



April 16, 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

**DEN-LOU ROAD UNDERPASS STRUCTURE - SITE 46-566
HIGHWAY 17 FOUR LANING EXTENSION FROM 20.5 KM
WEST OF HIGHWAY 144, EASTERLY FOR 6.5 KM
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 156-98-00, WP 5593-09-01**

Submitted to:

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REPORT





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 INVESTIGATION PROCEDURES	1
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1 Regional Geology	3
4.2 General Overview of Local Subsurface Conditions	4
4.2.1 Fill/Topsoil.....	4
4.2.2 Silty Clay.....	4
4.2.3 Silt to Silt and Sand.....	5
4.2.4 Sand to Gravelly Sand	5
4.2.5 Bedrock/Refusal.....	6
4.2.6 Groundwater Conditions	7
5.0 CLOSURE	7

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	9
6.1 General.....	9
6.2 Foundations.....	9
6.2.1 Abutments: Steel H-Pile Foundations	9
6.2.1.1 Geotechnical Axial Resistances	10
6.2.1.2 Set Criteria and Pile Driving Note	10
6.2.1.3 Resistance to Lateral Loads	11
6.2.1.4 Frost Protection	13
6.2.2 Pier: Spread Footing on Bedrock	13
6.2.2.1 Geotechnical Resistance	13
6.2.2.2 Resistance to Lateral Loads	14
6.2.2.3 Frost Protection	14
6.3 Seismic Considerations	14
6.4 Lateral Earth Pressures	14



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

6.5	Embankment Stability	15
6.6	Embankment Settlement	16
6.6.1	Settlement of Embankment Fill	17
6.6.2	Settlement Performance Requirements	18
6.6.3	Results of Analysis.....	19
6.7	Construction Considerations.....	20
6.7.1	Subgrade Preparation and Embankment Construction	20
6.7.1	Control of Groundwater and Surface Water and Protection of Cut Slopes	20
6.7.2	Obstructions.....	21
7.0	CLOSURE	21

REFERENCES

TABLES

Table 1	Evaluation of Foundation Alternatives – Abutments
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DRAWINGS

Drawing 1	Site Location Plan
Drawing 2	Borehole Locations and Soil Strata
Drawing 3	Soil Strata

FIGURES

Figure 1	Stability Analysis – North Abutment Front Slope
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APPENDICES

Appendix A **Record of Boreholes and Drillholes**

Lists of Symbols and Abbreviations	
Record of Boreholes	DL-1 to DL-6
Record of Drillholes	DL-1 to DL-3

Appendix B **Record of Boreholes – PML (2007)**

Record of Boreholes	DLR-1 to DLR-5
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Appendix C **Laboratory Test Results – Current Investigation**

Figure C1	Plasticity Chart – Silty Clay
Figure C2	Grain Size Distribution – Silty Clay
Figure C3	Grain Size Distribution – Silt to Silt and Sand
Figure C4	Grain Size Distribution – Sand to Gravelly Sand

Appendix D **Non-Standard Special Provisions**

NSSP	CSP for Integral Abutment
NSSP	H-Piles
NSSP	Mass Concrete
NSSP	Dowels into Rock
NSSP	Obstructions



PART A

FOUNDATION INVESTIGATION REPORT

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WEST OF HIGHWAY 144, EASTERLY 6.5 KM

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Ltd. (DMW) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering for the Den-Lou Road Underpass structure crossing over the proposed Highway 17 Eastbound Lanes (EBL) and Westbound Lanes (WBL). The proposed structure is part of the Highway 17 new interchange and extension of the existing four-lane section and highway at the West Junction of Sudbury Municipal Road 55, from 20.5 km west of Highway 144, easterly for 6.5 km. The general location of the Highway 17 four-laning extension is shown on the Site Location Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal, dated March 2011. Golder's proposal for foundation engineering services is contained in Section 6.8 of DMW's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated November 11, 2011.

This report addresses the investigation carried out for the Den-Lou Road Underpass structure and the associated approach embankments only. The foundation investigations for the high fill embankments, interchange structure at Municipal Road 55 and culverts for this project are presented in separate reports.

Preliminary subsurface information for this project is available and was supplied by MTO, in the reports and subsequent appendices titled:

- *Planning, Preliminary Design, and Environmental Supplementary Report, Highway 17, Town of Walden, GWP 156-98-00, dated March 2009, by Stantec Consulting Limited; and*
- *Preliminary Foundation Investigation and Design Report for Den-Lou Road Underpass, Highway 17, Town of Walden, District of Sudbury, GWP 156-98-00, Index No: 075FIDR, PML Ref: 05TF059F1, dated July 4, 2008, by Peto MacCallum Ltd. (PML).*

2.0 SITE DESCRIPTION

The proposed Den-Lou Road Underpass structure is located along the existing Den-Lou Road in the City of Greater Sudbury and the Township of Denison. The proposed structure will cross the new Highway 17 four-lane extension, which is parallel to and approximately 300 m south of the existing Highway 17 alignment.

In general, the topography of the structure site is an elevated area of rolling terrain with sparsely populated treed areas. The land use in the general area includes residential with scattered rural farm use. The ground surface at the borehole locations advanced at the proposed structure site ranges from about Elevations 265 m to 267 m along the centreline of the structure.

3.0 INVESTIGATION PROCEDURES

The investigation for the Detail Design of the proposed Den-Lou Road Underpass was carried out between December 10 and 18, 2013, and March 6 to 10, 2014, during which time a total of six boreholes (DL-1 to DL-6) were advanced at the locations of the proposed structure foundation elements and approach embankments as



FOUNDATION REPORT DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

shown on Drawing 2. The Record of Borehole and Drillhole sheets for this bridge structure are presented in Appendix A.

The current investigation was supplemented by the five (5) boreholes advanced by PML in April 2007 (Boreholes DLR-1 to DLR-5) as part of the preliminary investigation at the structure site design for the project. The locations of the PML boreholes drilled in the vicinity of the bridge are shown on Drawing 2 and the Record of Boreholes are provided in Appendix B.

The boreholes for the current field investigation were carried out using a CME-55 track and buggy drill rig supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario. The boreholes were advanced using 108 mm inner diameter hollow-stem augers, NW casing using wash boring techniques and NQ size core barrel for bedrock coring. Soil samples were generally obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sample operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils). All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes during the current investigation were advanced to depths up to 16.2 m below existing ground surface including between 3.0 m and 3.1 m of bedrock coring in Boreholes DL-1 to DL-3.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A. A standpipe piezometer was installed in Borehole DL-3 to permit monitoring of the groundwater level. The piezometer consists of a 50 mm diameter polyvinyl chloride pipe, with a 1.5 m long slotted screen, sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer was partially backfilled with cuttings (due to cave), then backfilled to near surface within bentonite and asphalt to ground surface. The piezometer installation details and water level readings are indicated on the Record of Borehole sheets contained in Appendix A.

The field work for the current investigation was observed by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and examined and cared for the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Geotechnical Laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected representative samples. The results of the laboratory testing for the current investigation are shown in Appendix C. Strength testing, such as uniaxial compression strength (UCS), was carried out on selected specimens of the rock core.

The foundation elements were staked in the field by DMW's surveyors in the winter of 2013 prior to drilling. Borehole locations, in station and offsets, were measured in reference to the locations staked by the surveyors and were subsequently converted to northing and easting coordinates in AutoCAD. Borehole elevations were surveyed by a member of our technical staff in reference to the ground surface elevations at the foundation elements. The borehole/drillhole locations shown on Drawing 2 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole/drillhole locations and ground surface elevations are as follows:



FOUNDATION REPORT DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
DL-1	5 136 717.0	276 293.9	265.4	16.0
DL-2	5 136 678.0	276 293.4	266.2	8.6
DL-3	5 136 636.0	276 293.2	267.3	16.2
DL-4	5 136 720.0	276 298.5	265.3	13.9
DL-5	5 136 678.0	276 299.1	266.1	7.1
DL-6	5 136 638.9	276 297.4	267.2	8.2 Borehole 12.4 DCPT

The supplemental boreholes drilled by PML as part of the preliminary investigation are also shown on Drawing 2 and were positioned based on the northing and easting coordinates and ground surface elevations as follows:

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
DLR-1	5 136 617	276 294	267.2	7.7
DLR-2	5 136 637	276 291	266.8	16.0
DLR-3	5 136 678	276 292	265.7	8.5
DLR-4	5 136 719	276 291	264.7	15.8
DLR-5	5 136 738	276 294	265.5	9.8

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in the NOEGTS¹ Mapping, the subsurface conditions in this section of Highway 17 are comprised of bedrock knobs, outcrops and ridges with undulating to rolling glaciolacustrine plain, alluvial plain and organic terrain deposits. In the lower-lying glaciolacustrine plain and alluvial plain areas the primary material consists of wet silts, sands and clays, while within the organic terrain deposit the primary material consists of peat. The drainage in the area could be considered dry to wet, with moderate to low relief.

Based on geological mapping by the Ministry of Natural Resources (Map 2542)², the site is underlain by rocks of the Paleoproterozoic Era belonging to the Huronian Supergroup and Elliot Lake Group consisting of conglomerate, wacke, arkose, quartz arenite, argillite, limestone and dolostone. Areas of mafic and related intrusive rocks comprised of diabase sills, dykes and related granophyre are also present in the vicinity of the site. Based on geological mapping by the Ontario Department of Mines (Map 2170)³ this area is characterized by extensive faults from distinct time periods. The Murray Fault has been identified to run parallel to the approximate proposed alignment of Highway 17.

¹ Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Digital Map Reference Number 411SW.

² Ministry of Natural Resources. Bedrock Geology of Ontario – West Central Sheet, Ontario Geological Survey - Map 2542

³ Ontario Department of Mines (1969). Sudbury Mining Area, Sudbury District, Map 2170.



4.2 General Overview of Local Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the borings advanced during this investigation together with the results of the laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets and Drillhole sheets attached in Appendix A. The results of the in situ tests (i.e., SPT 'N'-values) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The Record of Borehole sheets for the boreholes from PML are attached in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling processes and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy as encountered in the boreholes and drillholes is shown in profile/sections on Drawings 2 and 3.

In general, the subsurface conditions in the underpass structure area consist of a surficial layer of topsoil or fill underlain by deposits of silty clay to clayey silt, silt to silt and sand, and sand to gravelly sand overlying bedrock. A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill/Topsoil

In Boreholes DL-1 to DL-4, DLR-1 and DLR-5, asphalt or surface treated rock chip and tar was encountered at ground surface with the thickness of these materials ranging from 50 mm to 100 mm.

In Borehole DLR-3, a 150 mm thick layer of topsoil fill was encountered from ground surface. In Borehole DLR-2, a 200 mm thick deposit of topsoil was encountered from ground surface.

Below the asphalt or rock chip and tar in Boreholes DL-1 to DL-4, DLR-1 and DLR-5, underlying the topsoil in Borehole DLR-3 and from ground surface in Boreholes DL-5, DL-6 and DLR-4, fill material consisting of gravelly sand to sand and gravel, trace to some silt was encountered. The fill ranges from 0.1 m to 0.9 m in thickness. In Borehole DLR-5, at a depth of 0.9 m, clayey silt and organic inclusions were present in a 0.3 m thick layer of sand and gravel fill.

Two SPT 'N'-values measured within the fill in Borehole DLR-5 are 5 blows and 25 blows per 0.3 m of penetration indicating a loose to compact relative density.

The natural water content measured on samples of the fill ranges from about 5 per cent to 22 per cent.

4.2.2 Silty Clay

A deposit of silty clay was encountered underlying the fill in Boreholes DL-1, DL-2, DL-4, DL-5 and DLR-1, and DLR-3 to DLR-5 and underlying the topsoil in Borehole DLR-2. The thickness of the deposit ranges from 0.9 m to 2.4 m and the surface of the deposit was encountered from Elevation 266.9 m to 264.3 m.

SPT 'N'-values measured within this deposit generally ranges from 4 blows to 26 blows per 0.3 m of penetration suggesting a firm to very stiff consistency, with one 'N'-value of 55 blows in Borehole DL-4, likely due to the material being frozen.



Atterberg limits testing carried out on four samples of the silty clay from the current investigation and three samples of the silty clay to clayey silt from the previous investigation, indicates liquid limits ranging from about 27 per cent to 45 per cent, plastic limits ranging from about 20 per cent to 23 per cent and plasticity indices calculated to be from 7 per cent to 23 per cent. The results of the Atterberg limits testing carried out on the four samples from the current investigation are shown on the plasticity chart on Figure C1 in Appendix C, and indicate that the material is a silty clay of intermediate plasticity. Results from the previous investigation indicate the material is clayey silt of low plasticity to silty clay of intermediate plasticity as shown on the respective borehole logs in Appendix B.

The results of grain size distributions of two samples of the silty clay from the current investigation are shown on Figure C2 in Appendix C. Three grain size distribution tests of the silty clay were carried out in the previous investigation and the results are shown on the respective borehole logs in Appendix B.

The natural water content measured on samples of the silty clay ranges from about 21 per cent to 30 per cent.

4.2.3 Silt to Silt and Sand

Silt to silt and sand was encountered underlying either the fill or the silty clay deposit in all boreholes. The thickness of the deposit ranged from 1.6 m to 4.1 m and the surface of the deposit was encountered from Elevationd 266.9 m to 262.8 m. Cobbles and/or boulders were encountered in Boreholes DL-5 and DLR-3 at about Elevationd 262.4 m and 261.1 m, respectively. Borehole DL-5 was terminated in this deposit on auger refusal after exploring the deposit for 4.1 m.

SPT 'N'-values measured within the silt to silt and sand deposit ranged from 14 blows to 24 blows per 0.3 m of penetration, indicating a compact relative density, with one 'N'-value of 65 blows in Borehole DL-6, likely due to the material being frozen.

The grain size distributions of six samples of the silt to silt and sand from the current investigation and the results are shown on Figure C3 in Appendix C. Four grain size distribution tests of the silt to silt and sand were carried out in the previous investigation and the results are shown on the respective borehole logs in Appendix B.

Two Atterberg limits tests from the previous investigation on this deposit yielded non-plastic results as shown on the respective borehole logs in Appendix B.

The natural water content measured on samples of the silt to silt and sand ranges from about 5 per cent to 25 per cent.

4.2.4 Sand to Gravelly Sand

A deposit of sand to gravelly sand was encountered underlying the silt to silt and sand deposit in all the boreholes except Boreholes DL-2, DL-5 and DLR-3. The thickness of the deposit ranges from 7.0 m to 10.2 m where it was fully penetrated and the surface of the deposit was encountered between Elevations 264.3 m and 259.5 m. Boreholes DL-4, DL-6, DLR-1 and DLR-5 were terminated within this deposit after exploring the deposit to depths between 3.7 m and 9.4 m below the surface of the deposit, with Boreholes DL-4 and DLR-1 terminated due to auger refusal. Cobbles and/or boulders ranging in estimated diameter from 90 mm to 300 mm are inferred to be present in the deposit based on:



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

- Borehole DL-1: augers grinding in at 9.9 m depth;
- Boreholes DL-3: augers grinding at 5.9 m depth and the recovery of cobbles ranging from 90 mm to 125 mm in the NW casing/NQ core barrel at various depths below 6.5 m;
- Borehole DL-6: augers grinding between 6.4 m and 6.7 m depth; and
- Boreholes DLR-2 and DLR-4: notes on the Record of Borehole logs including 300 mm boulders in Borehole DLR-2.

SPT 'N'-values measured within the silt to silt and sand deposit range from 9 blows to 55 blows per 0.3 m of penetration, indicating a loose to very dense relative density, however majority are in the compact range.

The results of grain size distributions of four samples of the sand to gravelly sand from the current investigation are shown on Figure C4 in Appendix C. Three grain size distribution tests of this deposit were carried out in the previous investigation and the results are shown on the respective borehole logs in Appendix B.

The natural water content measured on samples of the silt to silt and sand ranges from about 2 per cent to 21 per cent.

4.2.5 Bedrock/Refusal

Bedrock was encountered below the silt in Boreholes DL-2 and DLR-3 and below the sand to gravelly sand in Boreholes DL-1, DL-3, DLR-2 and DLR-4 and was cored for lengths between 3.0 m and 3.2 m. The depth to bedrock/refusal below ground surface and corresponding bedrock surface elevations (inferred or actual) are summarized below.

Foundation Element	Borehole No.	Depth to Bedrock Surface/Refusal (m)	Bedrock Surface/Refusal Elevation (m)	Comments
South Approach	DLR-1	7.7	259.5	Split-spoon refusal
South Abutment	DL-3	13.2	254.1	Bedrock cored
	DLR-2	12.9	253.9	Bedrock cored
	DL-6	12.4	254.8	DCPT refusal
Center Pier	DL-2	5.5	260.7	Bedrock cored
	DLR-3	5.3	260.4	Bedrock cored
	DL-5	7.1	259.0	Auger refusal
North Abutment	DL-1	12.9	252.5	Bedrock cored
	DLR-4	12.2	252.5	Bedrock cored
	DL-4	13.9	251.4	Auger refusal
North Approach	DLR-5	N/A	N/A	No refusal

The recovered bedrock core is generally described as grey, very fine grained, fresh, argillite. The Total Core Recovery measured on the core samples is 100 per cent in the current investigation and between 93 per cent and 100 per cent in the previous investigation (except at the bottom of the core in Borehole DLR-3, which was 40 per cent). The Rock Quality Designation is generally between 67 per cent and 100 per cent in both the



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

current and previous investigations (except at the bottom of the core in Borehole DLR-3, which was measured to be 0 per cent), generally indicating a rock mass of fair to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006).

Laboratory UCS testing was carried out on three selected samples of the recovered bedrock core from the current investigation. The UCS values are presented on the Record of Drillhole sheets in Appendix A and are summarized below. The UCS values indicate that the bedrock is strong to very strong (R4 to R5, 50 MPa < UCS < 250 MPa) in accordance with Table 3.5 of CFEM (2006).

Borehole	Elevation (m)	UCS (MPa)
DL-1	249.8	60
DL-2	259.7	129
DL-3	252.5	125

4.2.6 Groundwater Conditions

For the current investigation, the overburden samples taken in the boreholes were moist to wet. Boreholes DL-2 and DL-5 were noted to be dry upon completion of drilling and the water level observed in the remaining boreholes upon completion of drilling during the current investigation varied between Elevations 259.4 m and 255.8 m, corresponding to depths between 7.9 m and 9.6 m below ground surface.

A standpipe piezometer was installed in Borehole DL-3 to allow monitoring of the groundwater level at the site. Details of the piezometer installation are shown on the Record of Borehole sheets in Appendix A. The groundwater level measured in the piezometer installation is summarized below.

Foundation Element	Borehole No.	Ground Surface Elevation (m)	Groundwater Elevation (m)	Date of Measurement
North Approach	DL-3	267.3	259.3	April 29, 2014

From the previous investigation, the groundwater levels were measured upon completion of drilling in Boreholes DLR-1 and DLR-2 at depths of 6.7 m and 5.5 m, corresponding to Elevations 260.5 m and 261.3 m, respectively. The groundwater level was observed during drilling in Boreholes DLR-3, DLR-4 and DLR-5 between 2.0 m and 8.4 m below ground surface (between Elevation 263.7 m and 256.3 m).

The groundwater levels in the area are subject to fluctuations seasonally and following precipitation events, and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

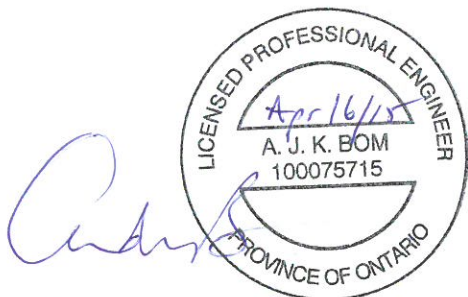
The field drilling program for the current investigation was supervised by Mr. Ed Savard. This report was prepared by Mr. Adam Core, E.I.T. and the technical aspects were reviewed by André Bom, P.Eng., a geotechnical engineer with Golder. Mr. Jorge Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.



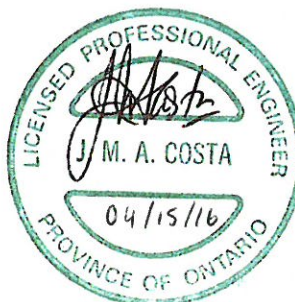
Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

DEN-LOU ROAD UNDERPASS STRUCTURE – SITE 46-566

HIGHWAY 17 FOUR LANING EXTENSION FROM 20.5 KM

WEST OF HIGHWAY 144, EASTERLY 6.5 KM

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GWP 156-98-00, WP 5593-09-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the subsurface investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

Based on the General Arrangement (GA) drawing provided by DMW, the proposed 81 m long two-span structure will consist of a 39 m long south span and a 42 m long north span. The existing ground surface varies between about Elevations 267.3 m and Elevation 264.7 m at the borehole locations, generally decreasing from south to north, and the proposed grade at the south and north abutments will be at Elevations 268.7 m and Elevation 267.1 m, resulting in new south and north approach embankments about 1 m and 3 m above the existing ground surface, respectively. The proposed Highway 17 EBL and WBL will be constructed in a cut about 6 m below existing ground surface with a grade at about Elevation 260 m.

It is anticipated that traffic will be rerouted from Den-Lou Road to other roads in the area to facilitate the construction of the new bridge and the new Highway 17 four-lane embankment cuts at this site.

6.2 Foundations

Due to the presence of the approximately 10.5 m to 12.5 m thick deposit of compact native silts and sands encountered at the abutments and shallow depth to bedrock encountered at the pier, deep foundations consisting of steel H-piles driven to bedrock are recommended for support of the abutment and a spread footing founded on bedrock is recommended at the pier.

Spread footings founded on the compact silt/sand deposit at the abutments are not recommended due to the potential for differential settlement between the footings at the abutments on compact soils and the footing at the pier founded on bedrock.

A summary of the advantages and disadvantages, relative costs and risks/consequences for the abutment foundations is provided in Table 1 following the text of this report. Table 1 also includes comparison of 610 mm diameter drilled steel casings socketted into bedrock as an alternative to steel H-piles driven to bedrock at the abutment. However, as steel H-piles are much more practical at this site compared to drilled steel casings, this alternative is not discussed further in the report. Discussion and design recommendations for the steel H-piles at the abutments and spread footings at the pier are given in the sections below.

6.2.1 Abutments: Steel H-Pile Foundations

The estimated pile lengths given below are based on the underside of pile cap elevations shown on the GA drawing. The tip elevations correspond to the estimated termination depth of the piles at the bedrock/refusal



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

surface. There should be a provision made in the Contract for dealing with varying pile lengths due to the variability in depth to the bedrock surface and the lengths given below should be considered minimum lengths.

Foundation Element	Borehole Numbers	Proposed Underside of Pile Cap Elevation (m)	Bedrock/Refusal Elevation (m)	Estimated Approx. Pile Length (m)
South Abutment	DL-3, DL-6, DLR-2	262.7	253.9	9
North Abutment	DL-1, DL-4, DLR-4	263.3	251.4*	12

Note: *Auger refusal was encountered in Borehole DL-4 at Elevation 251.4 m

The surficial silty clay soils will be excavated for pile cap construction at the south and north abutments. As the foundation soils along the pile below the pile cap are non-cohesive (sands, silts) and compact to very dense in relative density, downdrag loads need not be considered for foundation design.

Where integral abutment design includes the installation of 3 m long corrugated steel pipe (CSP) liners, the annular space between the pile and the liner need to be backfilled with uniform grained, loose sand to allow for lateral movement/flex within the CSP. An example NSSP for the construction of an integral abutment incorporating an upper CSP section is presented in Appendix D.

6.2.1.1 Geotechnical Axial Resistances

For HP310X110 piles driven to bedrock, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be used for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

Due to the presence of the cobbles and boulders within the cohesionless soils at the site, consideration should be given to using the heavier pile section HP310X132. A factored geotechnical axial resistance at ULS of 2,300 kN may be used for design for this larger pile section and ULS will govern the design.

6.2.1.2 Set Criteria and Pile Driving Note

Pile installation should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) 903 (Deep Foundations). The piles should be fitted with driving shoes to minimize damage to the pile tip during driving in accordance with OPSD 3000.100 (Steel H-Pile Driving Shoe). Given the presence of cobbles and boulders within the sand deposit and potential for damage to the pile tip during driving along with the chance that it could cause the piles to “hang up” or be deflected from their intended vertical alignment, consideration should be given to using the heavier pile section (HP310X132).

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The criteria need to



be set to also avoid overdriving and possibly damaging the pile. Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules, but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced to 75 per cent and the pile should be re-driven in 2 sets of 10 blows to improve the process of seating the pile on the sloping bedrock surface.
- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

A Non-Standard Special Provision (NSSP), which outlines the above criteria for seating the piles on bedrock, should be included in the Contract and an example is included in Appendix D.

The pile driving note that should be added to the drawings for this project is Note 5 in Clause 3.3.3 of the MTO's Structural Manual (MTO 2008), as follows:

- "Piles to be driven to bedrock."

6.2.1.3 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could be resisted fully or partially by the use of battered piles.

It is understood that an integral abutment foundation design is being considered with CSP liners. The upper portion of the H-piles will generally be free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance for the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The following equation (CFEM, 1992 as referenced in the Canadian Highway Bridge Design Code [CHBDC] Commentary, 2006) may be used to calculate values of k_h (kPa/m) for non-cohesive soils. The value should be reduced by a factor of 0.75 to account for sloping ground surrounding the casing, where applicable.

$$k_h = \frac{n_h z}{B}$$

where:

n_h	=	constant of subgrade reaction (kPa/m)
z	=	depth (m)
B	=	pile diameter or width (m)

The lateral resistance of the steel piles should be developed primarily from the passive resistance of the soil. The values of n_h (Terzaghi, 1955) to be incorporated into the calculations of the coefficient of horizontal



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

subgrade reaction (k_n) within the native subsoils to be utilized for the structural lateral analysis of the piles at this site are summarized below.

Foundation Element (Relevant Boreholes)	Soil Unit	Elevation (m)	n_h (kPa/m)
South Abutment (DL-3, DL-6 and DLR-2)	Loose Sand within CSP	262.7 to 259.7	2,200
	Dense to Very dense Sand to Gravelly Sand	259.7 to bedrock	11,000
North Abutment (DL-1, DL-4 and DLR-4)	Loose Sand within CSP	263.3 to 260.3	2,200
	Compact Sand	260.3 to 254.0	4,400
	Dense to Very Dense Sand	254.0 to bedrock	11,000

For a single HP 310x110 or HP310X132 vertical pile, the estimated factored lateral resistances at ULS as well as the estimated lateral reactions at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below for both the south and north abutment. These values are based on analysis carried out using the commercially available program LPILE Plus (Version 7.0), developed by Ensoft Inc.

Pile Type	Lateral Resistance/Reaction (kN)	
	ULS (Factored)	SLS (10 mm of deflection)
HP310X110	170	60
HP310X132	180	70

The lateral resistances given above are based on an assumed fixed-head condition of 1,000 kN unfactored axial load applied at the top of the pile for both HP310X110 and HP310X132 piles. The lateral resistance should be reviewed if greater vertical loads are anticipated.

Group action for lateral loading should also be considered when the pile spacing in the direction of loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction Unified Facilities Criteria (U.S. Navy, NAVFAC 1986) in the direction of loading by a reduction factor, R, as follows:

Pile Spacing in Direction of Loading (d = pile diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacing in between those listed above.

Reduction for group effects is negligible when the centre to centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.



6.2.1.4 Frost Protection

All pile caps should be provided with a minimum of 2.0 m of soil cover for frost protection as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario).

6.2.2 Pier: Spread Footing on Bedrock

The anticipated founding elevation for the pier spread footing on bedrock is summarized below.

Location Along Pier	Reference Borehole	Bedrock Surface Elevation (m)
Centre	DL-2	260.7
West	DLR-3	260.4
East	DL-5	259.0*

Note: * Auger refusal encountered at Elevation 259.0 m in Borehole DL-5, likely indicative of close proximity to the bedrock surface.

A NSSP should be included in the Contract Documents for mass concrete to raise the founding level of the footing to the level of the highest bedrock surface elevation due to the variations in the bedrock surface; an example is provided in Appendix D. Furthermore, following excavations of the overburden soils, it will be necessary to clean, scale and remove all loose, shattered and/or fractured rock within the area of the footing to ensure a proper bond to the bedrock. A provision should be included in the Contract Documents to address the requirements for field inspection, as per OPSS 902 (Excavating and Backfilling – Structures). In order to carry out this inspection, the excavation should be dry.

6.2.2.1 Geotechnical Resistance

Spread footings placed on the surface of the properly prepared bedrock may be designed based on a factored geotechnical axial resistance at ULS of 10,000 kPa. The geotechnical reaction at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

The geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.2 and 6.7.4 of the CHBDC (2006) and its commentary.

All loose, softened or disturbed subgrade soil and rock should be removed immediately prior to placement of concrete. Construction and inspection of footings should be carried out in accordance with OPSS 902 (Excavating and Backfilling – Structures).

For a footing placed on mass concrete, the factored geotechnical axial resistance at ULS, as given above assumes that the strength of the concrete used to form the pad has an unconfined compression strength of at least 25 MPa.



6.2.2.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between mass concrete and the bedrock at the pier should be calculated in accordance with Section 6.7.5 of the CHBDC. A coefficient of friction, $\tan \delta$, of 0.70 may be used for the interface between the concrete and bedrock. This value is unfactored.

If necessary, the sliding resistance between the concrete footing and/or mass concrete and the bedrock at the pier can be supplemented by dowelling into the bedrock. Construction should be performed “in-the-dry”. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is as strong or stronger than concrete, the design of the dowels into the rock may be handled in the same way as the dowel embedment into the concrete for UCS of the grout similar to that of the concrete. The dowels should have a minimum length within the sound bedrock of 1 m, and the structural strength of the dowels and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels, such as the example provided in Appendix D. These values assume that construction is carried out in dry conditions.

6.2.2.3 Frost Protection

At the pier, as the footings will be founded directly on the bedrock, frost protection is not required.

6.3 Seismic Considerations

Based on the latitude and longitude of the site (46.3691° N and 81.3705° W), the peak horizontal acceleration is equal to 0.045 g at the bedrock level at the site based on the information obtained from the NRCan (2014) website for a probability of exceedance of 10 per cent in 50 years. According to Table 4.1 of the CHBDC, this site is located in Seismic Performance Zone 1 and the corresponding site-specific zonal acceleration ratio, A_s , is 0. Given this assessment, and in accordance with Section 4.4.5.1 of the CHBDC, no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.4 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of walls for this site. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II, but with less than 5 per cent passing the No. 200 sieve, should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in



accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) or OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), as applicable.

- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.0 m behind the back of the wall (in accordance with Figure C6.20 (a) of the Commentary to the CHBDC). For unrestrained walls, granular fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m ³	0.43	0.27
Granular 'B' Type II	21 kN/m ³	0.43	0.27

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the foundation design of the structure. If the wall support and superstructure does not allow lateral yielding, at-rest earth pressures should be assumed for foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.

6.5 Embankment Stability

The stability analysis is focused at the Den-Lou Road north approach embankment front slope where the proposed embankment will be up to about 9 m high relative to the proposed Highway 17 WBL, which will be located within a cut extending to about 6 m below existing ground surface. The side slopes of the north approach will be about 3 m above existing ground surface.

The analyses assume that fill and organic soils will be removed from below the footprint of the new embankments. Further, as discussed in Section 6.2.1, the upper relatively thin surficial deposit of silty clay will be removed for abutment construction; as such, we recommend that this layer also be removed from below the footprint of the new embankment section within 20 m of the abutments. The analyses assume that the new



embankments will be constructed of granular fill at the bridge approaches; the use of rock fill has also been assessed, however, it should not be used directly behind the abutments.

The geometry of the proposed north embankment and existing ground surface included in the analyses are based on the information provided by DMW. The piezometric conditions required in the stability analyses are based on the groundwater level as encountered during the subsurface investigation.

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to assess if the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

For the new granular fill and the cohesionless deposits, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the in-situ SPT 'N'-values. The correlations proposed by Terzaghi and Peck (1967) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed works areas.

Soil Deposit	Bulk Unit Weight (kN/m³)	Effective Friction Angle (°)
New Granular B Type I or Type II Fill	21	35°
Rock Fill, as an alternative to Granular fill	19	40°
Silt, Compact	19	30°
Sand, Compact to Very Dense	20	32°

The analysis indicates that the new north approach embankment constructed of Granular B Type I or II or rock fill will have a FoS greater than 1.3 against global instability, as shown on Figure 1 for the case of Granular B embankment fill constructed at a slope inclination of not steeper than 2 Horizontal to 1 Vertical (2H:1V), assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials will be carried out. If rockfill is used the embankment side slopes and front slopes may be constructed at an inclination of 1.25H:1V.

6.6 Embankment Settlement

Settlement of the south and north approach embankments can be expected as a result of the loading from the new fills, and replacement fill after sub-excavation of the clayey silt to silty clay deposit, on the cohesionless foundation soils at this site. In addition, if rock fill is used for embankment construction, settlement due to



compression of the new rock fill itself will also occur. Settlement of granular fill that is properly placed and compacted would occur during construction.

To estimate the magnitude of the expected immediate settlements of the native cohesionless soils, analyses were carried out using hand calculations. The immediate compression of the cohesionless deposits was modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The simplified stratigraphy together with the associated strengths and unit weights employed for the different foundation soil types at the south and north abutments, at Boreholes DL-3 and Boreholes DL-1, respectively, are summarized below.

Soil Type	Approximate Thickness	γ (kN/m ³)	E (MPa)
Silt, Compact	2.4 m (South Approach) 3.3 m (North Approach)	19	20
Sand, Compact	3.1 m (South Approach, to Elevation 261.2 m) 5.6 m (North Approach, to Elevation 254.0 m)	20	25

6.6.1 Settlement of Embankment Fill

Where rock fill is used for the construction of the proposed embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e., compacted versus dumped rock fill) as outlined in "MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates" (MTO, September 2010).

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with OPSS.PROV 206 (Grading). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and, reduce settlements and should be used to construct rock fill



embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Height of Rock Fill, H	Short-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one year following the completion of embankment construction to full height.

Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Total Height of Rock Fill, H	Long-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one year following the completion of construction over the life of the embankment.

6.6.2 Settlement Performance Requirements

The settlement performance criterion for design of high fill embankments is in accordance with MTO Foundations Guideline, "Embankment Settlement Criteria for Design" (MTO, July 2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site (MTO, July 2010).



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

The total settlement and differential settlement rate are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for the embankment in the vicinity of the culvert extension and the high fill area.

6.6.3 Results of Analysis

The estimated settlement of the foundation soils and embankment rock fill, including backfilling in areas of sub-excavation of the clayey silt to silty clay stratum as a result of the embankment construction in the area of the approaches is presented in the table below. Rock fill, if used, has a thickness of up to about 5 m at the north approach. Further, recommendations for settlement mitigation, as applicable, are also presented.

Location of Embankment	Estimated Immediate Settlement of Foundation Soils (mm)	Estimated Post-Construction Settlement of Granular Fill (mm)	Estimated Post-Construction Settlement of Rock Fill (mm)	Recommended Settlement Mitigation
South Approach (Sub-excavation = 1.4 m; grade raise = 0.8 m)	< 10	~0	15	■ Not required for granular fill or rock fill embankments
North Approach (Sub-excavation = 2.4 m; grade raise = 3.3 m)	30	~0	50	■ Mitigation not required for granular fill embankment ■ Rock fill embankment must be preloaded for three months to achieve post-construction settlement less than 25 mm to meet MTO settlement criteria

A time period of three months for preload of the north approach embankment should be included in the construction schedule and a note should be included on the GA regarding the preload period.



6.7 Construction Considerations

6.7.1 Subgrade Preparation and Embankment Construction

For the bridge approach embankments, removal of the fill, organic soils and clayey silt to silty clay deposit is recommended prior to construction of the new approach embankments. Based on the borehole information, sub-excavation at the south and north approach embankments is required to a depth of up to 1.4 m and 2.4 m, respectively. All softened/loosened soils should be stripped from below the approach embankment, prior to placement of new fill.

Fill for construction of the new embankments should consist of a Granular 'B' Type I or Type II meeting the specifications of OPSS.PROV 1010 (Aggregates) or rock fill. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes should be no steeper than 2H:1V in granular fill and 1.25H:1V in rock fill.

All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

The native cohesionless soils at this site may be classified as Type 3 soil, while the silty clay soils along with any organic soils may be classified as Type 4 soils when referencing the Occupational Health and Safety Act (OHSA 2006) and Regulation for Construction Projects. All excavations must be carried out in accordance with the latest edition of the OHSA (2006) and Regulation for Construction Projects and good construction practices.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments (unless rock fill is used). If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting) and OPSS.PROV 1004 (Aggregates – Miscellaneous) to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.7.1 Control of Groundwater and Surface Water and Protection of Cut Slopes

At the abutments and pier, as the proposed underside of pile cap/footing is at/or above the proposed grade of the Highway 17 at this site, and given the depth to the groundwater level(s) measured in the boreholes, unwatering may not be required. Diversion of water away from the site may be required depending on the water level at the time of construction.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations. Seepage from the granular fills should be expected, particularly after precipitation events. It is anticipated that minor surface water seepage and seepage from the granular fills can be controlled by using properly filtered sumps within the excavation. Further, depending on the timing of pile cap construction and the cut for the new Highway 17, temporary and likely localized drainage of groundwater into the cut should be expected.

In order to achieve adequate long-term drainage of the front embankment cut slopes and to minimize surficial sloughing, a granular blanket sheeting should be provided on the slope as per OPSS 511 (Rip-Rap, Rock



Protection and Granular Sheeting). The granular sheeting material should be as per OPSS.PROV 1004 (Aggregates - Miscellaneous). The granular sheeting should be used on the permanent cut slopes formed within the native soils below the existing ground surface and should be connected to toe drains/interceptor ditches that are adequately drained. The granular sheeting should be a minimum of 600 mm thick. We recommend that a non-woven geotextile (i.e., Terrafix 270R or equivalent) be placed on the native soil slope prior to the granular sheeting being placed to minimize migration of the fines into the gravel sheeting and out through the slope.

6.7.2 Obstructions

The soils at this site are very dense in some locations and contain coarse gravel, cobbles and boulders as noted in the Record of Borehole sheets, which could affect the installation of deep foundations. An NSSP should be included in the Contract Documents to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils, an example of which is included in Appendix D.

7.0 CLOSURE

This report was prepared by Mr. Adam Core, E.I.T. and Mr. André Bom, P.Eng. Mr. Jorge Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, conducted an independent quality control review of the report.



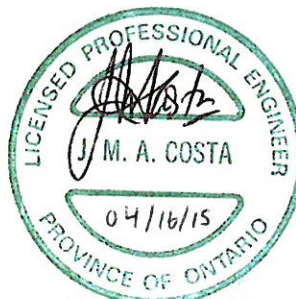
FOUNDATION REPORT
DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

Report Signature Page

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- ASTM International
- ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils
- Commercial Software
- GeoStudio (Version 7.23) by Geo-Slope International Ltd.
- LPILE (Version 7.0) by Ensoft Inc.
- Ministry of Transportation, Ontario
- MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.
- MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates, September, 2010.



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

Backfill of Structures Adjacent to Rock Embankment Approaches, Northern Region Directive, November, 2002.

Ontario Occupational Health and Safety Act

Ontario Regulation 213/91 Construction Projects

Ontario Provincial Standard Drawings

OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls Abutment, Backfill Rock
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip Rap, Rock Protection and Granular Sheeting
OPSS 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act

Ontario Regulation 903/90 Wells: O.Reg. 468/10 Amendment to Ontario Regulation 903



FOUNDATION REPORT

DEN-LOU ROAD UNDERPASS, GWP 156-98-00, WP 5593-09-01

Table 1: Evaluation of Foundation Alternatives – Abutments

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Steel H-piles driven to bedrock	1	<ul style="list-style-type: none"> ■ Conventional construction. ■ Higher axial resistance compared to spread footings. ■ Suitable for integral abutment design. ■ No issues associated with groundwater conditions. 	<ul style="list-style-type: none"> ■ Potential for “hanging-up” on cobbles/boulders within cohesionless deposits. ■ Requires up to 5m deep excavation to allow for installation of CSP for integral abutments and for pile cap construction. 	<ul style="list-style-type: none"> ■ Typically higher cost than shallow foundations. 	<ul style="list-style-type: none"> ■ Potential for “hanging-up” on cobbles/boulders within cohesionless deposits. ■ Low risk of not achieving design resistance at design pile tip elevation as larger pile section can be adopted.
Drilled Steel Casings (610 mm diameter) socketted into bedrock using Down-the-Hole hammer drilling	2	<ul style="list-style-type: none"> ■ Higher axial resistance compared to steel H-piles, fewer units required. ■ Cap can be constructed at the underside of the bridge eliminating need for sub-excavation. 	<ul style="list-style-type: none"> ■ Requires specialized drilling equipment. ■ Likely not suitable for integral abutment design given the compact to dense relative density of the subsurface deposits. ■ MTO does not normally accept drilled casings for integral abutment design. ■ Longer foundation section due to need to socket into bedrock. 	<ul style="list-style-type: none"> ■ Mobilization of specialized equipment relatively expensive. 	<ul style="list-style-type: none"> ■ Not suitable for integral abutment design at this site.
Spread Footings on compact silt to silt and sand	NR	<ul style="list-style-type: none"> ■ Conventional construction. ■ No issues associated with groundwater conditions. 	<ul style="list-style-type: none"> ■ Much lower geotechnical axial resistances than for deep foundations. ■ Excessive differential settlements between abutments founded on native soils and pier founded on bedrock due to bridge and embankment loading. ■ Not suitable for integral abutment design. ■ Requires sub-excavation to below frost penetration depth for footing foundation. 	<ul style="list-style-type: none"> ■ Typically lower cost than deep foundations. 	<ul style="list-style-type: none"> ■ Potential for differential settlements between foundation elements.

NR: Not Recommended

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

CONT No.
GWP No. 156-98-00

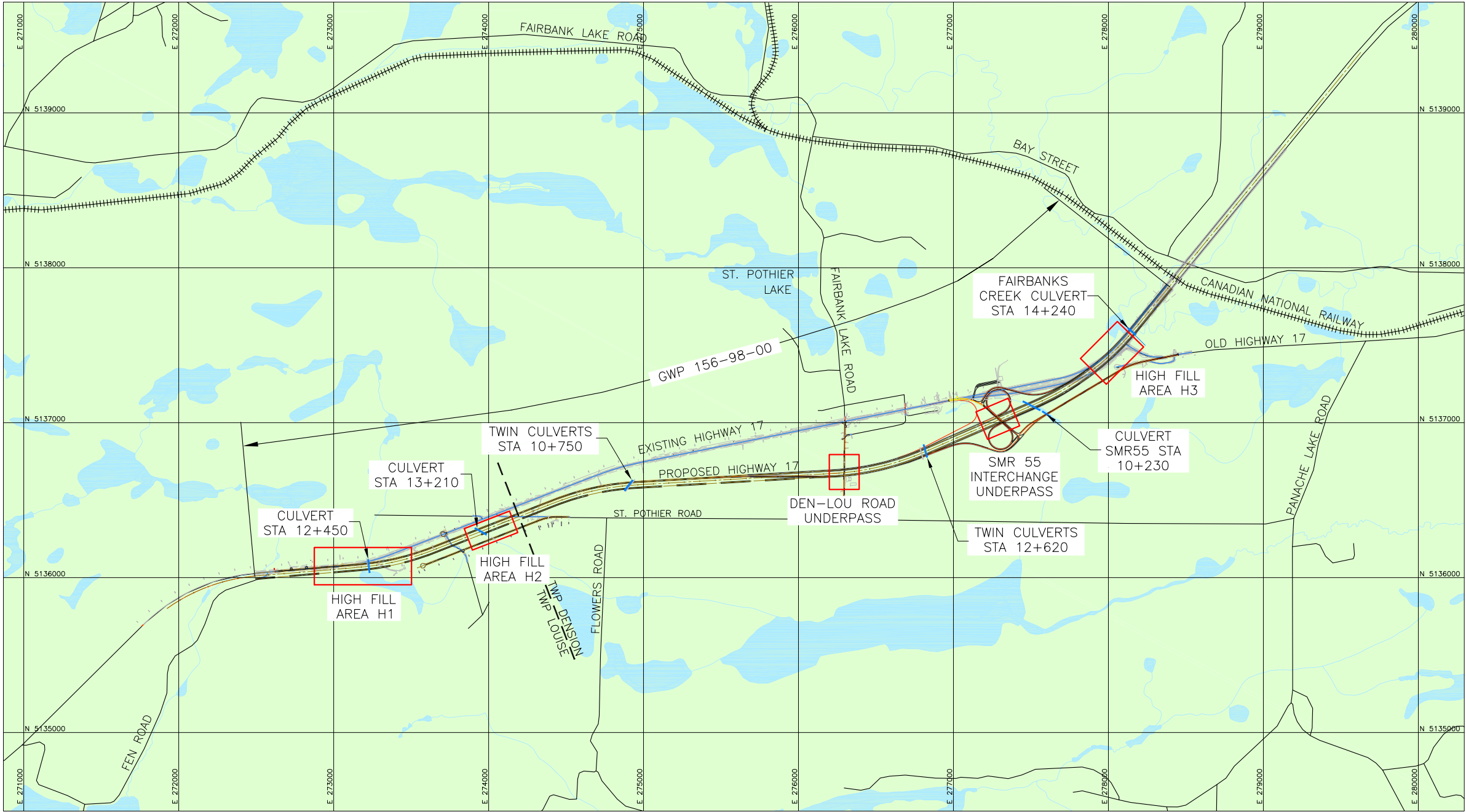


HIGHWAY 17
SITE LOCATION PLAN

SHEET



Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA



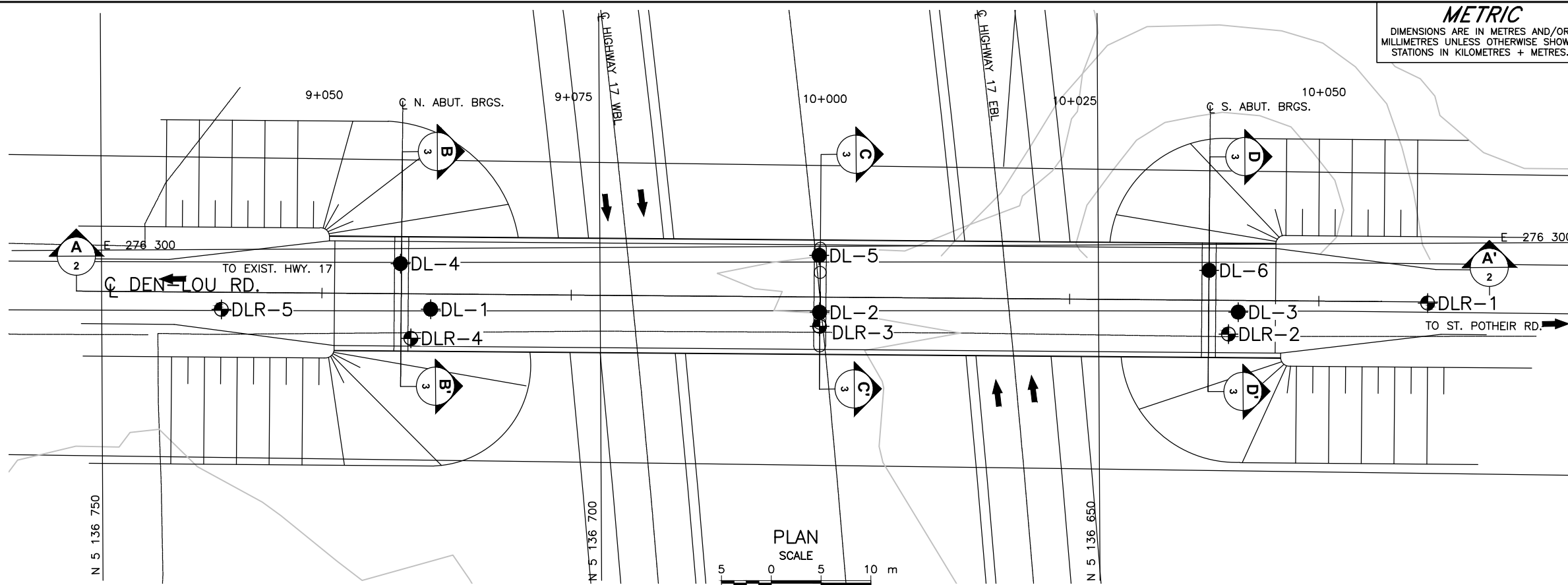
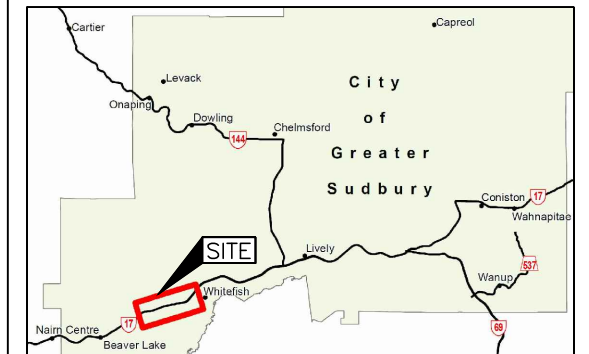
PLAN



REFERENCE

Base plans provided by Golder GIS and highway alignment provided in digital format by DM Wills, drawing file EBL & WBL PROFILES.dwg received Feb 28, 2013.

NO.	DATE	BY	REVISION
HWY. 17	PROJECT NO. 11-1191-0007		DIST.
SUBM'D. MT	CHKD.	DATE: APR 2015	SITE:
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 1

CONT No.
WP No.5593-09-01HIGHWAY 17 4 LANING
DEN-LOU ROAD UNDERPASS
BOREHOLE LOCATIONS AND
SOIL STRATASHEET
SEMPKEY PLAN
N.T.S.

LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation PML (2007)
- 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)
- R Refusal
- REC Recovery %
- ▽ WL in piezometer, measured on APR 29, 2014
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
DL-1	265.4	5136717.0	276293.9
DL-2	266.2	5136678.0	276293.4
DL-3	267.3	5136636.0	276293.2
DL-4	265.3	5136720.0	276298.5
DL-5	266.1	5136678.0	276299.1
DL-6	267.2	5136638.9	276297.4
DLR-1	267.2	5136617.0	276294.0
DLR-2	266.8	5136637.0	276291.0
DLR-3	265.7	5136678.0	276292.0
DLR-4	264.7	5136719.0	276291.0
DLR-5	265.5	5136738.0	276294.0

REFERENCE

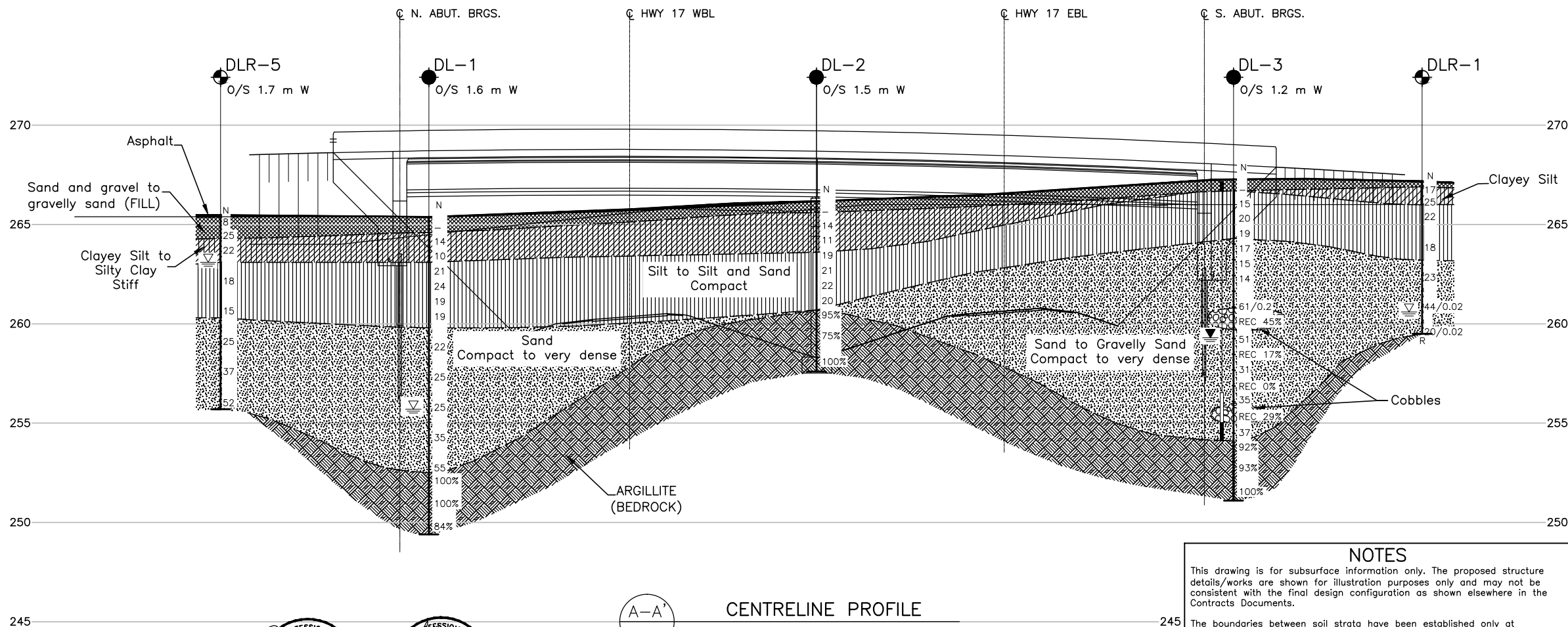
Base plans provided in digital format by DM Wills, drawing files 581_base.dwg, GWP156-98-00_B&C Plans.dwg and 581_contours.dwg received Jan 17, 2012. 09-4326 - Den-Lou Road Underpass GA Preliminary (Updated).dwg provided in digital format by DM Wills received on May 26, 2015. Supplemental boreholes DLR-1 to DLR-5 from Preliminary Foundation Investigation and Design Report - Peto MacCallum Ltd. July 2008.

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.

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.



NO.	DATE	BY	REVISION
1			
Geocres No. 411-324			
HWY. 17	PROJECT NO. 11-1191-0007	DIST. .	
SUBM'D. AC	CHKD. .	DATE: APR 2015	SITE: 46-566
DRAWN: J.J.L.	CHKD. AB	APPD. JMAC	DWG. 2

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

CONT No.
WP No.5593-09-01

HIGHWAY 17 4 LANING
DEN-LOU ROAD UNDERPASS
SOIL STRATA

LEGEND

- Borehole - Current Investigation
- Previous Borehole Location (2007)
- 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)
- R Refusal
- REC Recovery %
- WL in piezometer, measured on APR 29, 2014
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

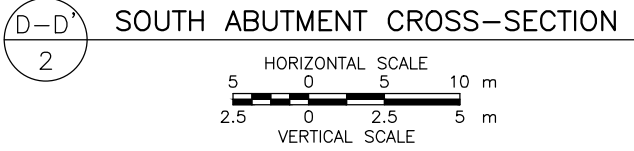
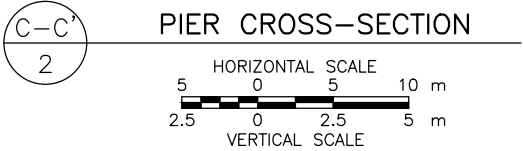
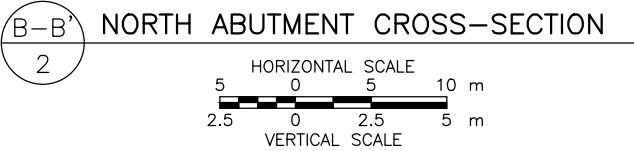
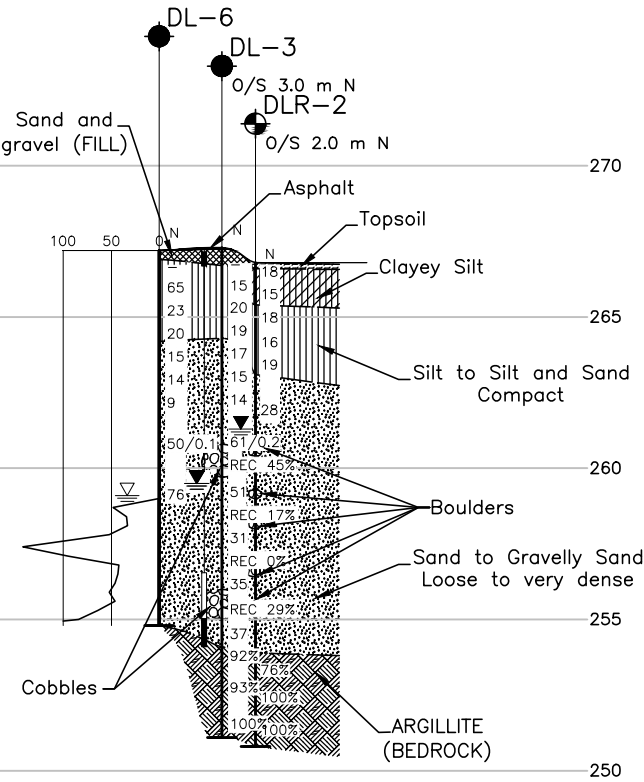
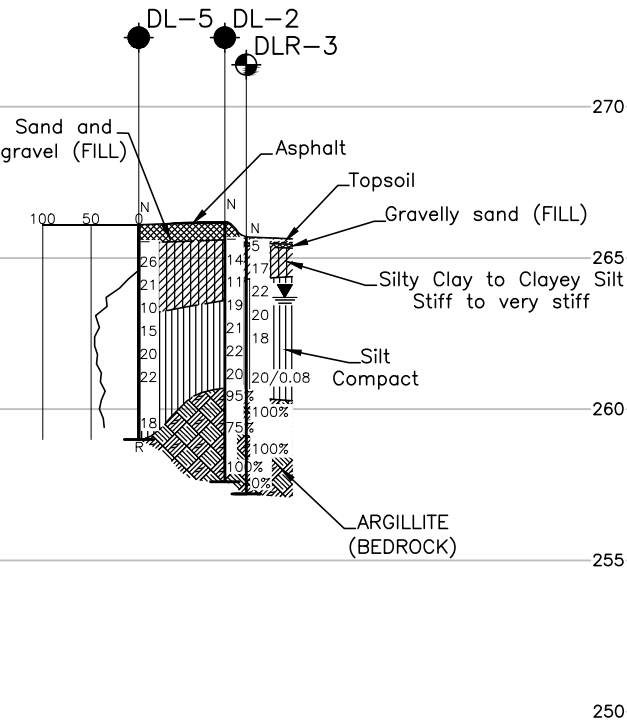
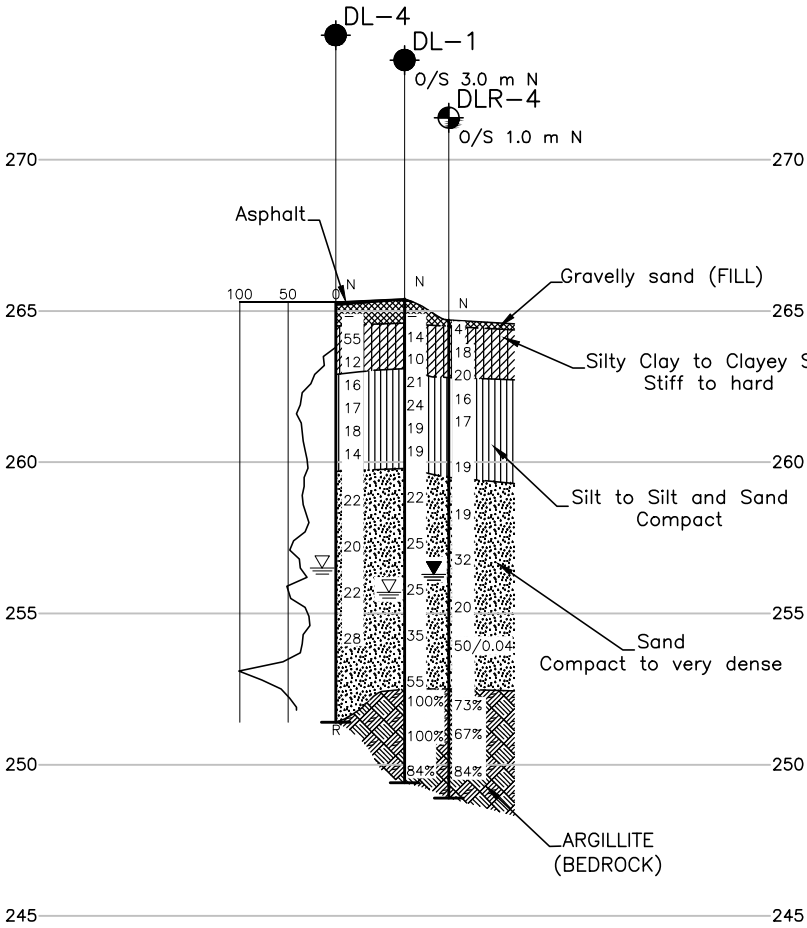
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DL-5	266.1	5136678.0	276299.1
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DLR-1	267.2	5136617.0	276294.0
DLR-2	266.8	5136637.0	276291.0
DLR-3	265.7	5136678.0	276292.0
DLR-4	264.7	5136719.0	276291.0
DLR-5	265.5	5136738.0	276294.0

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NO.	DATE	BY	REVISION
1	APR 15, 2015	JUL	1
2	APR 15, 2015	AB	2

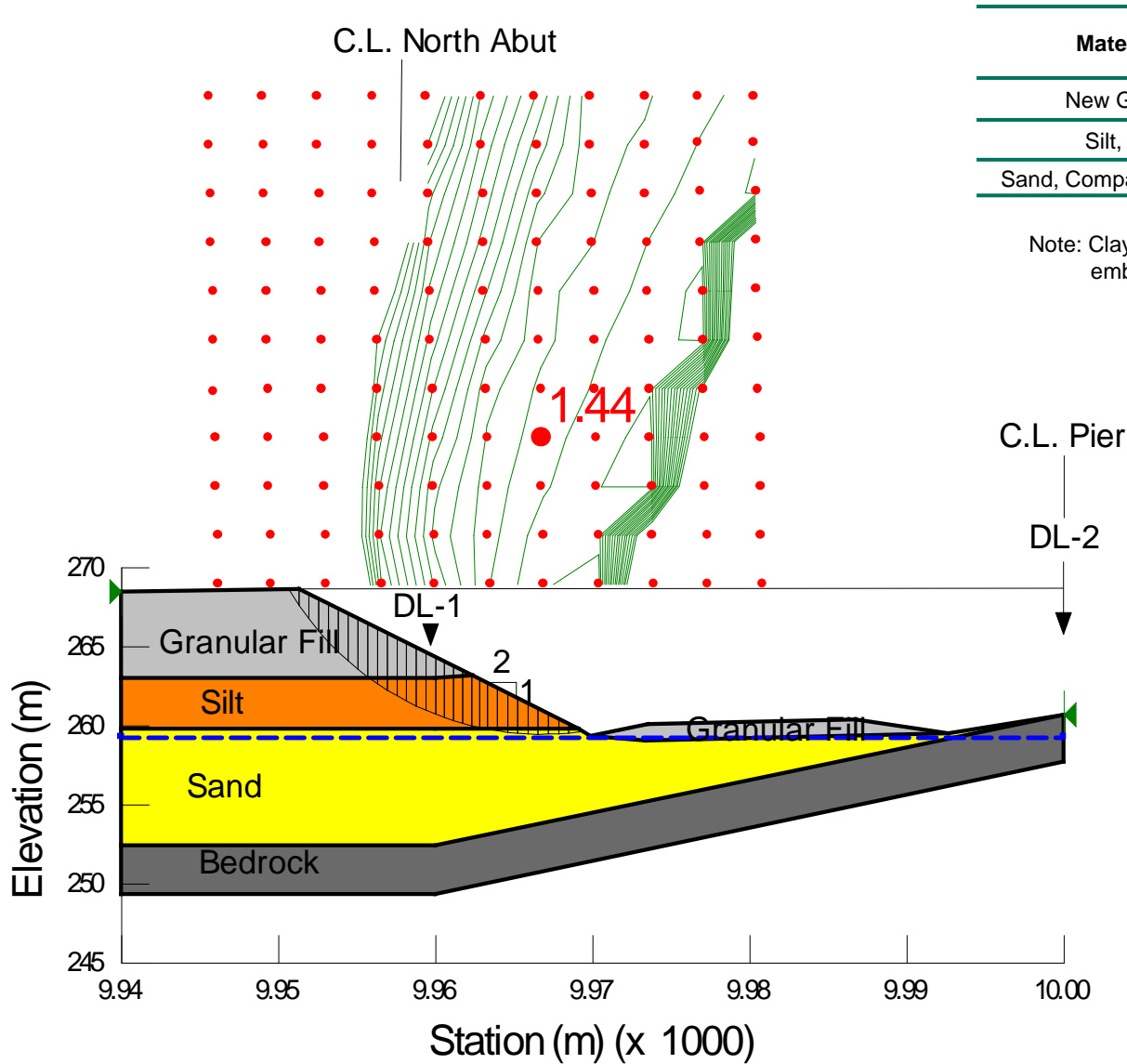
Geocres No. 411-324

HWY. 17	PROJECT NO. 11-1191-0007	DIST. .
SUBM'D. AC	CHKD. .	DATE: APR 2015
DRAWN: JUL	CHKD. AB	APPD. JMAC
SITE: 46-566		DWG. 3



Stability Analysis
North Abutment Front Slope

Figure 1



Material Name	Unit Weight (kN/m ³)	Friction Angle (°)
New Granular Fill	21	35
Silt, Compact	19	30
Sand, Compact to Very Dense	20	32

Note: Clayey Silt to Silty Clay deposit removed and replaced with embankment fill.



APPENDIX A

Record of Boreholes and Drillholes



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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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

(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 = σ'_p / σ'_{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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



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, 60° 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 cm² 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	N
Relative Density	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	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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

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° 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	

PROJECT 11-1191-0007				RECORD OF BOREHOLE No DL-1				1 OF 2 METRIC							
G.W.P. 156-98-00				LOCATION N 5136717.0; E 276293.9				ORIGINATED BY EHS							
DIST _____ HWY 17				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring				COMPILED BY AC							
DATUM Geodetic				DATE December 10 and 11, 2013				CHECKED BY AB							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
265.4	GROUND SURFACE							20 40 60 80 100							
0.9	Chip and tar		1	AS	-		265								
264.6	Gravelly sand, trace to some silt (FILL)														
0.8	Brown Moist (frozen)		2	SS	14		264								
	SILTY CLAY, trace sand														
	Stiff		3	SS	10										
	Brown Moist (frozen to 0.9 m)														
263.1							263								
2.3	SILT to Sandy SILT, trace clay		4	SS	21										
	Compact														
	Brown		5	SS	24		262								
	Moist														
			6	SS	19		261								
			7	SS	19										
259.8							260								
5.6	SAND, trace to some gravel														
	Compact to very dense		8	SS	22		259								
	Brown to grey														
	Moist to wet														
			9	SS	25		258								
							257								
			10	SS	25		256								
							255								
							254								
			12	SS	55		253								
252.5															
12.9	ARGILLITE (BEDROCK)														
	Bedrock cored from 12.9 m depth to 16.0 m depth.		1	RC	REC 100%		252								RQD = 100%
	For coring details see Record of Drillhole DL-1.		2	RC	REC 100%		251								RQD = 100%

Continued Next Page

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

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:

PROJECT <u>11-1191-0007</u>				RECORD OF BOREHOLE No DL-1				2 OF 2 METRIC												
G.W.P. <u>156-98-00</u>				LOCATION <u>N 5136717.0; E 276293.9</u>				ORIGINATED BY <u>EHS</u>												
DIST <u> </u> HWY <u>17</u>				BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>				COMPILED BY <u>AC</u>												
DATUM <u>Geodetic</u>				DATE <u>December 10 and 11, 2013</u>				CHECKED BY <u>AB</u>												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
	--- CONTINUED FROM PREVIOUS PAGE ---		2	RC																
			3	RC	REC 100%		250											RQD = 100%		
249.4																				
16.0	END OF BOREHOLE Note: 1. Water level at a depth of 9.6 m below ground surface (Elev. 255.8 m) upon completion of drilling.																			

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:

PROJECT: 11-1191-0007

RECORD OF DRILLHOLE: DL-1

SHEET 1 OF 1

LOCATION: N 5136717.0 ;E 276293.9

DRILLING DATE: December 10 and 11, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % RETURN	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.										NOTES WATER LEVELS INSTRUMENTATION									
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q AVG								
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION		Jr	Ja	Ja									
13	NW Coring	GROUND SURFACE		252.5																						
		ARGILLITE Strong Grey Very fine grained Fresh		12.9	1	GREY 100%																				
14	NQ Coring				2	GREY 100%							JNPLK													
15													JNPLSM													
													JNPLSM													
													JNIRRo													
16					3	GREY 100%							JNIRRo													
													JNIRRo													
													JNIRRo													
													JNPLRo													
													JNIRRo													
		END OF DRILLHOLE		249.4																						
				16.0																						
17																										
18																										
19																										
20																										
21																										
22																										

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0007.GPJ GAL-MISS.GDT 2503/15 DATA INPUT:

PROJECT 11-1191-0007			RECORD OF BOREHOLE No DL-2			1 OF 1 METRIC																			
G.W.P. 156-98-00			LOCATION N 5136678.0; E 276293.4			ORIGINATED BY EHS																			
DIST _____ HWY 17			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY AC																			
DATUM Geodetic			DATE December 12, 2013			CHECKED BY AB																			
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)										
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					W _p W W _L			WATER CONTENT (%)			γ			GR SA SI CL			
266.2	GROUND SURFACE							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					20 40 60			20 40 60			kN/m ³					
8.9	Chip and tar		1	AS	-		266																		
265.6	Sand and gravel, trace to some silt (FILL)		2	SS	14		265																	0 5 50 45	
0.6	Dark brown Moist (frozen)		3	SS	11		264																		
	SILTY CLAY, trace sand		4	SS	19		263																		
263.6	Stiff Brown Moist (frozen to 1.1 m)		5	SS	21		262																	0 5 87 8	
			6	SS	22		261																		
			7	SS	20		260																		
260.7	ARGILLITE (BEDROCK)		1	RC	REC 100%		259																	RQD = 95%	
5.5	Bedrock cored from 5.5 m depth to 18.6 m depth.		2	RC	REC 100%		258																	RQD = 75%	
	For coring details see Record of Drillhole DL-2.		3	RC	REC 100%																			RQD = 100%	
257.6	END OF BOREHOLE																								
8.6	Note: 1. Borehole dry upon completion of drilling.																								

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:

PROJECT: 11-1191-0007

RECORD OF DRILLHOLE: DL-2

SHEET 1 OF 1

LOCATION: N 5136678.0 ;E 276293.4

DRILLING DATE: December 12, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR FLUSH % RETURN	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.										NOTES WATER LEVELS INSTRUMENTATION									
							RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY k, cm/s					Diametral Point Load Index (MPa)	RMC -Q AVG				
							TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10 ⁰	10 ¹	10 ²	10 ³						
	NW	GROUND SURFACE		260.7																						
6		ARGILLITE Very strong Grey Very fine grained Fresh		5.5	1	GREY 100%							JNIRRo JNIRRo JNUNRo													
7	NQ Coring				2	GREY 100%							JNIRRo JNIRRo JNIRRo													
8					3	GREY 100%							JNIRRo JNIRRo													
		END OF DRILLHOLE		257.6																						
9				8.6																						
10																										
11																										
12																										
13																										
14																										
15																										

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0007.GPJ GAL-MISS.GDT 2503/15 DATA INPUT:

PROJECT 11-1191-0007				RECORD OF BOREHOLE No DL-3				1 OF 2 METRIC						
G.W.P. 156-98-00				LOCATION N 5136636.0; E 276293.2				ORIGINATED BY EHS						
DIST _____ HWY 17				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring				COMPILED BY AC						
DATUM Geodetic				DATE December 16 to 18, 2013				CHECKED BY AB						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
267.3	GROUND SURFACE													
8.9	Chip and tar		1	AS	-									
266.7	Sand and gravel, trace to some silt (FILL)		2	SS	15									
0.6	Dark brown Moist (frozen)		3	SS	20									
	SILT to SILT and SAND		4	SS	19									
	Compact Brown Moist (frozen to 0.9 m)													
264.3	SAND to Gravelly SAND, trace to some silt		5	SS	17									0 40 57 3
3.0	Compact to very dense Brown to grey Moist to wet		6	SS	15									
			7	SS	14									1 95 (4)
	Augers grinding at 5.9 m depth.		8	SS	61/0.2									
	Cobbles 125 mm and 110 mm diameter recovered from 6.5 m to 7.6 m depth.		-	RC	REC 45%									
			9	SS	51									21 55 19 5
			-	RC	REC 17%									
	Cobble encountered at 10.1 m depth.		10	SS	31									
			-	RC	REC 0%									
			11	SS	35									
			-	RC	REC 29%									
	Cobbles and gravel 90 mm to 125 mm diameter recovered from 11.3 m to 12.3 m depth.		12	SS	37									
254.1	ARGILLITE (BEDROCK)		1	RC	REC 100%									RQD = 92%
13.2	Bedrock cored from 13.2 m depth to 16.2 m depth.		2	RC	REC 100%									RQD = 93%
	For coring details see Record of Drillhole DL-3.													

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:

Continued Next Page

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

PROJECT <u>11-1191-0007</u>		RECORD OF BOREHOLE No DL-3				2 OF 2 METRIC																	
G.W.P. <u>156-98-00</u>		LOCATION <u>N 5136636.0; E 276293.2</u>				ORIGINATED BY <u>EHS</u>																	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, NQ Coring</u>				COMPILED BY <u>AC</u>																	
DATUM <u>Geodetic</u>		DATE <u>December 16 to 18, 2013</u>				CHECKED BY <u>AB</u>																	
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)						
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>																
251.1		2	RC			252																	
16.2	END OF BOREHOLE Note: 1. Water level at a depth of 7.9 m below ground surface (Elev. 259.4 m) upon completion of drilling. 2. Water level in piezometer 8.0 m below ground surface (Elev. 259.3) measured on April 29, 2014.	3	RC	REC 100%													RQD = 93%						
																	RQD = 100%						

PROJECT: 11-1191-0007

RECORD OF DRILLHOLE: DL-3

SHEET 1 OF 1

LOCATION: N 5136636.0 ;E 276293.2

DRILLING DATE: December 16 to 18, 2013

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	COLOUR % RETURN	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.										NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
				DEPTH (m)			FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION		Jr		Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja	Ja

DEPTH SCALE

1 : 50



LOGGED: EHS

CHECKED: AB

SUD-RCK 11-1191-0007.GPJ GAL-MISS.GDT 2503/15 DATA INPUT:

PROJECT		11-1191-0007		RECORD OF BOREHOLE No DL-4		1 OF 2 METRIC	
G.W.P.		156-98-00		LOCATION		N 5136720.0; E 276298.5	
DIST		HWY 17		BOREHOLE TYPE		108 mm I.D. Continuous Flight Hollow Stem Augers	
DATUM		Geodetic		DATE		March 6, 2014	
						ORIGINATED BY EHS	
						COMPILED BY AC	
						CHECKED BY AB	
SOIL PROFILE				SAMPLES		DYNAMIC CONE PENETRATION RESISTANCE PLOT	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	
265.3	GROUND SURFACE						
0.0	Chip and tar		1	AS	-		
0.1	Gravelly sand, trace silt (FILL)						
264.5	Brown Moist (frozen)						
0.8	SILTY CLAY, trace sand		2	SS	55		
	Stiff to hard Brown Moist (frozen to 1.5 m)		3	SS	12		
262.9			4	SS	16		
2.4	SILT to SILT and SAND, trace clay		5	SS	17		
	Compact Brown Moist to wet		6	SS	18		
			7	SS	14		
259.7							
5.6	SAND, trace to some silt, trace to some gravel		8	SS	22		
	Compact Brown to grey Moist to wet						
			9	SS	20		
			10	SS	22		
			11	SS	28		
	Approximately 0.6 m of heave encountered at 11.3 m depth.						
251.4							
13.9							


Continued Next Page

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





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

SSUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:

PROJECT 11-1191-0007			RECORD OF BOREHOLE No DL-5			1 OF 1 METRIC														
G.W.P. 156-98-00			LOCATION N 5136678.0; E 276299.1			ORIGINATED BY EHS														
DIST _____ HWY 17			BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers			COMPILED BY AC														
DATUM Geodetic			DATE March 7, 2014			CHECKED BY AB														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L WATER CONTENT (%)			γ	GR	SA	SI	CL
266.1	GROUND SURFACE							20 40 60 80 100												
0.0	Sand and gravel, trace to some silt (FILL)		1	AS	-		266													
265.5	Brown Moist (frozen)																			
0.6	SILTY CLAY to CLAYEY SILT, trace sand		2	SS	26			265												
	Stiff to very stiff Brown Moist (frozen to 1.2 m)		3	SS	21			264												
			4	SS	10		263													
263.1	SILT, trace to some clay, trace to some sand Compact Brown Moist to wet		5	SS	15		262													
3.0	Auger refusal on cobbles and / or boulders encountered at 3.7 m depth, moved 1.0 m east and advanced borehole to 3.7 m depth to continue sampling.		6	SS	20		261													
			7	SS	22		260													
			8	SS	18		259													
259.0	END OF BOREHOLE AUGER REFUSAL																			
7.1	Note: 1. Borehole dry upon completion of drilling. 2. Advanced DCPT 1.2 m west of borehole, preaugered to 1.5 m depth, refusal at 6.6 m depth.																			

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:

PROJECT 11-1191-0007				RECORD OF BOREHOLE No DL-6				1 OF 1 METRIC										
G.W.P. 156-98-00				LOCATION N 5136638.9; E 276297.4				ORIGINATED BY EHS										
DIST _____ HWY 17				BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers				COMPILED BY AC										
DATUM Geodetic				DATE March 10, 2014				CHECKED BY AB										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
267.2	GROUND SURFACE							20	40	60	80	100						
0.0	Sand and gravel, trace to some silt (FILL)		1	AS	-		267											
266.9	Brown Moist (frozen)		2	SS	65		266											
0.3	SILT to Sandy SILT Very dense to compact Brown Moist (frozen to 1.5 m depth)		3	SS	23		265											0 28 71 1
			4	SS	20		264											
264.2							263											
3.0	SAND to Gravelly SAND, trace to some silt Loose to very dense Brown to grey Moist	5	SS	15	262													
		6	SS	14	261													
		7	SS	9	260													
		8	SS	50/0.1	259													
	Augers grinding from 6.4 m to 6.7 m depth.				258													
259.0	END OF BOREHOLE START OF DCPT		9	SS	76		257											
8.2							256											
							255											
254.8	END OF DCPT																	
12.4	Note: 1. Water level at a depth of 8.1 m below ground surface (Elev. 259.1 m) upon completion of drilling.																	

SUD-MTO 001 11-1191-0007.GPJ GAL-MISS.GDT 25/03/15 DATA INPUT:



APPENDIX B

Record of Boreholes – PML (2007)

RECORD OF BOREHOLE No DLR-1

1 of 1

METRIC

G.W.P. 156-98-00 LOCATION Co-ords 5 136 617 N; 276 294 E ORIGINATED BY F.P.
DIST Sudbury HWY 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.
DATUM Geodetic DATE April 19, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
267.2	Ground Surface							20	40	60	80	100					
0.0	Asphalt (50mm) over Sand and gravel						267										
0.3	Compact Brown (PAVEMENT FILL)		1	SS	17												
266.0	Clayey silt trace sand, organics		2	SS	25		266										
1.2	Very stiff Brown Moist Silt some sand, trace clay thin layers of sandy silt		3	SS	22												
	Compact Brown Moist						265										
			4	SS	18		264										
263.2	Sand, trace silt Compact Brown Damp		5	SS	23		263										
							262										
260.6	trace gravel Moist		6	SS	44/23cm		261										
6.6	Gravelly sand, trace silt cobbles																
	Very dense Grey Wet						260										
259.5			7	SS	20/2cm												
7.7	End of borehole Refusal on probable bedrock																
	Samples 6 and 7: Sampler bouncing																
	* 2007 04 19																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																

RECORD OF BOREHOLE No DLR-2

1 of 2

METRIC

G.W.P. 156-98-00 LOCATION Co-ords 5 136 637 N; 276 291 E ORIGINATED BY F.P.
DIST Sudbury HWY 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.
DATUM Geodetic DATE April 20, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
266.8	Ground Surface						20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL				
0.0	Topsoil		1	SS	18															
0.2	Clayey silt, trace sand layers of silty sand Very stiff Brown Moist		2	SS	15															
265.4																				
1.4	Silt with sand, trace clay Compact Brown Moist		3	SS	18															
			3A	SS	16															
			4	SS	19															
263.1																				
3.7	Sand, some silt thin layers of silty sand Compact Brown Dry		5	SS	28															

	trace gravel																			

	cobbles and boulders																			

	0.3m boulder																			
															</					

Cont'd

ON_MOT VER3 05TF059-DLR.GPJ ON_MOT.GDT 7/3/2008 4:12:07 PM

+⁷, X⁵: Numbers refer to
Sensitivity 20
15-5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No DLR-2

2 of 2

METRIC

G.W.P. 156-98-00 LOCATION Co-ords 5 136 637 N; 276 291 E ORIGINATED BY F.P.
 DIST Sudbury HWY 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.
 DATUM Geodetic DATE April 20, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
251.8			8	RC NQ	REC 100%												RQD = 100%
250.8																	
16.0	End of borehole																
	Borehole advanced by coring from 5.9m depth.																
	* 2007 4 20																
	▽ Water level observed during drilling																

RECORD OF BOREHOLE No DLR-3

1 of 1

METRIC

G.W.P. 156-98-00 LOCATION Co-ords 5 136 678 N; 276 292 E ORIGINATED BY F.P.
DIST Sudbury HWY 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.
DATUM Geodetic DATE April 21, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
265.7	Ground Surface																
0.0	Topsoil (150mm)																
0.2	Gravelly sand		1	SS	5												
0.3	Brown Wet (FILL)																
	Silty clay, trace sand		2	SS	17												
264.3	Very stiff Brown Moist																
1.4	Silt with sand, trace clay		3	SS	22												
	Compact Brown Wet																
	layers of sandy silt		3A	SS	20												
			4	SS	18												
	trace gravel cobbles and boulders		5	SS	20/8cm												
260.4	Bedrock																
5.3	Argillite		6	RC NQ	REC 100%												
	Dark grey to black																
	High strength, becoming very high strength																
	Excellent to very poor quality		7	RC NQ	REC 100%												
			8	RC NQ	REC 40%												
257.2	End of borehole																
8.5																	
	Sample 5: Sampler bouncing																
	RC 8 : Bottom 0.4m of core could not be retrieved																
	* 2007 04 21																
	Water level observed during drilling																

METRIC

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

RECORD OF BOREHOLE No DLR-4 2 of 2 METRIC

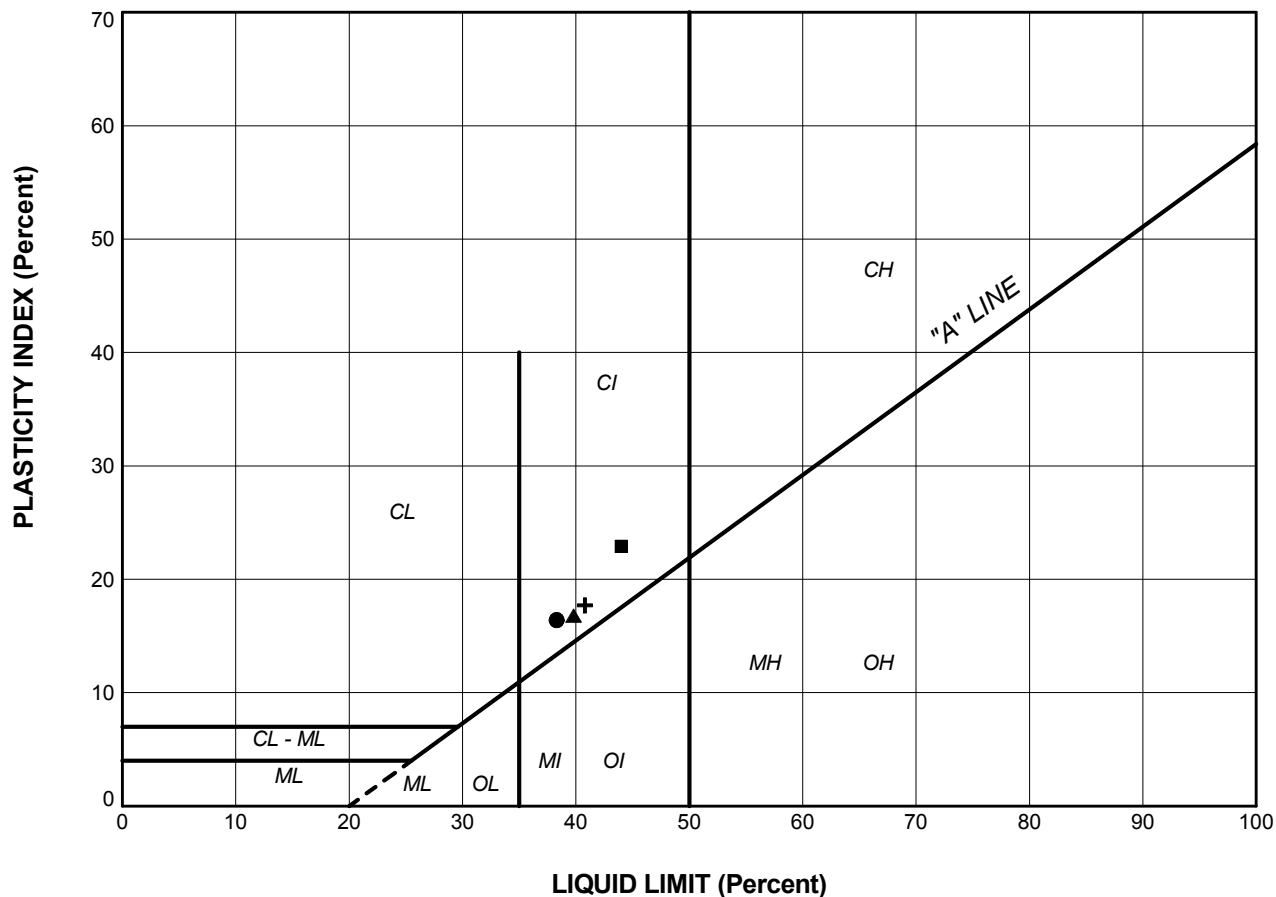
G.W.P. 156-98-00 LOCATION Co-ords 5 136 719 N; 276 291 E ORIGINATED BY F.P.
 DIST Sudbury HWY 17 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY F.P.
 DATUM Geodetic DATE April 22, 2007 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
249.7																	
248.9			12	RC NQ	REC 89%		249										RQD = 84%
15.8	End of borehole																
	Sample 9: Sampler bouncing																
	* 2007 4 22																
	▽ Water level observed during drilling																




APPENDIX C

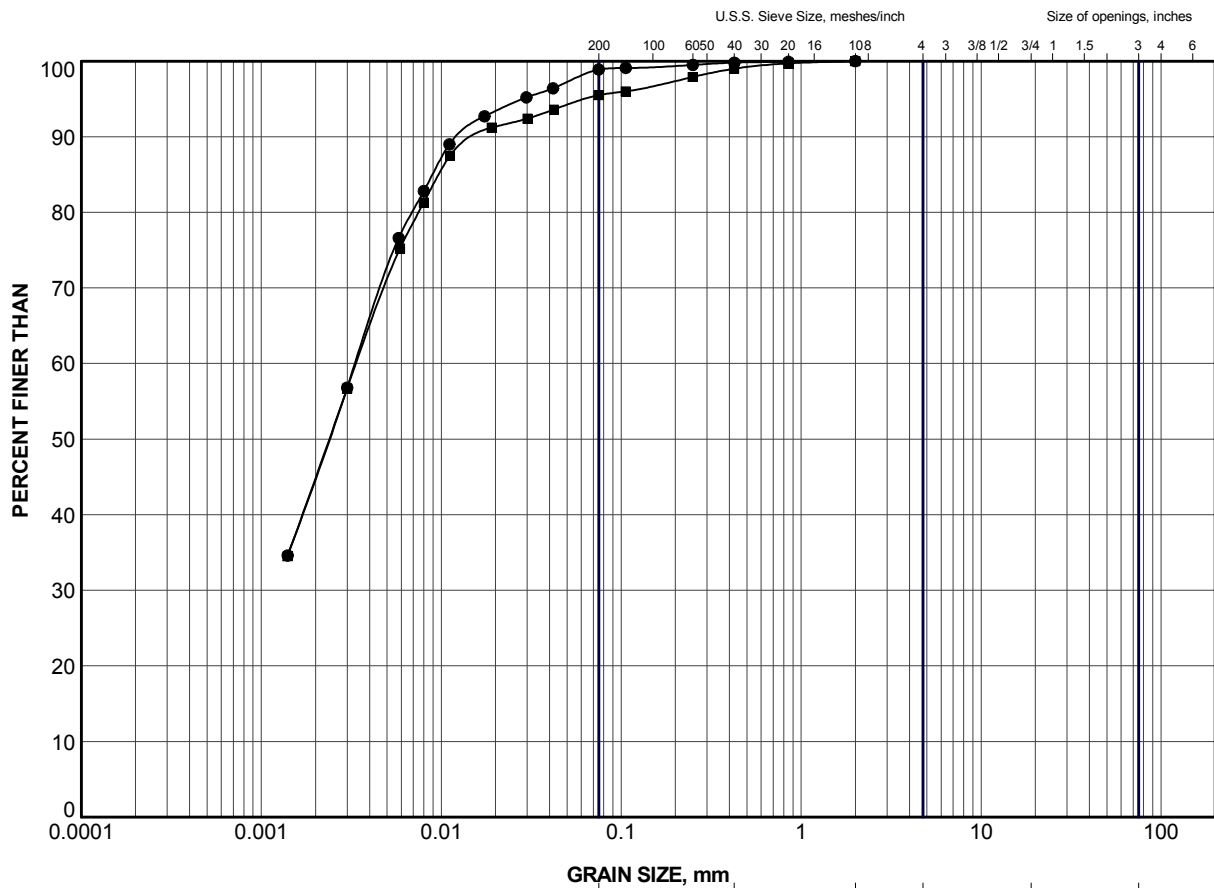
Laboratory Test Results – Current Investigation



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	DL-1	3	38.3	21.9	16.4
■	DL-2	2	44.0	21.1	22.9
▲	DL-4	3	39.8	23.0	16.8
+	DL-5	2	40.8	23.1	17.7

PROJECT					
HIGHWAY 17 DEN-LOU ROAD UNDERPASS					
TITLE					
PLASTICITY CHART SILTY CLAY					
PROJECT No.		11-1191-0007		FILE No.	
DRAWN		JJL		Apr 2014	
CHECK		AB		Apr 2014	
APPR				Apr 2014	
 Golder Associates SUDBURY, ONTARIO				SCALE N/A REV.	
FIGURE C1					



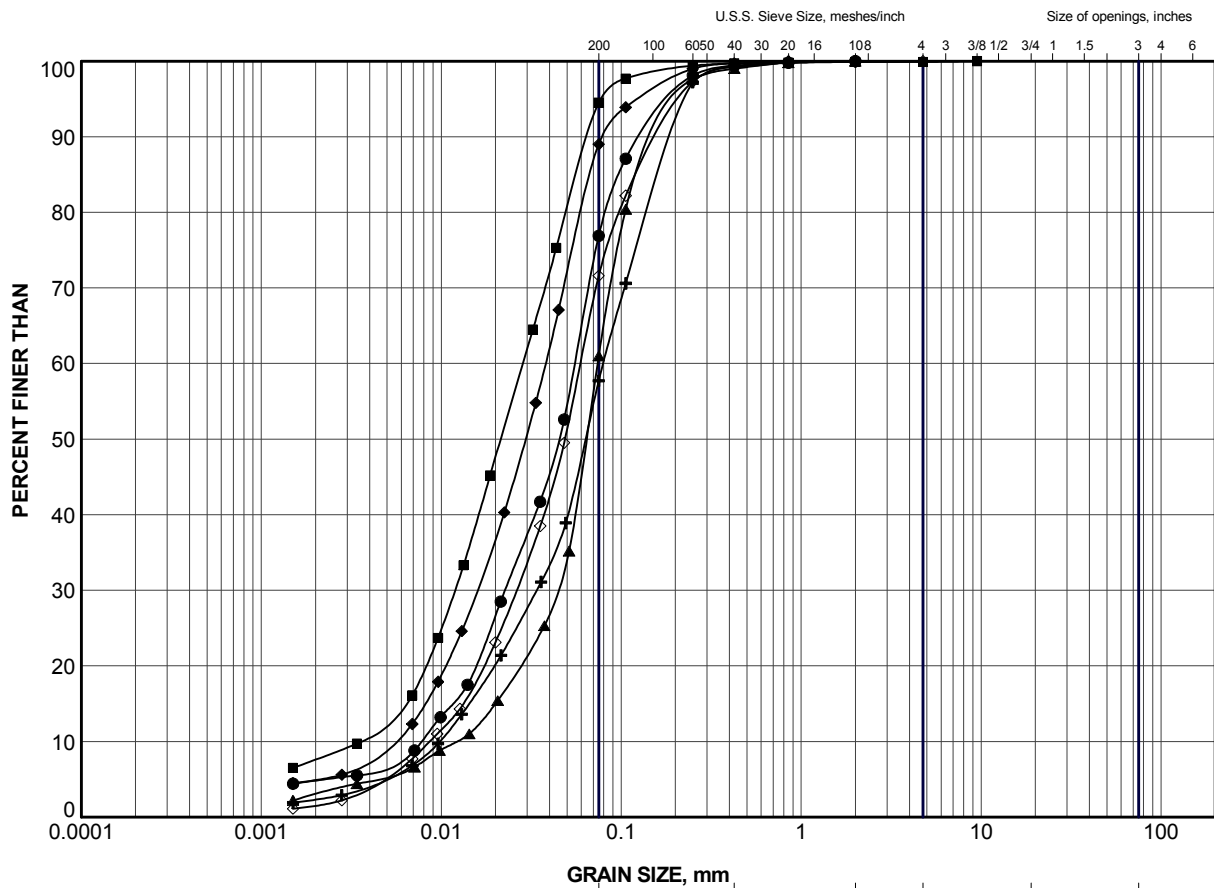
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	DL-1	3	263.6
■	DL-2	2	265.1

PROJECT					HIGHWAY 17 DEN-LOU ROAD UNDERPASS				
TITLE					GRAIN SIZE DISTRIBUTION SILTY CLAY				
PROJECT No.		11-1191-0007		FILE No.		11-1191-0007.GPJ			
DRAWN	JJL	Apr 2014		SCALE	N/A	REV.			
CHECK	AB	Apr 2014		FIGURE C2					
APPR		Apr 2014							





LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	DL-1	7	260.5
■	DL-2	5	262.8
▲	DL-3	4	264.7
+	DL-4	7	260.4
◆	DL-5	6	262.0
◇	DL-6	3	265.4

PROJECT

HIGHWAY 17
DEN-LOU ROAD UNDERPASS

TITLE

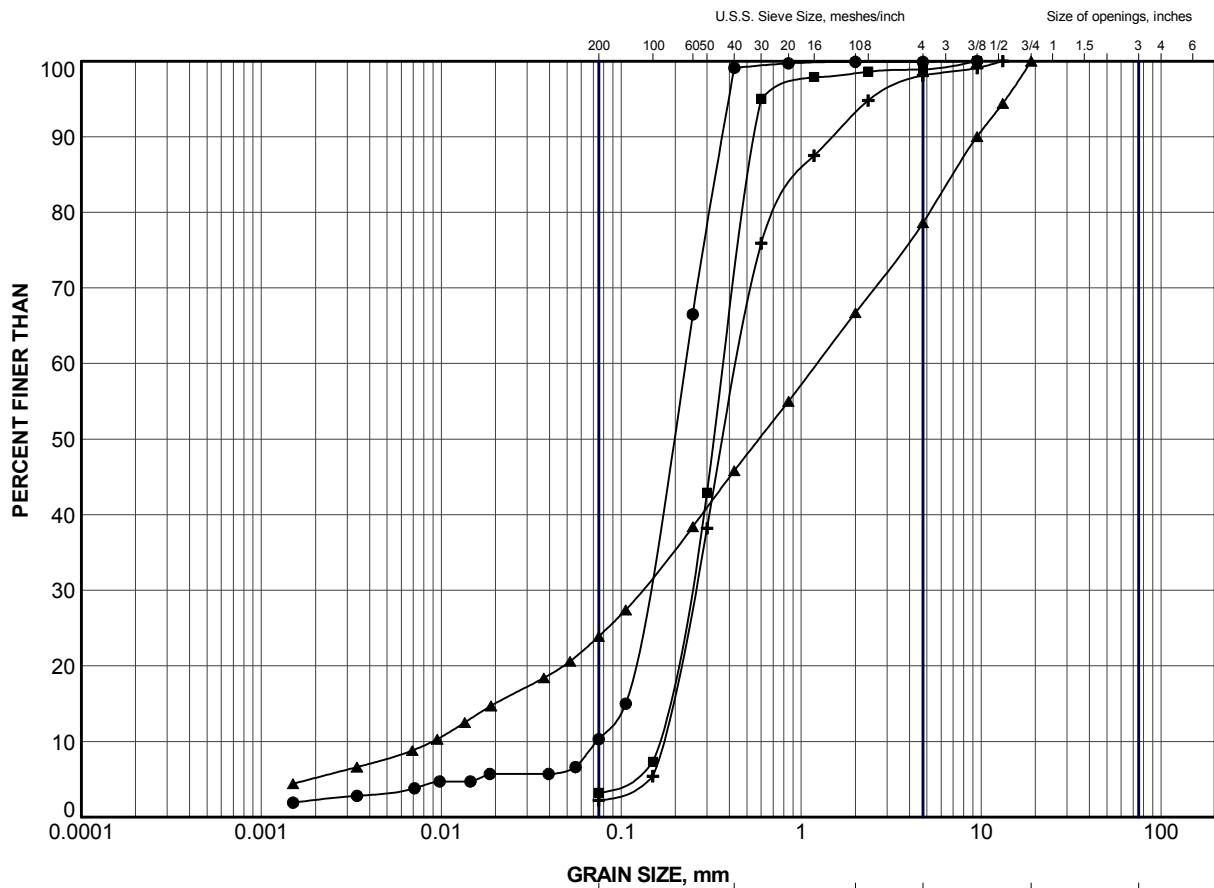
GRAIN SIZE DISTRIBUTION

SILT to SILT and SAND



PROJECT No.	11-1191-0007	FILE No.	11-1191-0007.GPJ
DRAWN	JJL	Apr 2014	SCALE N/A
CHECK	AB	Apr 2014	REV.
APPR		Apr 2014	


FIGURE C3



CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	DL-1	10	256.0
■	DL-3	7	262.4
▲	DL-3	9	259.4
+	DL-6	7	262.3

PROJECT					
HIGHWAY 17 DEN-LOU ROAD UNDERPASS					
TITLE					
GRAIN SIZE DISTRIBUTION SAND to GRAVELLY SAND					
PROJECT No.		11-1191-0007		FILE No. 11-1191-0007.GPJ	
DRAWN	JJL	Apr 2014	SCALE	N/A	REV.
CHECK	AB	Apr 2014	FIGURE C4		
APPR		Apr 2014			
 Golder Associates SUDBURY, ONTARIO					



APPENDIX D

Non-Standard Special Provisions

CSP FOR INTEGRAL ABUTMENTS – Item No.

Non-Standard Special Provision

Scope

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801 and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract Drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract Drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Weight
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Install piles by driving to the design tip elevation or bedrock if end-bearing piles are selected.
4. Place loose sand into the CSP.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the top of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

Basis of Payment

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

H-PILES - Item No.

Non-Standard Special Provision

903.07.02 Driven Piles

903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 12 mm of penetration shall be obtained at the maximum hammer energy.

If unusually excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

MASS CONCRETE - Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes mass concrete under the centre pier footings.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904.

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.

DOWELS INTO ROCK - Item No.

Non-Standard Special Provision

Scope of Work

As part of the work under the above tender item, the Contractor shall include mass concrete under the east abutment.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days.

If hole contains water, the Contractor shall remove the water, otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D3689-07, ASTM D1143-07 and ASTM D4435-08. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of rock dowels where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Bridge	Foundation	Number of Dowels for Performance Testing
Den-Lou Road Underpass	Centre Pier	2

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25
Cycle-Step	3-1	3-2	3-3	3-4	3-5		
% Design Load	50	75	100	110	25		

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three (3) additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

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.

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor is hereby notified that the soils at the site of the Den-Lou Road Underpass structure site are compact to very dense and should be expected to contain cobbles and boulders, which could affect excavations and the installation of deep foundations. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for sub-excavation and installation of the foundations.

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.

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. 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 who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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