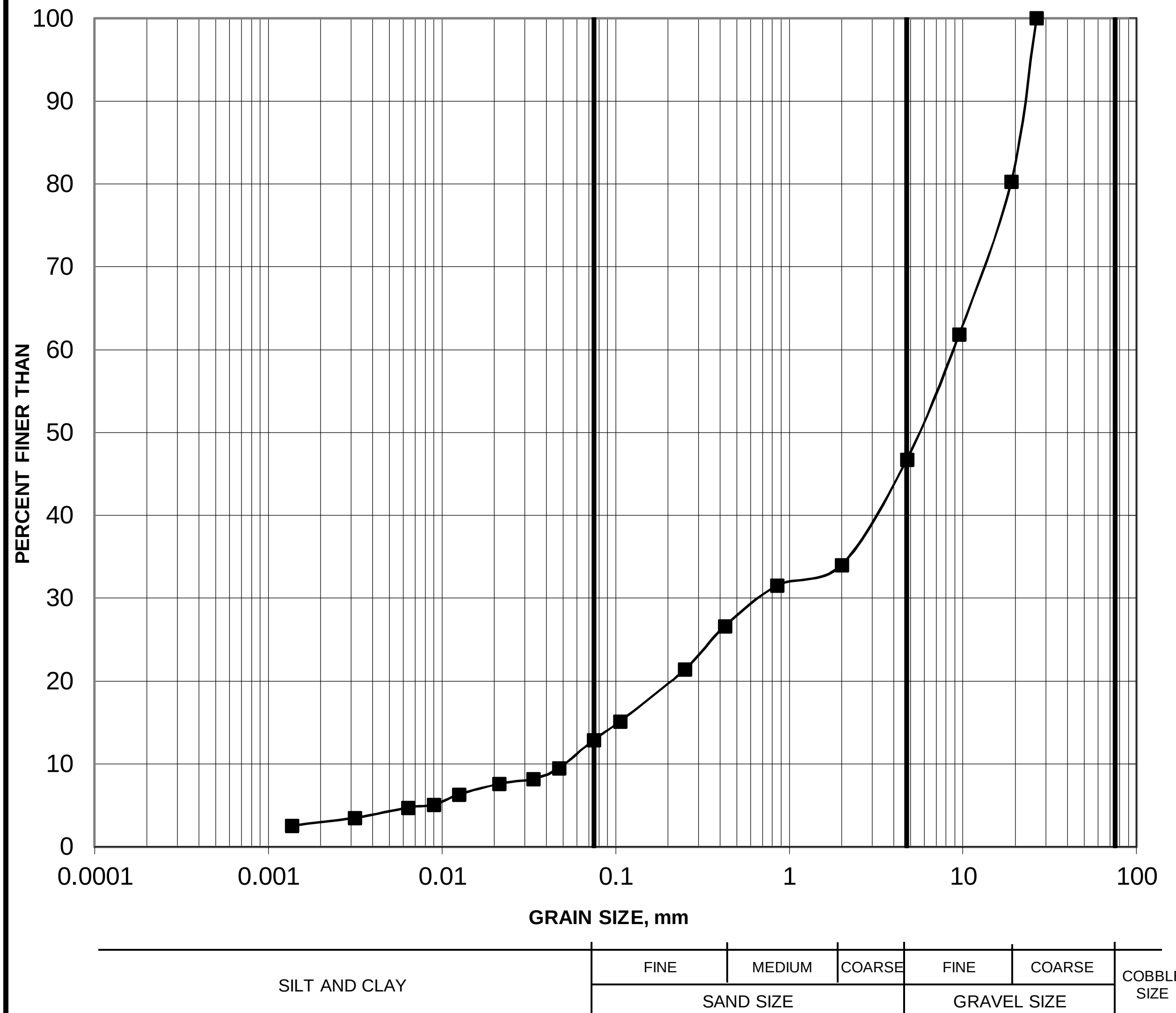
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

GRAVEL AND SAND, SOME SILT (FILL)



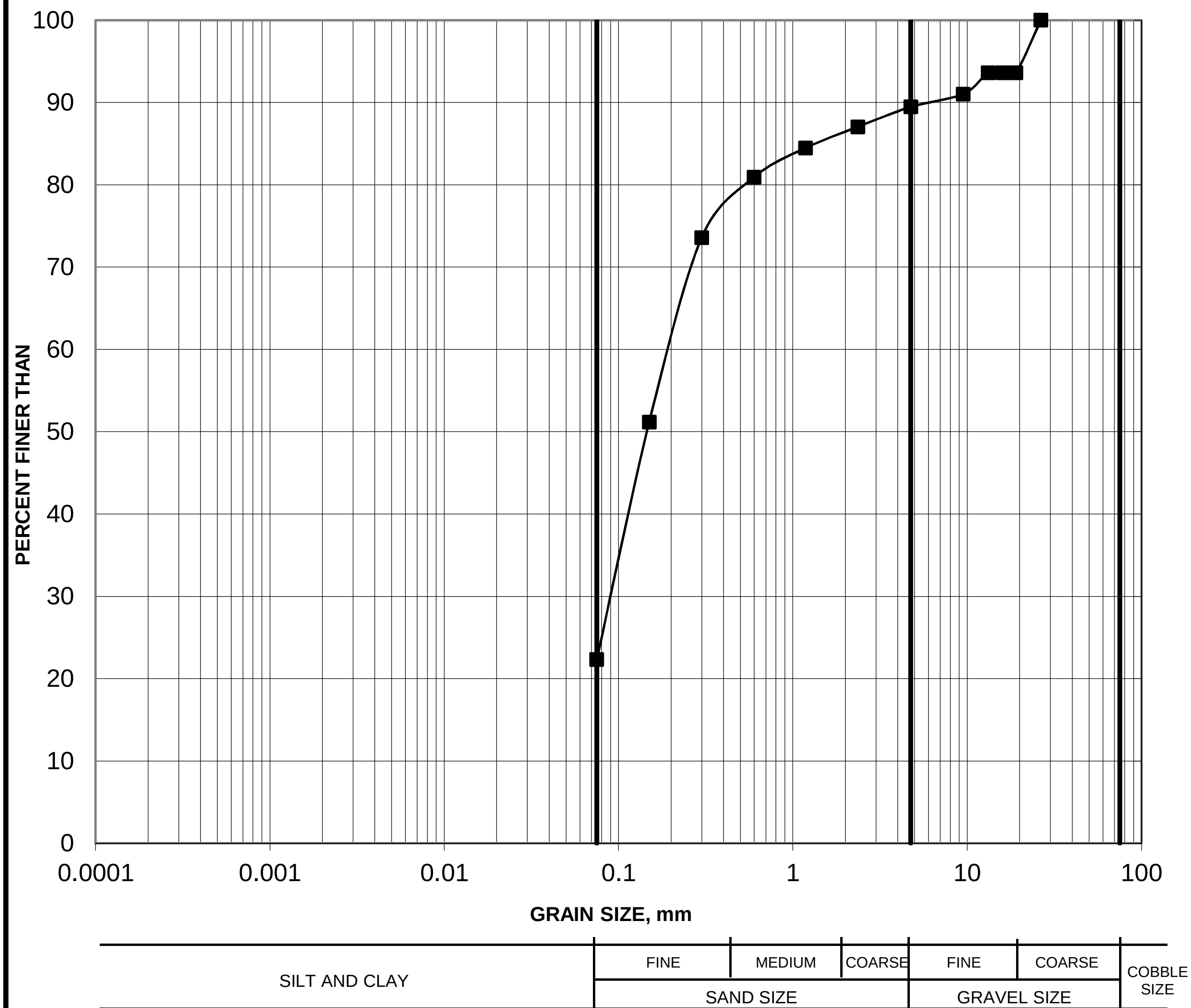
Borehole	Sample	Depth (m)
15-04B	1	0.76-1.37

Project: 1413191/1020/001 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B2

SILTY SAND, SOME GRAVEL (FILL)

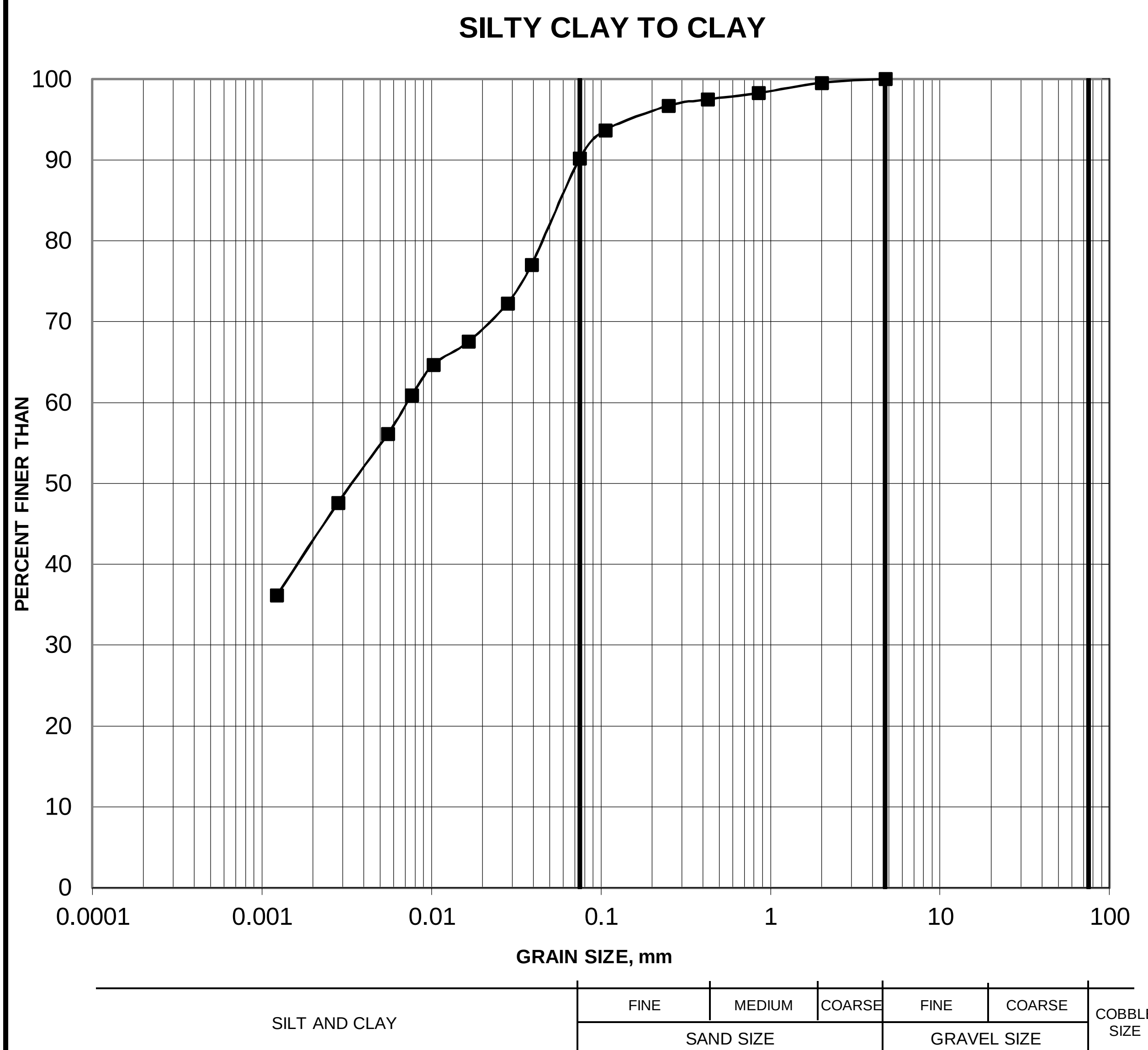


Borehole	Sample	Depth (m)
15-05A	1	0.76-1.37

Project: 1413191/1020/001 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B3

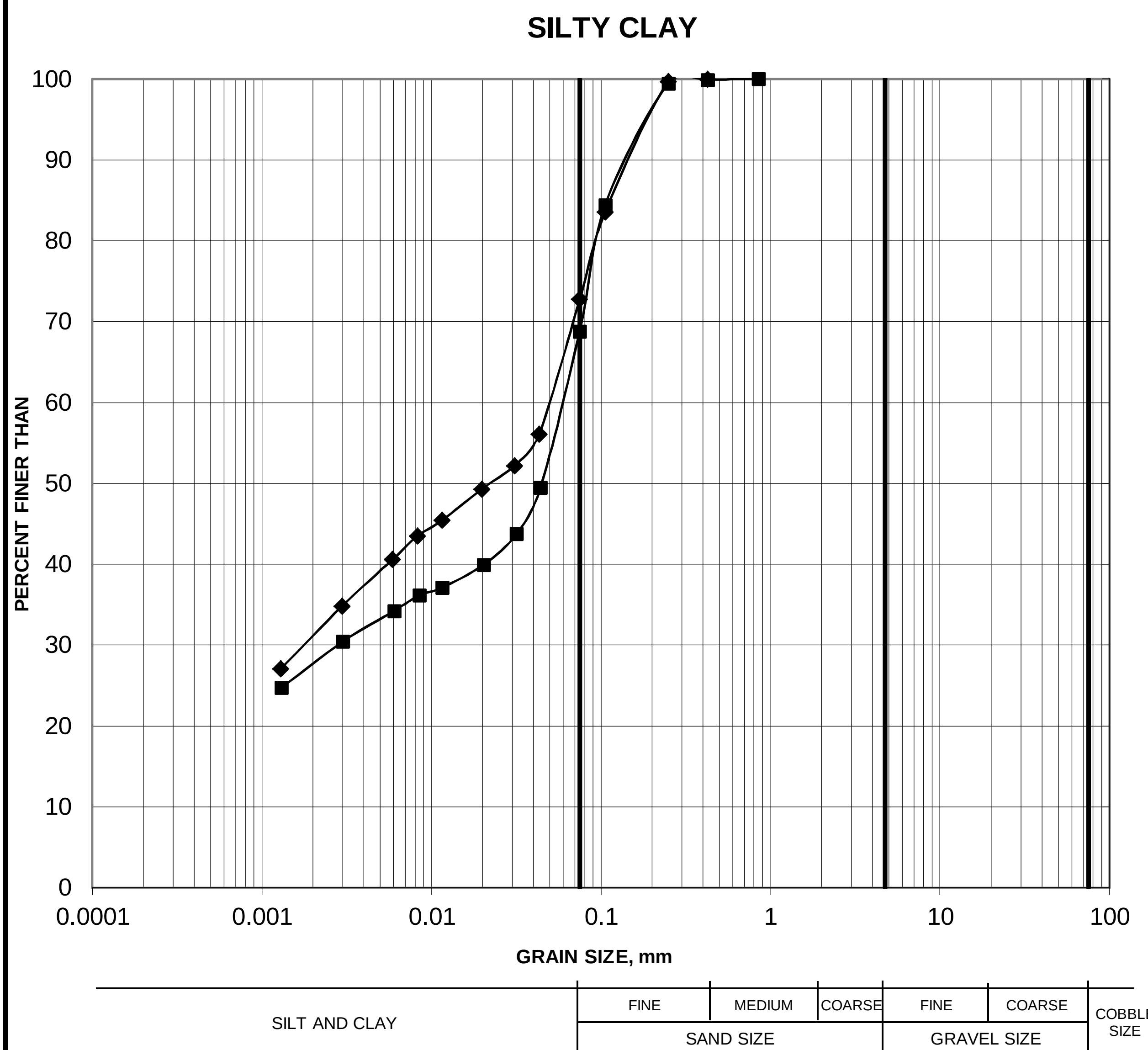


Borehole	Sample	Depth (m)
15-01A	6	6.10-6.71

Project: 1413191/1020/001 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B4



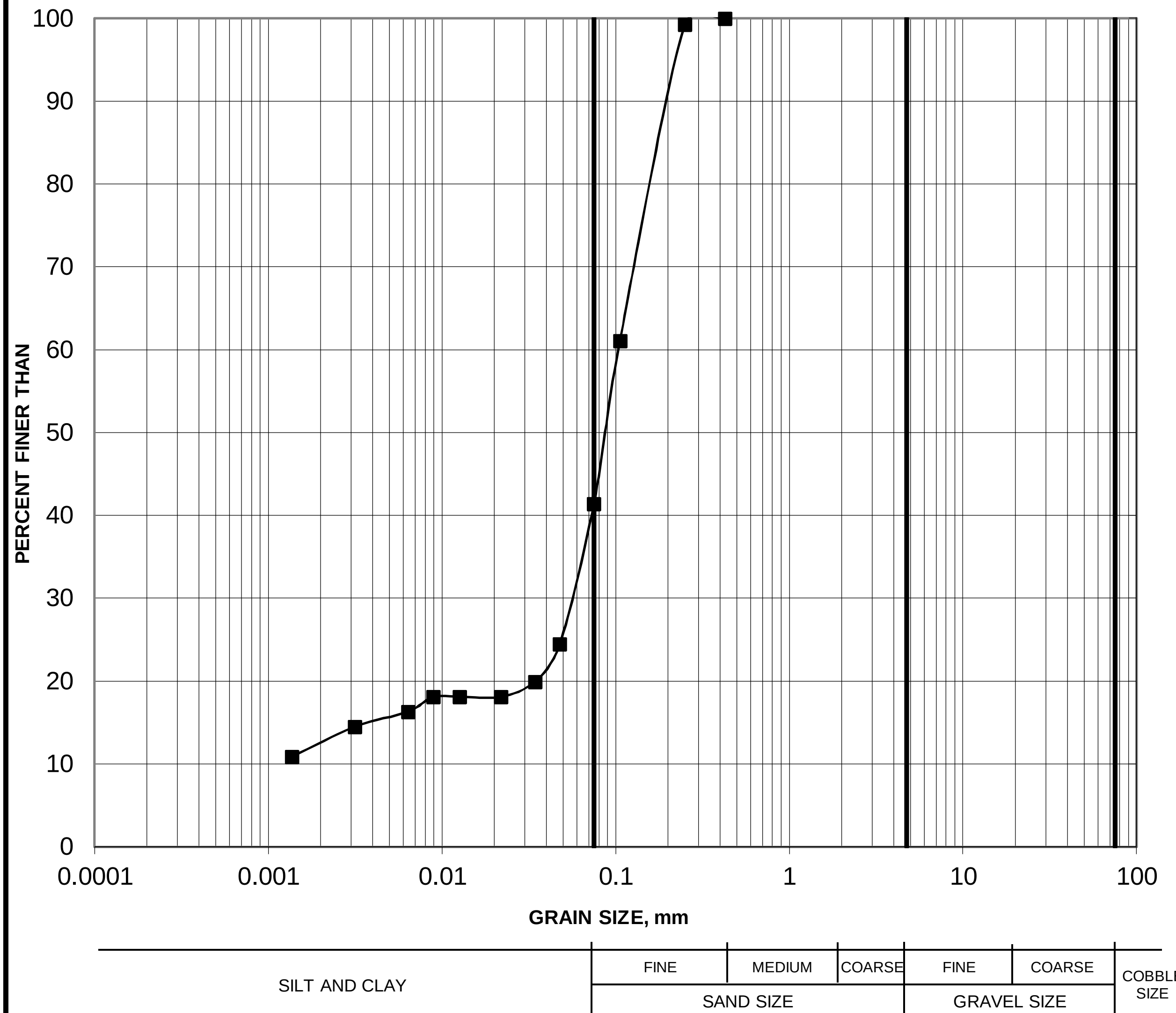
Borehole	Sample	Depth (m)
15-05A	5	3.81-4.42
15-05A	7A	5.34-5.64

Project: 1413191/1020/001 **Golder Associates**

Created by: MI
Checked by: CNM

GRAIN SIZE DISTRIBUTION	FIGURE B5

SILTY SAND, SOME CLAY

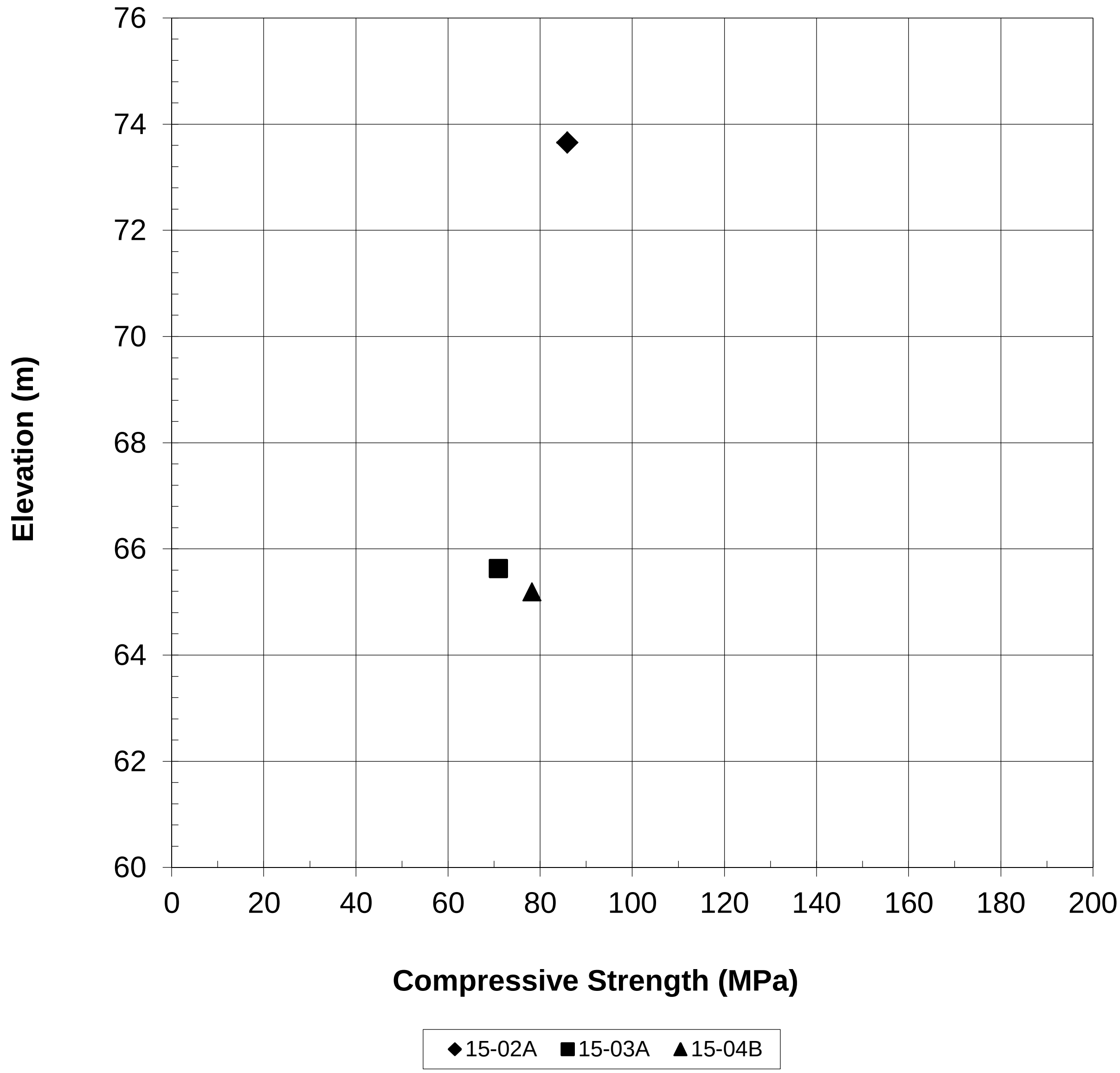


Borehole	Sample	Depth (m)
15-05A	7B	5.64-5.95

Project: 1413191/1020/001 **Golder Associates**

Created by:	MI
Checked by:	CNM

SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS	FIGURE B6



Project: 1413191/1020/001 **Golder Associates**

Created by: MJK
Checked by: CNM

APPENDIX C

Non Standard Special Provisions

CAISSON FOUNDATIONS FOR SIGN SUPPORTS

Non Standard Special Provision

SCOPE

Where OPSS 903 is called up by OPSS.PROV 915, OPSS 903 is amended by the following. Where conflict occurs, this NSSP shall take precedence.

The Contractor shall construct sign support foundations in conformance with the design and at the locations indicated in the Contract Documents.

The Contractor shall construct the sign support foundations against undisturbed bases and sides of excavations. The bases of caisson excavations shall be cleaned of loosened and/or softened materials prior to pouring concrete for the foundation. The construction methods and techniques shall be the responsibility of the Contactor, but consideration could be given to using temporary liners or tremie concreting techniques where conditions warrant.

The Contractor is advised that variable subsurface conditions may be encountered at sign support caisson locations. For bidding purposes, the Contractor shall assume that the overburden has zones of non-cohesive soil and contains cobbles and boulders, and that the groundwater levels are near the surface. The Contractor is advised that non-cohesive soil is susceptible to disturbance under conditions of unbalanced hydrostatic head. As a lower priority than the above-noted instruction, the Contractor shall assume that the subsurface conditions at sign support caisson locations are generally similar to the closest of the boreholes, as illustrated in the Foundation Investigation Report.

Pre-augering/pre-coring for some caissons for the sign support foundations will extend into dolostone or sandstone bedrock, which is classified as strong. Appropriate construction procedures and equipment will be required to penetrate 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

DOWELS INTO ROCK

Non Standard Special Provision

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS.PROV 904, including all special provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the structure footings.

1.2 Instructions to Contractor

1.2.1 These instructions are to be read in conjunction with the Contract Drawings.

1.2.2 A total of 1 test Dowels into Rock are required for the Dowels into Rock at each structure footing.

1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.

1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years of experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years of experience in all aspects associated with the installation of Dowels into Rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
- All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
- Test results verifying the 28 day strength of non-shrink grout.
- The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- The procedures to verify hole length. Records of measurements that verify the hole length.
- Records of all drilling procedures, rock conditions encountered, and installation times.
- Test procedures for Dowels into Rock.
- Drawings and design calculations for a suitable reaction system for the applied test loads.
- Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- Drawings and details for reference system arrangement.
- Current calibration curves shall be provided for all gauges.
- Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

1.7.1 Soils, rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.15.35 non-shrink grout.

The Contractor shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout,
- installation procedures

3.0 EQUIPMENT

3.1 General

3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that includes the above noted records.

4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.

4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3.1 Grout

4.3.2 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

4.3.3 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

4.3.4 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the each structure location.

5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.

5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator.

5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days of notice in writing of this operation.

5.3 Testing Equipment

5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.

5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

- The beams shall be independently supported with the support firmly embedded in the ground.
- The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
- Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at each structure location.

Tests for Dowels into Rock shall have a capacity of at least **[insert value]** kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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