August 31, 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 524 Commanda Creek Bridge Replacement (Site No. 44-029)
Ministry of Transportation, Ontario
Pringle Township, Parry Sound District, Ontario
GWP 5260-13-00

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REPORT

Golder Associates

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HIGHWAY 524 COMMANDA CREEK BRIDGE REPLACEMENT
(SITE NO. 44-029)
PRINGLE TOWNSHIP, PARRY SOUND DISTRICT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide detailed foundation engineering services for the replacement of the Commanda Creek Bridge on Highway 524 (Site No. 44-029) in Pringle Township, Parry Sound District, Ontario.

The purpose of this field investigation is to establish the subsurface conditions at the location of the proposed bridge, including the associated approach embankments, by methods of borehole drilling, rock coring, in situ testing and laboratory testing on selected soil and rock cores samples.

This report summarizes the factual results of field and laboratory work (including field investigation procedures, borehole stratigraphy, bedrock lithology, and geotechnical and analytical laboratory test results) as well as a description of the interpreted soil, bedrock and groundwater conditions at the Commanda Creek Bridge site.

The Terms of Reference and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal, dated December 8, 2015. Golder's proposal for foundation engineering services is contained in Section 17.8 of AECOM's Technical Proposal for this assignment. The Base Plan showing the proposed horizontal alignment of Highway 524, the vertical profile drawing showing the proposed grade of Highway 524, and the General Arrangement drawing prepared by MTO of the proposed Commanda Creek Bridge were provided to Golder by AECOM on November 29, 2016.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

It is understood that the existing Commanda Creek Bridge, which currently accommodates one lane of traffic, will be replaced with a new bridge on a new Highway 524 alignment. The new single span bridge will accommodate one lane of traffic in each direction. The centreline of the new Highway 524 alignment at the location of the new bridge will be shifted about 10 m west of the current centreline alignment.

2.2 Site Description

The site of the proposed bridge replacement is located in Pringle Township within the Almaguin Highlands Region of the Parry Sound District. The existing modular bridge carries Highway 524 over Commanda Creek. Highway 524 cuts through the site in a generally southwest to northeast direction, and then turns northwesterly about 40 m north of the bridge. The highway consists of one lane in each direction; however, the bridge currently accommodates only one lane of traffic. In the vicinity of the site, Commanda Creek is about 10 m wide and flows northwesterly. Overhead power lines run generally along the west side of the bridge, but also extend over Highway 524 at the bridge and about 15 m south of the bridge.

The natural ground surface at the site varies from approximately Elevation 223.5 m to Elevation 228 m. The existing Highway 524 has been constructed on an approximately 1 m to 2 m high embankment, with the pavement grade at approximately Elevation 228 m in the vicinity of the existing bridge. The creek water level was measured by others at about Elevation 223.2 m on June 1, 2016.

The topography of the area is relatively hilly/rolling and is interspersed with lakes and rivers/creeks. The site is surrounded primarily by agricultural fields to the south and east and densely treed areas elsewhere. Several residential dwellings are also located south and east of the site. Vegetation in this area consists primarily of coniferous trees, but also includes deciduous trees, shrubs, and grasses.

3.0 FIELD INVESTIGATION PROCEDURES

The field work at the proposed bridge site was carried out between November 21 and 26, 2016, during which time six boreholes were advanced within the footprint of the proposed bridge abutments and approach embankments, as follows:

Foundation Element / Approach Embankment	Relevant Borehole(s)
South Approach Embankment	16-01
South Abutment	16-02 and 16-03
North Abutment	16-04 and 16-05
North Approach Embankment	16-06

The subsurface soil and bedrock conditions encountered in the boreholes are shown in detail on the borehole/drillhole records in Appendix A. Lists of abbreviations and symbols and a description of lithological and geotechnical rock description terminology are also provided in Appendix A to assist in the interpretation of the borehole and drillhole records. The locations of the as-drilled boreholes are shown in plan on Drawing 1.

The boreholes were drilled using a CME-850 track-mounted drill rig supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario. The boreholes were advanced through the overburden using 178 mm outer diameter continuous flight hollow stem augers and/or NW casing with wash boring techniques. Soil samples were generally obtained at intervals of depth of about 0.75 m and 1.5 m, using a50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Rock core samples were recovered from Boreholes 16-02 to 16-05 using an ‘NQ’ double tube rock core barrel. Bedrock quality and discontinuity data were recorded in the field based on visual inspection of the recovered bedrock core extracted from the core barrel. Photographs of the recovered rock core samples are provided in Appendix B.

The boreholes were advanced to depths ranging between about 8.4 m and 18.6 m below existing ground surface. Boreholes 16-01 and 16-06 were terminated upon encountering casing or auger/split-spoon refusal, while Boreholes 16-02 to 16-05 were cored for lengths ranging between about 3.1 m and 6.3 m prior to terminating the boreholes in the bedrock.

The groundwater conditions and water level in the boreholes (i.e., inside the hollow stem augers or casing) were generally observed during drilling operations and measured prior to wash boring/rock coring operations. A monitoring well was installed in Borehole 16-05 to permit monitoring of the groundwater level at this location. The monitoring well consists of a 38 mm diameter PVC pipe with a 3 m long slotted screen which is surrounded with filter sand. The borehole and annulus surrounding the well pipe above the screen/filter sand was backfilled to the surface with bentonite pellets. The well installation details and water level readings are presented on the record for Borehole 16-05 provided in Appendix A. Boreholes 16-01 to 16-04 and 16-06, which were not instrumented with a monitoring well, were backfilled upon completion of drilling/coring in accordance with Ontario Regulation 903 (Wells) (as amended).

Prior to commencement of field work, Golder carried out a site visit and arranged for the clearance of underground utilities/services. The field work was observed on a full-time basis by a member of Golder’s engineering staff who monitored the drilling/coring, in situ testing and sampling operations, and logged the boreholes in the field. The soil and rock core samples were transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and geotechnical laboratory testing.

Geotechnical classification testing (i.e., water content, organic content, Atterberg limits and grain size distribution) was carried out on selected soil samples. In addition, a consolidated drained direct shear test was carried on a non-cohesive sample of silty sand recovered from Borehole 16-05, and a one-dimensional consolidation (i.e., an Oedometer) test was carried out on a cohesive soil sample recovered from Borehole 16-03. Unconfined Compressive Strength (UCS) testing was carried out on selected bedrock core samples. The results of the geotechnical laboratory testing are summarized on the borehole and drillhole records in Appendix A and the details of the geotechnical testing are provided in Appendix B. All of the laboratory tests were carried out to MTO Laboratory and/or ASTM Standards, as appropriate.

Classification of the rock mass quality of the bedrock cores with respect to the Rock Quality Designation (RQD) is described based on Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)[1]. The degree of weathering of the bedrock samples (i.e., fresh – W1) and the strength classification of the intact rock mass based off field identification (i.e., strong to very strong – R4 to R5) are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM, 1985)[2] standard classification system.

Two soil samples were also collected from the boreholes for corrosivity testing. The selected soil samples were submitted, under chain-of-custody procedures, to Maxxam Analytics of Mississauga, Ontario (a Standards Council of Canada accredited laboratory) for analysis of a suite of corrosivity parameters including pH, sulphate, chloride and resistivity/conductivity.

The planned borehole locations and corresponding ground surface elevations were surveyed by Callon Dietz Inc. prior to mobilizing to site. Upon completion of drilling/coring operations, any offsets to the borehole locations and corresponding ground surface elevation changes were recorded and tied in to the originally surveyed borehole locations to determine the as-drilled borehole locations and ground surface elevations. The borehole survey information, including northing and easting coordinates (presented in the MTM NAD83 zone coordinate system) and the ground surface elevations referenced to Geodetic datum, are provided on the borehole records in Appendix A, presented on Drawings 1 to 3, and summarized below.

Foundation Element	Borehole Designation	Coordinates (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m)	Easting (m)		
South Approach	16-01	5,095,962.0	292,711.6	227.3	11.5
South Abutment	16-02	5,095,974.6	292,717.9	227.1	18.6 (1)
	16-03	5,095,980.5	292,725.4	226.9	15.1 (1)

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

[2] International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech.Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

Foundation Element	Borehole Designation	Coordinates (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m)	Easting (m)		
North Abutment	16-04	5,096,003.9	292,739.5	227.0	14.0 (1)
	16-05	5,096,010.4	292,733.4	226.0	9.6 (1)
North Approach	16-06	5,096,028.1	292,744.9	227.0	8.4

Note:
1. Includes bedrock coring between 3.1 m and 6.3 m lengths

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[3], this site lies within the western limit of the physiographic region known as the Algonquin Highlands. This region occupies approximately 4 million hectares and is the largest physiographic region in Southern Ontario. The relief is rough and includes rounded knobs and ridges which are generally 15 m to 60 m high, but can be up to about 150 m high. Bedrock outcrops are also interspersed throughout this region. The overburden cover is typically shallow, but can vary significantly over short distances. The soil is comprised of stony, sandy and acidic soils, but the soils encountered within the valleys generally include outwash sand and gravel. In places, occasional drumlins can be found and the overburden in these areas is comprised of deeper till deposits.

The bedrock in this physiographic region generally consists of granite and other Precambrian igneous and high-grade metamorphic rocks.

4.2 Overview of Local Subsurface Conditions

The subsurface soil, bedrock and groundwater conditions encountered in the boreholes advanced at this site, together with the results of in situ and geotechnical/analytical laboratory testing, are presented on the borehole and drillhole records (provided in Appendix A) and the laboratory test figures/sheets (provided in Appendices B and C). The results of the in situ field tests (i.e., SPT ‘N’-values) as presented on the borehole records and in Section 4.2 are uncorrected, and are based on sampling procedures carried out with an automatic hammer.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile (i.e., Drawing 2) and cross-sections (i.e., Drawing 3) are inferred from observations of drilling progress, non-continuous sampling, and in situ testing and therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions encountered at the proposed bridge site consist of fill or a silt and sand deposit underlain by a deposit of clayey silt and/or an upper non-cohesive deposit of sandy silt to silt and sand to silty sand to sand. The non-cohesive deposit is underlain by bedrock on the north side of Commanda Creek, and a deposit of clayey silt to silty clay on the south side of Commanda Creek. On the south side of the creek, this cohesive deposit is underlain by a lower deposit of silt and sand to sand, which in turn is underlain by bedrock.

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

Detailed descriptions of the subsurface conditions encountered in the boreholes at this site are provided in the following subsections.

4.2.1 Asphalt

An approximately 200 mm thick layer of asphalt was encountered at the ground surface in Borehole 16-06 which was advanced on the west shoulder of Highway 524, north of the existing Commanda Creek Bridge.

4.2.2 Fill

A layer of fill was encountered below the asphalt in Borehole 16-06 and immediately below the ground surface in Boreholes 16-01, 16-03 and 16-04. Non-cohesive fill, comprised of silty sand, trace to some gravel, trace to some clay and trace organics, was encountered in Boreholes 16-03, 16-04 and 16-06. Cohesive fill, comprised of clayey silt, trace to some sand, trace organics, with wood pieces and rootlets, was encountered in Borehole 16-01. The top of the fill was encountered in the boreholes between Elevations 227.3 m and 226.8 m and the thickness of the fill varies between approximately 0.9 m and 3.0 m.

The SPT ‘N’-values measured within the cohesive fill range from 8 blows to 29 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency. Two SPT ‘N’-values measured in the non-cohesive fill in Boreholes 16-03 and 16-06 are 4 blows per 0.3 m of penetration, indicating a very loose relative density.

The result of a grain size distribution test carried out on one sample of the clayey silt fill recovered from Borehole 16-01 is shown on Figure B1 in Appendix B. An Atterberg limits test was carried out on one sample of the cohesive fill and measured a liquid limit of about 27 per cent, a plastic limit of about 19 per cent, and a plasticity index of about 8 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B2 in Appendix B, and indicates that the material is classified as a clayey silt of low plasticity. The natural water contents measured on two samples of the clayey silt fill recovered from Borehole 16-01 are about 22 per cent and 35 per cent. The natural water content measured on one sample of the silty sand fill recovered from Borehole 16-04 is about 20 per cent.

4.2.3 Surficial Silt and Sand

A surficial deposit of silt and sand, trace gravel, trace to some clay, trace organics was encountered at the ground surface in Borehole 16-02 which was advanced on the south side of Commanda Creek. The deposit is approximately 2.2 m thick, with its base encountered at approximately Elevation 224.9 m.

Two SPT ‘N’-values measured within the silt and sand deposit are 4 blows and 6 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

The result of a grain size distribution test carried out on a sample of the silt and sand deposit is shown on Figure B3 in Appendix B. An Atterberg limits test was carried out on a sample of the silt and sand deposit and measured a liquid limit of about 34 per cent, a plastic limit of about 24 per cent, and a corresponding plasticity index of about 10 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B4 in Appendix B, and indicates that the material is classified as a silt of low plasticity. The natural water content measured on a sample of the silt and sand deposit is about 27 per cent.

4.2.4 Surficial Clayey Silt

A surficial deposit of clayey silt, some sand was encountered below the silty sand fill in Borehole 16-06 which was advanced on the north side of Commanda Creek. The top of this deposit is at about Elevation 225.5 m and it is approximately 2.2 m thick.

The SPT 'N'-values measured within the surficial clayey silt deposit range between 3 blows and 7 blows per 0.3 m of penetration, suggesting a soft to firm consistency.

The result of a grain size distribution test carried out on a sample of the clayey silt deposit is in Figure B5 in Appendix B. Atterberg limits tests were carried out on two samples of the surficial clayey silt deposit and measured liquid limits of about 25 per cent, plastic limits of about 20 per cent, and corresponding plasticity indices of about 5 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B6 in Appendix B, and indicate that the material is classified as a clayey silt of low plasticity. The natural water contents measured on two samples of the clayey silt deposit are about 25 per cent and 31 per cent, near or above the liquid limit.

4.2.5 Upper Silt to Sand

An upper non-cohesive deposit comprised of silt to sandy silt to silt and sand to silty sand to sand was encountered in all six boreholes. Trace organics were encountered at various depths in Boreholes 16-02 to 16-06 as noted on the borehole records. An approximately 0.15 m thick layer of clayey silt was encountered in Borehole 16-02 at a depth of about 3.1 m below existing ground surface. The top of this deposit ranges between about Elevations 226.1 m and 223.3 m and the thickness of the deposit, where fully penetrated (i.e., in Boreholes 16-01 to 16-05), ranges between approximately 1.2 m and 6.6 m. Borehole 16-06 was terminated within this deposit at a depth of about 8.4 m below existing ground surface, upon auger refusal.

The SPT 'N'-values measured within silt to sandy silt to silt and sand to silty sand to sand deposit range from 0 blows (weight of hammer) to 16 blows per 0.3 m of penetration, indicating a very loose to compact relative density. One SPT 'N'-value of 0 blows (weight of rods) in Borehole 16-05 is considered unrepresentative due to an unbalanced hydrostatic head encountered during borehole advancement.

The results of grain size distribution tests carried out on seven samples of the silt to sandy silt to silt and sand portion of the non-cohesive deposit are shown on Figure B7A in Appendix B. The results of grain size distribution tests carried out on two samples of the silty sand portion of the non-cohesive deposit are shown on Figure B7B. Atterberg limits tests were carried out on five samples of this deposit. Two tests carried out on a sample of silt from Borehole 16-01 and sandy silt from Borehole 16-03 measured liquid limits of about 26 and 23 per cent, plastic limits of about 22 and 19 per cent, and plasticity indices of about 4 per cent. Both results of the Atterberg limits tests are shown on the plasticity chart on Figure B8 in Appendix B, and indicate that the material is classified as a silt / sandy silt of slight to low plasticity. Tests also carried out on three samples of the silt and sand to silty sand portions of the non-cohesive deposit indicate that that material is classified as non-plastic.

The natural water content measured on 16 samples of this deposit generally ranges between about 19 per cent and 33 per cent. A natural water content measured one sample recovered from Borehole 16-03 is about 46 per cent. The high water content can be attributed to the presence of organics at this depth. An organic content measured on a sample of the silty sand recovered from Borehole 16-05 is about 4.1 per cent by weight.

A consolidated drained direct shear test was also carried out on a sample of silty sand recovered from Borehole 16-05. The results are presented on Figure B9.

4.2.6 Clayey Silt to Silty Clay

A clayey silt to silty clay deposit was encountered underlying the upper non-cohesive deposit on the south side of Commanda Creek (i.e., in Boreholes 16-01, 16 02, and 16-03). An approximately 0.2 m thick interlayer comprised of silt, some sand, trace clay was encountered within this deposit in Borehole 16-01 at about Elevation 219.8 m. The top of the clayey silt to silty clay deposit ranges between about Elevation 223.1 m and 219.7 m and the overall thickness of the deposit varies between approximately 3.3 m and 6.2 m. The thickness of the deposit decreases moving northerly towards Commanda Creek, based on interpretation of the borehole results.

The SPT 'N'-values measured within the clayey silt to silty clay deposit range from 0 blows (weight of hammer) to 5 blows per 0.3 m of penetration. In situ field vane tests measured undrained shear strengths ranging from approximately 20 kPa to 58 kPa, with sensitivities ranging between about 3 and 19. The field vane test results indicate that the clayey silt to silty clay deposit has a soft to stiff consistency.

The result of a grain size distribution test carried out on a sample of the silt interlayer encountered within the clayey silt to silty clay deposit in Borehole 16-01 is shown on Figure B10 in Appendix B. Atterberg limits tests were carried out on eight samples of the clayey silt to silty clay deposit and measured liquid limits between about 22 per cent and 36 per cent, plastic limits between about 16 per cent and 24 per cent, and plasticity indices between about 5 per cent and 13 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B11 in Appendix B, and indicate that the material is classified as a clayey silt of low to plasticity to a silty clay of intermediate plasticity. The natural water contents measured on ten samples of the clayey silt to silty clay deposit range between about 26 per cent and 45 per cent. A natural water content measured on a sample of the silt interlayer within the deposit is about 26 per cent.

A laboratory consolidation test was carried out on one sample of the silty clay portion of the cohesive deposit obtained from a Shelby tube recovered from Borehole 16-03. A preconsolidation stress of about 135 kPa was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of about 17.5 kN/m^3 and a specific gravity of 2.74 were measured on the consolidation test sample. Details of the test results are shown on Figure B12 in Appendix B and the test results are summarized below.

Borehole and Sample No.	Sample Depth / Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	C_c	C_r	e_o	c_v [1] (cm^2/s)
Borehole 16-03 Sample 10	8.6 / 218.3	110	135	1.23	0.40	0.02	1.23	5.0×10^{-3}

Note:
1. For stress range between effective overburden stress and final stress due to 2.0 m high embankment, that is 110 kPa ≤ σ_{vo}' ≤ 155 kPa.

Where:

σ_{vo}' is the effective overburden stress in kPa

σ_p' is the preconsolidation stress in kPa

OCR is the consolidation ratio

C_c is the compression index

C_r is the recompression index

e_o is the initial void ratio

c_v is the coefficient of consolidation in cm^2/s

4.2.7 Lower Silt and Sand to Sand

A lower non-cohesive deposit of silt and sand to silty sand to sand was encountered underlying the clayey silt to silty clay deposit in Boreholes 16-01 to 16-03. The presence of cobbles within the lower silt and sand to sand was inferred below a depth of about 12.2 m in Borehole 16-02 due to difficulties experienced with advancing the casing using wash boring techniques. The top of this deposit ranges between Elevations 216.9 m and 216.4 m. The thickness of the deposit, where fully penetrated (i.e., in Boreholes 16-02 and 16-03), is about 3.4 m and 1.5 m, respectively; Borehole 16-01 was terminated within this deposit upon casing refusal after penetrating it for a thickness of 1.1 m, at a depth of about 12.0 m below ground surface.

The SPT 'N'-values measured within the lower silt and sand to sand deposit range from 0 blows (weight of rods) to 50 blows per less than 0.3 m of penetration, but on average range between about 17 blows and 24 blows per 0.3 m of penetration, indicating a typically compact relative density. The low SPT 'N'-values (i.e., weight of hammer) were measured immediately below the cohesive deposit in Boreholes 16-01 and 16-02 and are considered unrepresentative due to an unbalanced hydrostatic head encountered at these depths during the SPT sampling. The SPT 'N'-values of 50 blows for 0.15 m and 0.10 m of penetration measured in Boreholes 16-02 and 16-03 at the interface between the non-cohesive deposit and the bedrock are also considered unrepresentative of the soil deposit.

The result of a grain size distribution test carried out on a sample of the silty sand portion of the lower deposit is shown on Figure B13 in Appendix B. The natural water contents measured on three samples of the silt and sand to silty sand to sand deposit range between about 20 per cent and 28 per cent.

4.2.8 Gravel Layer

An approximately 0.1 m thick layer of gravel, trace to some sand was encountered in Borehole 16-04 at a depth of 7.5 m below ground surface, corresponding to Elev. 219.5 m. This gravel layer immediately overlies the bedrock as encountered at this location.

A SPT 'N'-value measured within the gravel layer is 50 blows per 0.10 m of penetration, indicating a very dense relative density.

4.2.9 Bedrock

Bedrock was encountered below the upper silt to sand deposit on the north side of the creek, and below the lower silt and sand to sand deposit on the south side of the creek in Boreholes 16-02, 16-03, 16-04 and 16-05. The approximate depths to top of bedrock below ground surface and corresponding top of bedrock surface elevations are summarized below and are shown on Drawings 2 and 3 and on the borehole/drillhole records in Appendix A.

Foundation Element	Borehole Designation	Approximate Depth to Bedrock Surface (m)	Approximate Bedrock Surface Elevation (m)	Remarks
South Approach	16-01	11.5	215.8	Inferred from casing refusal
South Abutment	16-02	13.9	213.2	Cored (4.7 m length)
	16-03	12.0	214.9	Cored (3.1 m length)
North Abutment	16-04	7.7	219.3	Cored (6.3 m length)
	16-05	6.5	219.5	Cored (3.1 m length)
North Approach	16-06	8.4	218.6	Inferred from auger refusal

Based on review of the bedrock core samples, the bedrock consists predominantly of granitic gneiss and is generally described as fresh, foliated, pink and black, medium-grained, non-porous and very strong. In Borehole 16-02, the granitic gneiss is interbedded with fresh, massive, black, fine-grained, non-porous, strong to very strong basalt. The bedrock details are presented on the drillhole records Appendix A. Bedrock core photographs are shown on Figures B14 to B17 in Appendix B.

The Total Core Recovery (TCR) measured on the recovered rock core samples ranges between 93 per cent and 100 per cent, and the Solid Core Recovery (SCR) ranges between 70 per cent and 100 per cent. The Rock Quality Designation (RQD) measured on the rock core samples ranges from about 86 per cent to 100 per cent, indicating a rock mass of good to excellent quality as per Table 3.10 of CFEM (2006)[1]. The RQD value measured on Run 2 of the rock core recovered from Borehole 16-03 is 72 per cent (indicating a rock mass of fair quality) and can be attributed to two broken core zones and discontinuities encountered between depths of about 14.2 m and 14.7 m below existing ground surface.

A total of three Unconfined Compression (UC) tests (ASTM D7012)[4] were carried out on selected samples of the granitic gneiss and basalt bedrock core recovered from Boreholes 16-02, 16-03 and 16-05. The UC test results are summarized below.

Foundation Element	Borehole Designation	Sample Depth / Elevation (m)	Rock Type	UCS (MPa)	Unit Weight (kN/m^3)
South Abutment	16-02	14.0 – 14.2 (213.1 – 212.9)	Basalt	98	29.4
	16-03	12.2 – 12.3 (214.7 – 214.6)	Granitic Gneiss	155	25.6
North Abutment	16-05	6.7 – 6.9 (219.3 – 219.1)	Granitic Gneiss	108	25.7

The test result are also shown on the drillhole records in Appendix A and summarized in Table B1 in Appendix B. Based on the laboratory UCS tests and in accordance with Table 3.5 of CFEM (2006)[1], the basalt and granitic gneiss are classified as strong and very strong, respectively.

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

[4] Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

4.3 Groundwater Conditions

A monitoring well was installed in Borehole 16-05 and details of the well installation and water level measurements are shown on the borehole record in Appendix A. The groundwater level measurements in the piezometer are summarized below.

Borehole Designation	Approximate Ground Surface Elevation (m)	Screened Depth / Elevation Interval (m)	Screened Deposits	Depth to Groundwater Level (m)	Groundwater Elevation (m)	Date of Measurement
16-05	226.0	2.8 to 9.6 (223.2 – 216.4) [1]	Silty Sand / Bedrock	2.1	223.9	November 23, 2016
				2.1	223.9	November 24, 2016
				2.1	223.9	November 25, 2016
				2.1	223.9	November 26, 2016
				2.2	223.8	December 16, 2016

Note:
1. The screened depth interval includes the filter sand above and below the screen.

The water levels observed inside the casing, hollow stem augers or open boreholes during or upon completion of drilling operations prior to wash boring or bedrock coring range from 2.2 m to 6.8 m below ground surface, corresponding to between Elevations 225.1 m and 220.1 m. These water levels, as noted on the borehole records, may not represent the longer-term, stabilized groundwater level at the site.

The groundwater levels are subject to seasonal fluctuations and precipitation events, and are expected to be higher during wet seasons and sustained periods of precipitation.

The surface of the water level in the Commanda Creek was measured by others at about Elevation 223.2 m on June 1, 2016.

4.4 Analytical Testing of Soil

Two soil samples were selected from Boreholes 16-03 and 16-04 and submitted to Maxxam Analytics of Mississauga, Ontario for corrosivity testing. The analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix C, and summarized below.

Borehole Designation	Sample No.	Average Approx. Sample Depth (m)	Average Approx. Sample Elevation (m)	Material Type	Resistivity (ohm·cm)	Conductivity (μohm/cm)	pH	Chloride (Cl) Content (ppm or μg/g)	Sulphate (SO_4) Content (ppm or μg/g)
16-03 [1]	SA 4	2.6	224.3	Silt and Sand	3,300	306	4.8	170	<20 [3]
16-04 [2]	SA 7	4.9	222.1	Silty Sand	3,800	266	4.7	100	60

Notes:
1. Borehole designated as 'BH-03' on the Certificate of Analysis.
2. Borehole designated as 'BH-04' on the Certificate of Analysis.
3. The chloride concentration is below the reportable detection limit of 20 μg/g.

5.0 CLOSURE

The field work for this investigation was supervised by Ms. Alysha Kobylinski, B.A.Sc., who also prepared this Foundation Investigation Report. The report was reviewed by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of the report.

Report Signature Page

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 524 COMMANDA CREEK BRIDGE REPLACEMENT
(SITE NO. 44-029)
PRINGLE TOWNSHIP, PARRY SOUND DISTRICT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5260-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed bridge replacement at Commanda Creek on the revised Highway 524 alignment (Site No. 44-029). These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the field investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and carry out the design of the bridge foundations. The foundation investigation report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor.

Contractors must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Based on the General Arrangement (GA) drawing prepared by MTO and provided to Golder by AECOM on November 29, 2016, it is understood that the proposed Commanda Creek replacement bridge will consist of a 28 m long single span, steel girder structure with integral abutments, to be constructed on a new alignment located approximately 10 m west of the existing highway centreline.

It is further understood that a grade raise of up to about 2.0 m and 2.2 m is proposed at the south and north approach embankments, respectively. The grade of the proposed bridge structure is at about Elevations 229. 0 m and 229.2 m at the south and north abutments, respectively.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* (CHBDC 2014) and its *Commentary*, the proposed bridge and foundation system is expected to carry relatively low to medium traffic volumes and its performance will have potential impacts on other transportation corridors, hence having a "typical consequence level" associated with exceeding limits states design. In addition, given the typical project-specific foundation investigation carried out at this site (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the *CHBDC* have been used for design.

6.3 Foundation Options

Given the presence of a thick deposit of very loose to loose silty/sandy soils and a deposit of generally soft to firm clayey silt to silty clay, a shallow foundation system is not recommended for the support of the abutments. Deep foundations will be required; viable deep foundation systems for the support of the proposed bridge abutments are as follows:

- **Driven steel H-piles or pipe piles:** Steel H-piles or pipe piles driven to refusal on bedrock / socketed into the bedrock are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only).

- **Drilled steel casings (small diameter):** Drilled steel casings installed with ring bits, using rotary percussive duplex and down-the-hole (DTH) hammer drilling methods are considered to offer the best chance of creating bedrock sockets in the strong to very strong bedrock at this site, provided that careful and controlled drilling practices are followed. Drilled steel casings typically range in diameter from about 305 mm to 750 mm; depending on the diameter of drilled steel casing adopted, single re-bar or a reinforcing bar cage would have to be lowered through the casing and into the rock socket prior to placement of concrete by tremie methods.

- **Drilled shafts/caissons (large diameter):** Drilled shafts (caissons) are considered feasible for the support of the abutments; however, this option would preclude integral abutment design. This option would be more expensive than driven pile foundations or smaller diameter drilled steel casings, although fewer caisson elements would be required in comparison to the number of driven steel piles or drilled steel casings that would be required. If caissons are adopted for support of the abutments, temporary liners will be required during construction to control potential ground losses in the water-bearing silt/sand soils.

Based on the above considerations, H-pile/pipe pile, drilled steel casing and drilled shaft (caisson) foundations are all considered feasible for support of the new abutments; however, as mentioned above, steel H-piles would permit integral abutment design, and are considered advantageous from this perspective. Given that a rock socket will be required at the north abutment (as discussed further in Section 6.4 below), an additional drilling/coring operation would be introduced in order to socket steel H-piles into the strong to very strong bedrock. Alternatively, it is noted that the drilling method associated with the smaller diameter drilled steel casing foundations would be more efficient, cost effective and reliable in penetrating the strong to very strong bedrock and creating a clean rock socket compared to drilled shaft foundations.

The advantages, disadvantages, relative costs and risks/consequences for the deep foundation options are summarized in Table 1.

6.4 Driven Steel H-Pile and Pipe Pile Foundations

6.4.1 Founding Elevations

Given that the steel H-piles / pipe piles will be relatively short at the north abutment (i.e., about 5.9 m long measured from the underside of the pile cap to the top of the bedrock surface), the piles should be socketed 1 m into the strong to very strong bedrock to ensure fixity of the pile toe. At the south abutment where the bedrock is deeper and the piles longer, the bedrock surface is expected to slope downwards to the south/west at an approximate angle of 26 degrees (or about 2 horizontal to 1 vertical); as such, the piles should be fitted with pile points to promote proper seating of the driven piles.

The underside elevation of the pile cap, the estimated pile tip elevation, and the founding stratum at the pile tip is provided below.

Foundation Element	Pile Type	Proposed Underside of Pile Cap (1)	Bedrock Surface Elevation	Estimated Design Pile Tip Elevation	Founding Stratum at Tip Elevation
South Abutment	HP 310x110 / HP 360x132 / 324 mm diameter pipe pile	225.2 m	214.9 m to 213.2 m	214.9 m to 213.2 m	Basalt and/or Granitic Gneiss (Bedrock)
North Abutment	HP 310X110 / HP 360x132 / 324 mm diameter pipe pile	225.2 m	219.5 m to 219.3 m	218.5 m to 218.3 m (placed within 1 m long rock socket; 0.6 m diameter)	Granitic Gneiss (Bedrock)

Note:
1. As per GA Drawing of the proposed bridge.

Based on the above elevations, the proposed piles are estimated to be up to approximately 10.3 m to 12 m long at the south abutment, and 6.7 m to 6.9 m long at the north abutment, respectively.

6.4.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances for steel HP 310x110 and HP 360x132 piles and closed-end, concrete-filled 324 mm (12-¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) are presented below. These values assume that piles at the north abutment are socketed a minimum 1 m into the strong to very strong bedrock, while piles at the south abutment are driven (with pile points) to refusal on bedrock.

Foundation Element	Pile Type	Approximate Length of Pile	Factored Ultimate Geotechnical Axial Resistance (at ULS)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement
South Abutment	HP 310X110 / 324 mm diameter pipe pile	10.3 m to 12 m	2,000 kN (1)	N/A (2)
	HP 360x132		2,400 kN (1)	N/A (2)
North Abutment	HP 310X110 / 324 mm diameter pipe pile	6.7 m to 6.9 m	2,000 kN (1)	N/A (2)
	HP 360x132		2,400 kN (1)	N/A (2)

Notes:
1. A factored ultimate geotechnical axial resistances (at ULS) of 2,000 kN and 2,400 kN represent structural limitations of the piles rather than geotechnical limitations.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

Pre-augering through the overburden followed by rock coring or churn drilling will be required at the location of the north abutment to create a 0.6 m diameter, 1 m long socket within the strong to very strong bedrock. A liner will need to be installed as the augering progresses in order to prevent the overburden soils from collapsing/sloughing into the pre-augered hole. The liner will need to be seated into the bedrock in order to prevent groundwater inflow into the drilled shaft and to allow flushing to clean the base of the rock socket. Balancing groundwater pressures during pre-augering by utilizing a head of water inside the liner may be required.

The base of the rock socket will have to be cleaned to remove all loose cuttings to ensure that the toe of the pile and the concrete within the socket (placed using tremie methods) is in intimate contact with the competent basalt or granitic gneiss bedrock. It is understood that based on the design developed to date, the liner will not be extracted from the ground and the annulus between the H-pile or the pipe pile and the liner above the rock socket will be backfilled in one of the following ways.

1) The entire annulus between the liner and the pile will be backfilled with loose, clean sand in accordance with the MTO Integral Abutment Bridges Manual (1996).

2) The first 2.9 m (approximately) above the rock socket will be backfilled with concrete using tremie methods, and the upper 3 m below the bottom of the pile cap will be backfilled with loose, clean sand in accordance with the MTO Integral Abutment Bridges Manual (1996).

As specified in the previous subsection, the piles at the north abutment should be fitted with pile points in order to ensure that the piles are seated properly into the sloping and strong to very strong bedrock. Where HP 310x110 piles are adopted, they should be fitted with Oslo points in accordance with OPSD 3000.201 (Steel HP 310 Oslo Point), or with Titus rock injector points. Where pipe piles are adopted, a Titus rock injector point (or equivalent) will be required.

6.4.3 Set Criteria

All pile installation/driving should be carried out in accordance with OPSS 903 (Deep Foundations). For piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

The following pile driving note should be added to the Contract Drawings (i.e., relevant note from Clause 3.3.3 of the Structural Manual (MTO, 2016)):

South Abutment:

- Piles to be fitted with pile points and driven to bedrock in accordance with OPSS 903.

North Abutment

- Piles to be installed following pre-augering/coring, in 0.6 m diameter, 1 m long cored sockets within the bedrock.

6.5 Drilled Steel Casing Foundations

6.5.1 Founding Elevations

The new abutments for the proposed bridge structure may also be supported on 600 mm diameter drilled steel casing socketed a minimum 1 m into the competent basalt or granitic gneiss bedrock, and filled with concrete.

The underside elevation of the pile cap, the estimated casing base elevation, and the founding stratum at the base of the casing is provided below.

Foundation Element	Proposed Underside of Pile Cap [1]	Bedrock Surface Elevation	Estimated Casing Base Elevation	Founding Stratum at Base Elevation
South Abutment	225.2 m	214.9 to 213.2	213.9 m to 212.2 m (1 m long rock socket; 0.6 m diameter)	Basalt and/or Granitic Gneiss (Bedrock)
North Abutment	225.2 m	219.5 to 219.3	218.5 m to 218.3 (1 m long rock socket; 0.6 m diameter)	Granitic Gneiss (Bedrock)

Notes:
1. As per GA Drawing of the proposed bridge.

Based on the above elevations, the proposed drilled steel casings are estimated to be up to approximately 13 m and 6.9 m long at the south and north abutments, respectively.

6.5.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances for a 0.6 m diameter drilled steel casing socketed a minimum 1 m into the strong to very strong bedrock and filled with concrete are presented below.

Foundation Element	Approximate Length of Casing	Factored Ultimate Geotechnical Axial Resistance (at ULS)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement
South Abutment	11.3 m to 13 m	11,000 kN (1)	N/A (2)
North Abutment	6.7 m to 6.9 m	11,000 kN (1)	N/A (2)

Notes:
1. Structural capacity of casing must be verified by the structural engineer.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

Given that the above drilled steel casing capacities have a significant end-bearing component, the performance of the casing in compression will depend to a large degree upon the final cleaning and verification of the condition of the bedrock at the base of the drilled rock socket. As such, the base of each drilled steel casing excavation must be cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the competent basalt and/or granitic gneiss bedrock. A qualified geotechnical engineer should be retained during construction to inspect the drilled casing to verify that the conditions encountered are consistent with the information obtained from the boreholes and to confirm the required minimum socket lengths and cleanliness. Visual remote inspection of the base of the casing can be accomplished by means of a shaft inspection device such as a video camera. The Contract Documents should include provisions for removal of groundwater / drilling mixture from within the casing to allow for inspection of the drilled casing. Should the camera inspection indicate that loosened material is present at the base of the caissons, the base would need to be cleaned and re-inspected.

6.6 Drilled Shaft (Caisson) Foundations

6.6.1 Founding Elevations

The new abutments for the proposed bridge structure may also be supported on drilled shafts (caissons) socketed a minimum of 1 m into the competent basalt or granitic gneiss bedrock and filled with concrete.

The underside elevation of the pile cap, the estimated caisson base elevation, and the founding stratum at the base of the caisson are summarized below.

Foundation Element	Proposed Underside of Pile Cap (1)	Bedrock Surface Elevation	Estimated Caisson Base Elevation	Founding Stratum at Base Elevation
South Abutment	225.2 m	214.9 m to 213.2 m	213.9 m to 212.2 m (1 m long rock socket; 0.9 m diameter)	Basalt and/or Granitic Gneiss (Bedrock)
North Abutment	225.5 m	219.5 m to 219.3 m	218.5 m to 218.3 m (1 m long rock socket; 0.9 m diameter)	Granitic Gneiss (Bedrock)

Notes:
1. As per GA Drawing of the proposed bridge.

Based on the above elevations, the proposed caissons are estimated to be up to approximately 13 m and 6.9 m long at the south and north abutments, respectively.

6.6.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) for a 0.9 m diameter caisson socketed 1 m into the strong to very strong bedrock and filled with concrete are presented below.

Foundation Element	Approximate Length of Caisson	Factored Ultimate Geotechnical Axial Resistance (at ULS)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement
South Abutment	11.3 m to 13 m	19,000 kN (1)	N/A (2)
North Abutment	6.7 m to 6.9 m	19,000 kN (1)	N/A (2)

Notes:
1. Structural capacity of caissons must be checked.
2. The geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

If drilled shafts are adopted, a permanent liner will be required to support the overburden soils from collapsing/sloughing during or after drilling operations. The liner will need to be seated into bedrock to prevent groundwater inflow into the drilled shaft and allow flushing to clean the base of the rock socket. Balancing groundwater pressures during construction by utilizing a head of water inside the permanent liner may also be required. In addition, given the presence of cobbles inferred within the silt and sand to sand deposit at the proposed south abutment, drilling equipment must be capable of penetrating through such obstructions. Further, placement of concrete by tremie methods would be required.

Given that the above drilled shaft capacities have a significant end-bearing component, the performance of the drilled shafts in compression will depend to a large degree upon the final cleaning and verification of the condition of the bedrock at the base of the drilled rock socket. As such, the base of each drilled shaft excavation must be cleaned to remove all loose cuttings to ensure that the concrete is in intimate contact with the competent basalt and/or granitic gneiss bedrock. A qualified geotechnical engineer should be retained during construction to inspect the drilled caissons to check that the conditions encountered are consistent with the information obtained from the boreholes and to confirm the required minimum socket lengths and cleanliness. To allow for visual remote inspection of the caisson bases, which can be accomplished by means of a shaft inspection device such as a video camera, the caisson excavations should be lined through the overburden. The liner must be maintained tight to the sides of the soil and seated into the bedrock to reduce seepage of water into the drilled excavations. The Contract Documents should include provisions for groundwater control to allow for inspection of the drilled caissons. Should the camera inspection indicate that loosened material is present at the base of the caissons, the base would need to be re-cleaned and re-inspected.

6.7 Frost Protection

The pile caps for all deep foundation elements (i.e., H-piles, pipe piles, drilled steel casings, and drilled shafts) should be provided with a minimum 1.9 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration Depths for Southern Ontario), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap.

If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.8 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (i.e., at the 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.

The resistance to lateral loading in front of a single pile/casing/shaft may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (*CFEM*, 1992 as referenced in the *Commentary of the CHBDC, 2014*):

for non-cohesive soils:

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

where:

n_h = coefficient related to soil density (kPa/m)
z = depth (m)
B = pile diameter or width (m)

and for cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where: s_u = undrained shear strength of the soil (kPa)
B = pile diameter or width (m)

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native overburden to be utilized for the structural analysis of the piles/casings/shafts at this site are summarized below.

Foundation Element (Relevant Borehole)	Soil Unit	Elevation	n_h	s_u
South Abutment (16-03)	Very Loose to Loose Sandy Silt to Silt and Sand	225.2 m to 219.7 m	10,000 kPa/m	--
	Firm Silty Clay	219.7 m to 216.4 m	--	27 kPa
	Compact Silt and Sand to Sand	216.4 m to 214.9 m	15,000 kPa/m	--
	Granitic Gneiss (Bedrock)	214.9 m to 212.9 m	k_h = 16,500 MN/m/m	
North Abutment (16-04)	Very Loose to Loose Silt and Sand to Silty Sand to Sand	225.15 m to 219.5 m	10,000 kPa/m	--
	Loose Sand Inside Permanent Liner (H-Pile or Tube Pile Options Only)	225.15 m to 222.15 m or 225.15 m to 219.3 m (1)	3,000 kPa/m	--
	Gravel	219.5 m to 219.3 m	19,000 kPa/m	--
	Granitic Gneiss (Bedrock)	219.3 m to 217.3 m	k_h = 16,500 MN/m/m	

Note:
1. Refer to Section 6.4.2 for details regarding backfilling options inside the permanent liner.

For a single H-pile, pipe pile filled with concrete, drilled steel casing filled with concrete, and drilled shaft, the estimated factored ultimate and serviceability geotechnical lateral resistances (at ULS and SLS for 10 mm of horizontal deflection at the pile caps) are presented below. These values are based on analyses carried out using the commercially available program LPILE Plus (Version 5.0), developed by Ensoft Inc.

Foundation Element	Deep Foundation Unit	Axial Load Applied at the Top of Pile	Factored Ultimate Geotechnical Lateral Resistance (at ULS)	Factored Serviceability Geotechnical Lateral Resistance (at SLS) for 10 mm of Deflection (2)
South Abutment	HP 310 x 110 driven (with pile point) to bedrock (1)	2,000 kN	210 kN	125 kN
	HP 360 x 132 driven (with pile point) to bedrock (1)	2,400 kN	230 kN	160 kN
	324 mm dia. tube pile driven (with pile point) to bedrock	2,000 kN	90 kN	110 kN
	0.6 m dia. drilled steel casing socketed 1 m into bedrock	5,000 kN	270 kN	310 kN
	0.9 m dia. caisson socketed 1 m into bedrock	9,000 kN	675 kN	640 kN

Foundation Element	Deep Foundation Unit	Axial Load Applied at the Top of Pile	Factored Ultimate Geotechnical Lateral Resistance (at ULS)	Factored Serviceability Geotechnical Lateral Resistance (at SLS) for 10 mm of Deflection (2)
North Abutment	HP 310 x 110 pile socketed 1 m into bedrock – liner backfilled with loose sand (1)	2,000 kN	160 kN	70 kN
	HP 360 x 132 pile socketed 1 m into bedrock – liner backfilled with loose sand (1)	2,400 kN	175 kN	90 kN
	HP 310 x 110 pile socketed 1 m into bedrock – liner backfilled with 2.9 m of concrete followed by 3 m of loose sand (1)	2,000 kN	235 kN	95 kN
	HP 360 x 132 pile socketed 1 m into bedrock – liner backfilled with 2.9 m of concrete followed by 3 m of loose sand (1)	2,400 kN	250 kN	110 kN
	324 mm dia. tube pile socketed 1 m into bedrock – liner backfilled with loose sand	2,000 kN	80 kN	65 kN
	324 mm dia. tube pile socketed 1 m into bedrock – liner backfilled with 2.9 m of concrete followed by 3 m of loose sand	2,000 kN	85 kN	70 kN
	0.6 m dia. drilled steel casing socketed 1 m into bedrock	5,000 kN	270 kN	305 kN
	0.9 m dia. caisson socketed 1 m into bedrock	9,000 kN	675 kN	585 kN

Notes:
1. Steel H-pile oriented for strong-axis bending.
2. Analyses assume a fixed-head condition.

Based on the above, both the structural and geotechnical resistances of the piles/casings/shafts should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles/casings/shafts will be controlled by deflections and the horizontal resistance of the pile/casing/shafts should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (see Section C6.11.2.2.2 of the *Commentary to the CHBDC, 2014*).

The upper zone of the soil (down to a depth below the pile cap equal to about 1.5$\cdot B$ (after Broms, 1964, where B is the pile/casing/caisson diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also be considered when the spacing in the direction of loading is less than eight (8) pile diameters between rows of piles/casings/shafts. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (U.S. Navy, 1986), 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/casing/shafts spacing in between those listed above.

6.9 Lateral Earth Pressures

The lateral earth pressures acting on the abutment stem 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 abutment walls. 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 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 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- 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* (2014) Section 6.12.3 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 1.9 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC, 2014*). For unrestrained walls, 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, 2014*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Soil 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

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). 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 C6.12.1 and Table C6.6 of the Commentary to the *CHBDC, 2014*.

6.10 Approach Embankment Design

Based on the GA Drawing of the proposed bridge, the proposed grade along Highway 524 at the proposed Commanda Creek bridge will be at approximately Elevation 229.1 m, requiring placement of up to about 2 m and 2.2 m of fill to raise the south and north approach embankment grades, respectively.

6.10.1 Global Stability

The following subsections outline the method used to evaluate static global stability of the proposed approach embankments. The geotechnical soil parameters used in the analyses are also presented. The results of the stability analyses are presented in Section 6.10.3 where they are discussed together with the results of the settlement analyses and recommendations regarding possible design and construction alternatives to mitigate post-construction settlement.

6.10.1.1 Method of Analysis

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_{gu}. (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, minimum Factors of Safety of 1.3 and 1.5 have been used for the design of the embankment slopes for the temporary and permanent conditions, respectively, as per Table 6.2 of CHBDC (2014).

6.10.1.2 Parameter Selection

The simplified stratigraphy together with the associated unit weights and foundation engineering parameters employed for the different native soil types at the approach embankments are summarized in Table 2. The following is a summary of the embankment slope inclination, unit weight and effective friction angle for the new Select Subgrade Material (SSM), new granular fill, and lightweight fill (i.e., Expanded Polystyrene (EPS) blocks) modelled in the slope stability analyses.

Fill Type	Recommended Slope Inclination	Unit Weight, γ	Effective Friction Angle, ϕ'	Cohesion, c'
SSM [1]	2H:1V	20 kN/m^3	30°	0 kPa
Granular Fill [1]	2H:1V	21 kN/m^3	35°	0 kPa

Fill Type	Recommended Slope Inclination	Unit Weight, γ	Effective Friction Angle, ϕ'	Cohesion, c'
Lightweight Fill (EPS)	2H:1V	0.5 kN/m^3	0°	15 kPa

Note:

1. The effective friction angle of the SSM is lower compared to the granular fill. As such, approach embankments constructed with SSM represent the worst case scenario in terms of global slope stability of the embankments. All slope stability figures presented in this report illustrate embankments using SSM.

For the non-cohesive soils present at this site, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in situ field vane shear tests, inferred from the laboratory consolidation test results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content, liquid limit, etc.), where appropriate.

Effective stress parameters were also employed to evaluate the stability of the embankments based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle (ϕ') and effective cohesion (c')) for the cohesive deposits were estimated from empirical correlations based on the plasticity index. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 223.8 m, which is based on the piezometric groundwater level measured in Borehole 16-05.

6.10.2 Settlement

The following subsections outline the methods used to carry out the settlement analyses at the proposed approach embankments for the realigned highway. The results of the analyses are presented in Section 6.10.3 where they are discussed together with the results of the stability analyses and recommendations regarding potential design and construction alternatives to mitigate stability issues and/or post-construction settlement.

6.10.2.1 Method of Analysis

To estimate the magnitude of expected settlement, analyses were carried out at the south and north approach embankments. Settlement analyses were carried out using the commercially available program Settle^{3D} (Version 2.0), developed by Rocscience Inc.

The sources of settlement are considered to include:

- Immediate settlement of the granular soils (short-term);
- Primary time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory – long-term); and
- Secondary time-dependent (creep) consolidation of the cohesive deposits (long-term).

The thickness of the compressible foundation soils and the height of the approach embankments vary along the proposed Highway 524 alignment, and as such the settlements along the length of a highway alignment will similarly vary; however, it should be noted that the settlements estimated from the settlement analysis represent the maximum anticipated value along a given section of the highway alignment.

6.10.2.2 Parameter Selection

The simplified stratigraphy together with the associated deformation and time-rate consolidation parameters employed for the different native soil types at the approach embankments are given in Table 2.

The immediate compression of the non-cohesive deposits (i.e., silt, sandy silt, silt and sand, silty sand, sand, and gravel) were 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). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in Section C6.9.3.6 of the *Commentary to the CHBDC* (2014) and adjusted, if necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation test, where appropriate, and in situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the laboratory index tests were employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Azzouz et al. (1976), Koppula (1986), Kulhawy and Mayne (1990), Nishida (1956) and Terzaghi and Peck (1967).

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests and/or estimated from the U.S. Navy (1986) correlation with liquid limit assuming normally consolidated or over consolidated soils, as applicable.

In addition to primary consolidation within the cohesive deposits (i.e., clayey silt to silty clay), secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress.

For the purpose of the stability analysis, the groundwater level was assumed to be at Elevation 223.8 m, which is based on the piezometric groundwater level measured in Borehole 16-05.

6.10.2.3 Settlement Performance Requirements

The settlement performance criterion for design of approach embankments is in accordance with MTO’s “Embankment Settlement Criteria for Design”, dated July 2, 2010. In general, embankments approaching structural elements such as a bridge abutment are to be designed as follows:

- Total settlements and differential settlement rates are not exceed 25 mm, over a 15-year period following completion of construction for a secondary highway.

6.10.3 Results of Analyses – South Approach Embankment

The stability analysis for the south approach embankment indicates that after completion of construction, the embankment will have a Factor of Safety greater than or equal to 1.3 and 1.5 during the temporary and permanent conditions, respectively, for deep-seated, global failure surfaces of the front slope and side slopes, that would impact the operation of the highway (see Figures 1A, 1B, 2A and 2B). These analyses and factors of safety are based on an approximately 2 m high embankment comprised of SSM constructed following sub-excavation of the existing fill/silt and sand deposit to Elevation 225.5 m (extending 20 m south of the south abutment) and replacement with SSM.

Based on the results of the settlement analysis (with the existing fill and near-surface deposit of silt and sand encountered in Borehole 16-02 sub-excavated to Elevation 225.5 m and replaced with SSM or granular fill), the settlement of the foundation soils under the loading imposed by a 2 m high granular fill embankment is estimated to be about 160 mm. The estimated total settlement is comprised of about 75 mm of immediate settlement due to compression of the non-cohesive deposit and about 85 mm of primary consolidation of the cohesive deposit.

The magnitude of secondary consolidation (creep) settlement for the cohesive deposit is estimated to be about 18 mm per log-cycle of time for this area, corresponding to about 30 mm over a 15-year period following completion of construction.

Based on an average coefficient of consolidation (c_v) of about 6.0 x 10^{-3} cm^2/s estimated for the 4.9 m thick clayey silt to silty clay deposit (i.e., thickest portion of the cohesive deposit bounded by non-cohesive deposits/interlayers) and assuming two-way drainage for the cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 100 days.

To reduce the post-construction settlement of the south approach embankment, the alternative mitigation options presented below could be considered. The alternatives described have been evaluated and ranked on the basis of the advantages, disadvantages, relative costs and risk/consequences and are summarized in Table 3. All settlement mitigation alternatives, except for the aggregate pier alternative, assume sub-excavation of the existing fill/silt and sand deposit to Elevation 225.5 m (extending 20 m south of the south abutment) and replacement with SSM or granular fill. Considering that there are no stability issues associated with a 2.5 m high surcharge embankment (consisting of a 2.0 m high embankment and 0.5 m of surcharge), and in order to minimize the post-construction settlement, surcharging is considered as the preferred alternative for this area from a foundations perspective, assuming that the construction schedule can accommodate the recommended delay period. It is noted that full sub-excavation of the clayey silt to silty clay would eliminate the time-dependent settlement of the cohesive deposit; however, given that the cohesive deposit extends to a depth of up to about 10.5 m below existing ground surface, as well as environmental concerns associated with deep excavation near Commanda Creek, this alternative is considered unfeasible and is not discussed further herein.

6.10.3.1 Preloading

Based on the estimated coefficient of consolidation (c_v) of about 6.0x 10^{-3} cm^2/s for the cohesive deposit, it is estimated that 90 per cent of primary consolidation settlement of the foundation soils under the final approach embankment height will be completed in about 100 days. However, in order to meet the settlement performance criterion, a minimum preload period of 250 days would be required.

Considering the length of the preload period, if this alternative is to be adopted, the magnitude and time-rate of settlement during and after construction of the preload embankment should be assessed by a monitoring program consisting of settlement plates (SPs) to confirm the end of the preload period.

Given the long duration required for the preloading mitigation option, this alternative is not ranked as the preferred alternative. However, if the construction schedule can accommodate this preload period, preloading the foundation soils for a duration of 250 days could be considered for this area.

6.10.3.2 Surcharging

In order to reduce the long preload period to achieve the post-construction settlement criterion, consideration could be given to surcharging the south approach embankment.

The stability analysis for the up to about 2.5 m high surcharge embankment (consisting of a 2.0 m high embankment and 0.5 m of surcharge) indicates that after completion of construction, the surcharge embankment will have a Factor of Safety greater than or equal to 1.3 during the temporary condition, for deep-seated, global failure surfaces of the front slope and side slopes, that would impact the operation of the highway (see Figures 3 and 4). Similar to the base case above, this assumes that sub-excavation of the existing fill and the native deposit of surficial silt and sand (encountered in Borehole 16-02) to Elevation 225.5 m, followed by replacement with SSM or granular fill.

Based on the estimated coefficient of consolidation (c_v) of about 6.0x 10^{-3} cm^2/s for the cohesive deposit, it is estimated that 90 per cent of primary consolidation settlement of the foundation soils under the final approach embankment height will be completed in about 100 days. However, in order to meet the settlement performance criterion, a minimum surcharge period of 120 days would be required.

Considering the length of the surcharge period, if this alternative is to be adopted, the magnitude and time-rate of settlement during and after construction of the surcharge embankment should be assessed by a monitoring program consisting of settlement plates (SPs) to confirm the end of the surcharge period.

Given the reduced delay period associated with this option compared to the preload period, surcharging is ranked as the preferred mitigation option for this area.

6.10.3.3 Lower Grade and Surcharging

In order to reduce the magnitude of the consolidation settlement of the thick cohesive deposit and the associated long preload period to achieve the post-construction settlement criterion, consideration could be given to lowering the final grade of the highway by 0.5 m, and placing a 1 m high surcharge during the preloading period.

The stability analysis for the up to about 2.5 m high surcharge embankment (consisting of a 1.5 m high embankment and 1 m of surcharge) indicates that after completion of construction, the surcharge embankment will have a Factor of Safety greater than or equal to 1.3 during the temporary condition, for deep-seated, global failure surfaces of the front slope and side slopes, that would impact the operation of the highway (see Figures 5 and 6). Similar to the base case above, this assumes that sub-excavation of the existing fill and the native deposit of surficial silt and sand (encountered in Borehole 16-02) to Elevation 225.5 m, followed by replacement with SSM or granular fill.

Based on the estimated coefficient of consolidation (c_v) of about 6.0x 10^{-3} cm^2/s for the cohesive deposit with a surcharge fill 1 m thick, a minimum surcharge period of 50 days would be required to meet the settlement performance criterion.

Considering the length of the surcharge period, if this alternative is to be adopted, the magnitude and time-rate of settlement during and after construction of the surcharge embankment should be assessed by a monitoring program consisting of settlement plates (SPs) to confirm the end of the surcharge period. The supply and installation of the SPs as well as monitoring of the instruments should be carried out in accordance with the specifications provided in Appendix D.

Given the need for increased handling of surcharge fill upon completion of the surcharge period and redesign of the bridge due to lower vertical clearance between the creek and the underside of the bridge deck, this alternative is not considered as the preferred foundation mitigation option. However, if the schedule cannot accommodate the required delay period associated with the recommended surcharge option, consideration could be given to lowering the grade of both the north and south approach embankments and surcharging for a duration of 50 days.

6.10.3.4 Surcharging and Lightweight Fill

Consideration could be given to constructing the south approach embankment with lightweight (i.e., EPS) fill to mitigate the post-construction settlement. For the up to about 2 m high embankment with 2H:1V side slopes (consisting of an approximately 275 mm thick granular base/levelling pad, a 1 m thick EPS core, a 125 mm thick reinforced concrete pad, and 600 mm thick pavement structure (base and subbase)), stability analysis indicate that embankment will have a Factor of Safety greater than or equal to 1.5 during the permanent condition, for deep-seated, global failure surfaces of the front slope and side slopes, that would impact the operation of the highway (see Figures 7 and 8).

Based on the results of the settlement analysis, the total settlement of the foundation soils under the loading imposed by the combined EPS and granular fill is estimated to be about 95 mm. The estimated total settlement is comprised of about 40 mm of immediate settlement due to compression of the non-cohesive deposits and about 55 mm of primary and secondary consolidation settlement of the cohesive deposit.

In order to meet the settlement performance criterion, it is estimated that the footprint of the south approach embankment would have to be first surcharged by a 2 m high SSM or granular fill embankment for a period of 20 days. After the delay period, the surcharge embankment would be removed for the placement of a levelling pad and installation the lightweight fill (i.e., EPS blocks).

It is recommended that the magnitude and time-rate of settlement, during and after construction of the surcharge embankment should be assessed by a monitoring program consisting of settlement plates (SPs) to confirm the end of the surcharge period.

Despite a relatively small volume of EPS, the construction of the surcharge embankment and the subsequent incorporation of lightweight fill into the final embankment would result in a higher construction cost as compared to the preloading option. However, if the schedule cannot accommodate the required delay period associated with the other settlement mitigation options, consideration could be given to this option to expedite the construction operations.

6.10.3.5 Wick Drains

Due to the long duration to complete primary consolidation associated with the south approach embankment, consideration could be given to the use of wick drains to expedite the consolidation settlement process in the thick cohesive deposit. It should be noted that with any wick drain design the magnitude and time rate of settlement as well as dissipation of excess pore pressures during and after construction of the embankment should be assessed by monitoring to confirm the end of the preload period. Monitoring instrumentation would consist of settlement plates/rods (SPs), vibrating wire piezometers (VWPs) and standpipe piezometers (SPPs).

Preliminary analyses using a wick drain spacing of 1 m in a triangular pattern to full depth through to the bottom of the cohesive deposit (i.e., up to a depth of about 10.5 m below existing ground surface) indicate that a preload period of 25 days would be required in order to meet the settlement performance criterion.

Given the need for a detailed wick drain investigation and design, this alternative is not considered as the preferred foundation mitigation option. However, if the schedule cannot accommodate the required delay period associated with the recommended surcharge option, consideration could be given to preloading in combination with wick drains for a duration of 25 days, subject to further investigation and detail design.

6.10.3.6 Aggregate Piers

Given the long delay period required to complete primary consolidation of the cohesive deposit under the loading of the south approach embankment and to satisfy the post-construction settlement criterion, consideration could be given to the installation of aggregate piers through the foundation soils below the footprint of the embankment. It should be noted that a temporary casing will be required for aggregate pier construction due to the very loose to loose nature of the non-cohesive deposits, relatively soft nature of the cohesive deposit, and the high water table. Alternatively, the use of displacement-type aggregate piers may be possible.

Given that the aggregate piers act as rigid inclusions and densify the native overburden to create a stiff composite soil mass, the stability of the south approach embankment will be improved compared to its in situ state.

Preliminary analyses using an aggregate pier spacing of 2 m in a triangular pattern for the full depth through the bottom of the clayey silt to silty clay deposit and founded on the bedrock (i.e., up to about 14 m below existing ground surface) indicates that a delay period of about 15 days would be required to meet the settlement performance criterion of 25 mm of settlement over a 15-year period following completion of construction. The magnitude of the remaining time-dependent settlement of the cohesive deposit within the aggregate pier-reinforced zone is estimated to be about 10 mm.

Given the need for a detail design for the aggregate pier system and the cost and time associated with construction of aggregate piers, the aggregate pier option is very expensive and not considered as the preferred settlement mitigation alternative.

6.10.4 Results of Analyses – North Approach Embankment

The stability analysis for north approach embankment indicates that after completion of construction, the embankment will have a Factor of Safety greater than or equal to 1.3 and 1.5 during the temporary and permanent conditions, respectively, for deep-seated, global failure surfaces of the front slope and side slopes, that would impact the operation of the highway (see Figures 9A, 9B, 10A and 10B). These analyses and factors of safety are based on an approximately 2.2 m high embankment comprised of SSM, constructed following sub-excavation of the existing fill to Elevation 225.5 m (extending 20 m north of the north abutment) and replacement with SSM.

Based on the results of the settlement analysis (with the existing fill sub-excavated and replaced with SSM or granular fill), the settlement of the foundation soils under the loading imposed by a 2.2 m high fill embankment is estimated to be about 115 mm. The estimated total settlement is comprised of about 65 mm of immediate settlement due to compression of the non-cohesive deposit and about 50 mm of primary consolidation of the cohesive deposit.

Based on an average coefficient of consolidation (c_v) of about 5.0 x 10^{-2} cm^2/s estimated for the 2.2 m thick clayey silt deposit and assuming two-way drainage for the cohesive deposit, it is estimated that about 90 per cent of the primary consolidation settlement will be completed in about 5 days.

Given that the clayey silt deposit is considered to be highly over-consolidated and the load imposed by the proposed approach embankment is low, the magnitude of total secondary consolidation (creep) settlement for the cohesive deposits is expected to be negligible.

In order to minimize post-construction settlements and satisfy the settlement performance criterion, a minimum preload period of 10 days is recommended.

6.11 Liquefaction Potential Below Embankments

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e. leading to potentially large surface deformations) and under undrained conditions generate excess pore water pressures. The excess pore water pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e. analogous to a slope failure) often referred to as "lateral spreading" or under certain conditions even catastrophic failure of the slope often referred to as "flow slides". Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with that presented in the *Commentary to the CHBDC, 2014*. It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using in-situ testing data collected at the borehole locations. The design groundwater level was determined based on the measured groundwater level in the standpipe piezometer installed in Borehole 16-05 at about Elevation 223.8 m. The CRR with depth was calculated at each borehole location using the parameter, $(N_1)_{60cs}$, that is based on the SPT 'N'-value obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment indicate that the silts and sands at the site are not considered liquefiable during the 2,475-year design earthquake.

6.12 Construction Considerations

The following subsections identify construction considerations that may impact the design and construction of the proposed bridge replacement Commanda Creek.

6.12.1 Open-Cut Excavations

The existing fill and the native surficial silt and sand at the south abutment is weak and contains organics up to Elevation 225.5 m, corresponding to a depth of up to about 1.8 m below existing ground surface, and should be sub-excavated and replaced with granular fill. In addition, construction of the new pile caps at both the north and south abutments is also expected to require excavations to a depth of about 2 m below the existing ground surface.

All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

The soils to be excavated can be classified according to OHSA as follows (assuming the groundwater level is below the foundation subgrade level):

- Silty sand or clayey silt fill – Type 3;
- Very loose to loose sandy silt to silt and sand to silty sand to sand – Type 3; and,

- Soft to firm clayey silt – Type 3.

Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes no steeper than 1H:1V based on the soil profile. However, if water inflow is observed, flatter slopes and dewatering measures may need to be implemented. Temporary excavations should be observed and reviewed during construction to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, a geotechnical engineer should review the excavation plan considering the conditions at that time.

6.12.2 Embankment Construction

Placement of Select Subgrade Material (SSM) or granular fill (satisfying OPSS.PROV 1010 SSM or Granular 'B' Type I or Type II requirements) above the water table for construction of new embankments (including backfilling operations) should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (Grading) and the SSM or granular fill should be compacted in accordance with OPSS.PROV 501 (Compacting). Inspection and field testing should be carried out by a qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are being achieved. Side slopes for the SSM or granular fill roadway embankment should be no steeper than 2H:1V.

6.12.3 Erosion Protection

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments. In the short term, if placement of cover material cannot be carried out soon after the construction of the embankments, erosion control blankets should be installed to minimize erosion of the embankment slopes. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).

6.12.4 Temporary Protection Systems

Temporary protection systems will be required to facilitate the construction of the new abutments/approaches and the removal of the existing modular bridge foundations. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the contractor.

6.12.5 Control of Groundwater and Surface Water

Given that the groundwater level measured in the standpipe piezometer installed at Borehole 16-05 is at about Elevation 223.8 m (which is consistent with the water level measured by others in the Commanda Creek at about Elevation 223.2 m on June 1, 2016) and that the excavations for the pile caps will extend to about Elevation 225.2 m, control of groundwater may only be required during wet periods of the year. Where required, pumping from within trenches/ditches with adequately sized and properly filtered sumps will be sufficient to control the groundwater inflow.

Surface water should be directed away from the excavations at all times

6.12.6 Control of Ground and Groundwater during Pre-Augering for Steel Pile Installation, or Drilled Shaft (Caisson) Construction

As discussed in Sections 6.4 and 6.6, disturbance (i.e., running or flowing) of the water-bearing non-cohesive soil deposits could occur during or after pre-augering/coring operations for the formation of bedrock sockets at the north abutment, or for drilling of caissons (if adopted). For socketing at the north abutment, or if drilled shaft foundations are adopted, permanent caisson liners with a balancing head of water will be required to support the overburden soils, balance groundwater pressures during construction, and seat the liners within the bedrock. In addition, placement of concrete by tremie methods would be required.

6.12.7 Obstructions

The presence of cobbles within the lower non-cohesive deposit was inferred between depths of 12.2 m and 13.9 m at the proposed south abutment. The presence of such obstructions could affect the construction of deep foundations. However, if drilled steel casing is selected as the preferred foundation alternative, it is recommended that rotary percussive duplex and Down-the-Hole (DTH) hammer drilling methods be utilized for the installation of the drilled steel casing. This drilling method is very effective in advancing the casing through obstructions such as cobbles and into the strong to very strong bedrock.

6.12.8 Analytical Testing of Construction Materials

The results of analytical tests carried out on two samples of the silt and sand to silty sand deposit are presented in Section 4.4 and on the Certificate of Analysis in Appendix C. The analytical test results were compared to CSA A23.1 Table 3 (Additional requirements for concrete subjected to sulphate attack) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured on the soil samples are less than 0.002 per cent and 0.006 per cent, which is below the moderate degree of exposure (i.e., below the class S-3 exposure limits). Therefore, based on the two soil samples tested, when the designer is selecting the exposure class for the concrete structure, the effects of sulphates from within the non-cohesive deposit in contact with the pile cap and any portion of the proposed structure constructed below the ground surface may not need to be considered. However, given that the proposed structure will most likely be exposed to de-icing salt, consideration should be given by the designer to designing the concrete structure for a “C” type exposure class as defined by CSA A23.1 Table 1.

The analytical test results of the soil samples were also compared to Table 7.1 (Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)), as presented in the Federal Highway Administration/National Highway Institute Publication No. FHWA-NHI-14-007 (Federal Highway Administration, 2015), to assess the relative level of corrosion potential on buried steel in contact with soil. The resistivity values measured on the soil samples are 3,300 ohm·cm and 3,800 ohm·cm. These results indicate a “moderately corrosive” potential.

It is also noted that the measured pH levels are below 5, suggesting the presence of acidic soils.

Ultimately, it is the designer’s decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) are satisfied.

7.0 CLOSURE

This report was prepared by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer, and reviewed by Mr. Christopher Ng, P.Eng., a geotechnical engineer and Associate with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

Report Signature Page

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OPSS.PROV 206	Construction Specification for Grading
OPSS 405	Construction Specification for Pipe Subdrains
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover

OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD):

OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

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TABLES

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – COMMANDA CREEK BRIDGE

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread/strip footings founded on native deposit or "perched" within the approach embankments on a Granular 'A' pad	• Given the presence of thick deposits of very loose to loose silty/sandy soils and a deposit of generally soft to firm clayey silt to silty clay, strip/spread footings founded on native soils are not considered a feasible foundation alternative. • Furthermore, given that the approach embankments are only up to about 2.2 m high and are underlain by these weak soils, which would undergo further settlement due to the combined embankment and footing loading, strip/spread footings "perched" within the approach embankments are also not considered a feasible foundation alternative.				
Steel H-piles (HP 310x110 or HP 360x132) or steel pipe piles (0.324 m diameter) driven (with pile points) to refusal on bedrock at the south abutment and socketed 1 m into bedrock (0.6 m diameter) at the north abutment	1 (H-piles only if integral abutment design is required)	• Conventional construction methods for driven H-pile and pipe pile foundations at the south abutment. • Pile caps can be constructed within the approach embankment fill or near the existing ground surface (assuming frost protection requirements are satisfied) so that foundation excavations can be minimized or will not be required. • Allows for integral abutment design (H-piles only)	• Piles at the south abutment need to be fitted with pile points. • Liners and pre-augering required at the north abutment, in conjunction with coring/churn drilling/down-hole hammer to create rock sockets for the piles. • Tremie placement of concrete required for the rock socket. • Tremie placement of concrete and/or sand within the permanent liners above the bedrock socket.	• Lower relative cost than drilled steel casing and drilled shafts/caissons. • Additional cost for pile points at the south abutment. • Additional cost for liners, pre-augering, coring/churn drilling or DTH hammer, and concrete / sand at the north abutment.	• Low risk of not achieving proper seating of piles on the sloping hard bedrock surface at the south abutment due to use of pile points and relatively shallow slope (based on interpolation of borehole results). • No risk of pile toes "kicking-out" at the north abutment due to socketing of piles into the bedrock.
Drilled steel casing using rotary percussive duplex and DTH hammer drilling system (0.6 m diameter socketed 1 m into bedrock)	2	• Reduced number of deep foundation elements compared to steel H-piles / tube piles due to higher axial and lateral geotechnical resistances compared to H-piles / pipe piles. • Eliminates the requirement for separate operations at the north abutment associated with pre-augering and formation of rock socket.	• Precludes use of integral abutments. • Requires specialized drilling equipment.	• Higher relative cost than driven H-piles / pipe piles. • Additional cost for specialized drilling equipment.	• Negligible risk of not being able penetrate casing into sloping hard bedrock surface. • Low risk of not being able to clean out the base of the rock socket and achieving geotechnical resistances. • Lowest likelihood of challenges with rock socket formation as compared to larger diameter sockets for H-

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – COMMANDA CREEK BRIDGE

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risk / Consequences
		• Temporary liners are not required during casing advancement. • Drilling method is effective in flushing out cuttings from the hole. • Drilling method can readily penetrate cobbles and boulders in overburden, and strong to very strong bedrock.			piles or pipe piles at the north abutment, or for drilled shafts.
Drilled shafts (0.9 m diameter socketed 1 m into bedrock)	3	• Conventional construction methods for drilled shaft/caisson foundations. • Higher axial and lateral geotechnical resistances compared to driven H-piles / pipe piles.	• Precludes use of integral abutments. • Requires a thorough cleaning and inspection of the base of the rock socket. • Temporary or permanent liners will be required, plus special measures such as tremie placement of concrete. • Potential difficulties creating sockets within the strong to very strong granitic gneiss bedrock, especially where the bedrock surface is sloping.	• Higher relative cost than driven H-piles / pipe piles or drilled steel casings.	• Low to moderate risk of difficulties / uncertainties with cleaning the base of the rock socket – risk of reduced axial geotechnical resistance. • High risk of difficulties with seating the steel liner into the strong to very strong bedrock, especially where the bedrock surface is sloping.

TABLE 2 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS

Foundation Investigation Area (Relevant Boreholes)	Stratigraphic Unit	Top Elevation (m)	Thickness (m)	γ' (kN/m^3)	ϕ' (°)	c' (kPa)	s_u (kPa)	σ_p' (kPa)	e_o	C_c	C_r	m_v (kPa^{-1})	E' (MPa)	c_v (cm^2/s)
South Abutment and Approach Embankment (Boreholes 16-01 to 16-03)	Clayey Silt Fill	~227.3	~3.0	19	32	0	50 to 100 [1]	--	--	--	--	--	--	--
	Silty Sand Fill	~226.9	~1.1	19	30	0	--	--	--	--	--	--	--	--
	Surficial Silt and Sand (Slight Plasticity)	~227.1	~2.2	19	30	0	--	--	--	--	--	--	5	--
	Upper Silt to Sand	225.8 to 224.3	1.2 to 6.1	19	32	0	--	--	--	--	--	--	3	--
	Clayey Silt to Silty Clay	223.1 to 223.1	3.3 to 6.2	17.5	34	0	27	125	1.23	0.40	0.02	--	--	6.0×10^{-3}
	Silt (Interlayer)	~219.8	~0.2	18	29	0	--	--	--	--	--	--	5	--
	Lower Silt and Sand to Sand	216.9 to 216.4	1.1 to 3.4	19	34	0	--	--	--	--	--	--	10	--
	Basalt and Granitic Gneiss Bedrock	215.8 to 213.2	--	26	--	--	--	--	--	--	--	--	--	--
North Abutment and Approach Embankment (Boreholes 16-04 to 16-06)	Silty Sand Fill	227.0 to 226.8	0.9 to 1.3	19	30	0	--	--	--	--	--	--	--	--
	Clayey Silt	~225.6	~2.2	18	36	0	50	225	--	--	--	5.0×10^{-4}	--	5.0×10^{-2}
	Upper Silt and Sand to Sand	226.1 to 223.3	4.7 to 6.6	19	32	0	--	--	--	--	--	--	3	--
	Gravel	~219.5	~0.2	22	34	0	--	--	--	--	--	--	--	--
	Granitic Gneiss Bedrock	219.5 to 218.6	--	26	--	--	--	--	--	--	--	--	--	--

Note:

1. The undrained shear strength is estimated to be 50 kPa above Elevation 225.8 m and 100 kPa below Elevation 225.8 m.

TABLE 3 – EVALUATION OF SETTLEMENT MITIGATION OPTIONS – SOUTH APPROACH EMBANKMENT

Settlement Mitigation Option [1]	Rank	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Surcharging (0.5 m high for 120 days)	1	• Standard construction operation. • Reduced time to reach post-construction settlement requirement as compared to preloading option.	• Increased handling of surcharge fill upon completion of surcharge period; however, there may be opportunity to reuse the surcharge fill elsewhere on the contract. • Instrumentation and monitoring program required to assess end of surcharge period.	• Schedule impacts will increase overall project costs. • Additional cost associated with construction of 0.5 m high surcharge. • Cost for instrumentation and monitoring program.	• Duration of surcharge period subject to the instrumentation monitoring program.
Preloading (2 m high granular embankment for 250 days)	2	• Standard construction operation.	• Requires long preload period to reach post-construction settlement criterion. • Instrumentation and monitoring program required to assess end of preload period.	• Schedule impacts will increase overall project costs. • Cost for instrumentation and monitoring program.	• Duration of preload period subject to the instrumentation monitoring program.
Lower Grade by 0.5 m and Surcharging (1 m high for 50 days)	3	• Standard construction operation. • Significantly reduced time to reach post-construction settlement requirement as compared to the preloading option. • Less fill material required to construct the south and north approach embankments in the permanent condition.	• Increased handling of surcharge fill upon completion of surcharge period; however, there may be opportunity to reuse the surcharge fill elsewhere on the contract. • Instrumentation and monitoring program required to assess end of surcharge period; however, duration of monitoring should be shorter. • Will require a revised bridge design due to lower vertical clearance between creek and underside of bridge deck.	• Schedule impacts may increase overall project costs. • Additional cost associated with construction of 1 m high surcharge. • Cost for instrumentation and monitoring program, although this should be shorter than for standard preloading approach. • Reduced cost associated with a lower embankment height.	• Duration of surcharge period subject to the instrumentation monitoring program.
Surcharging (2 m high granular embankment for 20 days) followed by construction of Lightweight Fill Embankment (1 m thick EPS core)	4	• Reduced total settlement of foundation soils. • Relatively short delay in construction schedule to allow for sufficient settlement to occur to meet the post-construction settlement criterion.	• Increased handling of granular fill upon completion of surcharge period. • Requires a 125 mm thick reinforced concrete pad on top of the EPS for protection. • Instrumentation and monitoring program required to assess end of surcharge period; however, duration of monitoring will be relatively short.	• Additional cost for lightweight fill; despite a relatively small volume of EPS. • Cost associated with the construction of a protective cap on top of the lightweight fill. • Additional cost associated with construction of 1 m high surcharge. • Additional cost for instrumentation and monitoring program, although this will be shorter than for the above two options.	• Low risk with respect to instability of embankment and post-construction settlement of foundation soils. • Duration of surcharge period subject to the instrumentation monitoring program.
Wick Drains (1 m spacing installed to approximately Elevation 216.5 m) with Preloading (25 days) *Note: wick drain analysis are considered preliminary.*	5	• Reduced time for primary consolidation settlement to occur.	• Detailed wick drain investigation and design will be required. • Additional time required for installation of wick drains. • Instrumentation and monitoring program required to assess end of preload period. • Additional secondary consolidation settlement will occur as a result of the accelerated completion of primary consolidation.	• Cost associated with detailed wick drain investigation and design. • Cost for the installation of wick drains as well as instrumentation and monitoring program.	• Higher risk associated with the complexity of a wick drain design; however, ultimately mitigation with wick drains should create a lower schedule risk as compared with preloading alone. • Duration of preload period subject to the instrumentation monitoring program.

TABLE 3 – EVALUATION OF SETTLEMENT MITIGATION OPTIONS – SOUTH APPROACH EMBANKMENT

Settlement Mitigation Option [1]	Rank	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Aggregate Piers (on 2.0 m triangular pattern) and Preloading (15 days) *Note: Aggregate pier analyses and comments herein are considered preliminary - detailed analysis will be required by a specialist contractor. The aggregate piers may need to be mixed with grout to create more rigid inclusion.*	6	• Improved embankment stability. • Reduced total settlement of foundation soils. • Relatively short delay in construction schedule, as compared to the other alternatives, to allow for sufficient settlement to occur to meet the post-construction settlement requirement.	• Detailed aggregate pier design will be required. • Additional construction time required for construction of aggregate piers. • Generation of excess excavation spoil. • Temporary casing will likely be required due to low strength of soil deposits encountered below the water table, unless displacement-type aggregate piers can be utilized.	• Schedule impacts may increase overall project costs. • Cost associated with detailed aggregate pier design. • High cost for the construction of aggregate piers, including the disposal of excavation spoils.	• Very low risk with respect to instability of embankment. • Would achieve the long-term settlement performance of the embankment. • Complex aggregate pier design to balance between spacing of piers/preloading period/costs. • Need to ensure that the aggregate piers/columns do not interfere with installation of deep foundations at the south abutment.

Note:
1. All settlement mitigation options, except for the aggregate pier option, assume that sub-excavation of the existing fill/silt and sand deposit to Elevation 225.5 m (extending 20 m south of the south abutment) and replacement with granular fill.

FIGURES

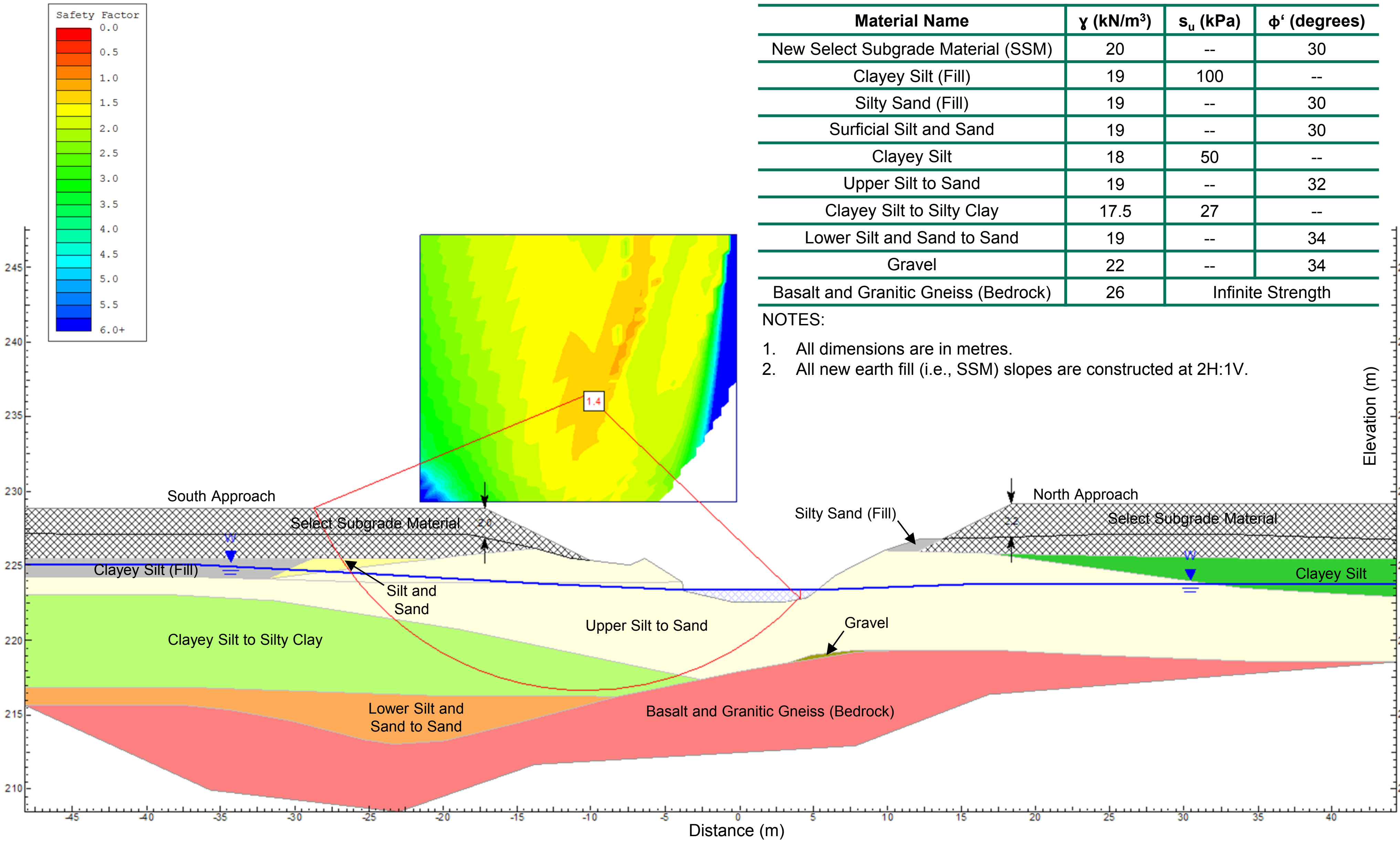
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Front Slope Stability
Base Case (Temporary Condition)

Figure 1A

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	100	--
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	50	--
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	27	--
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017
Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

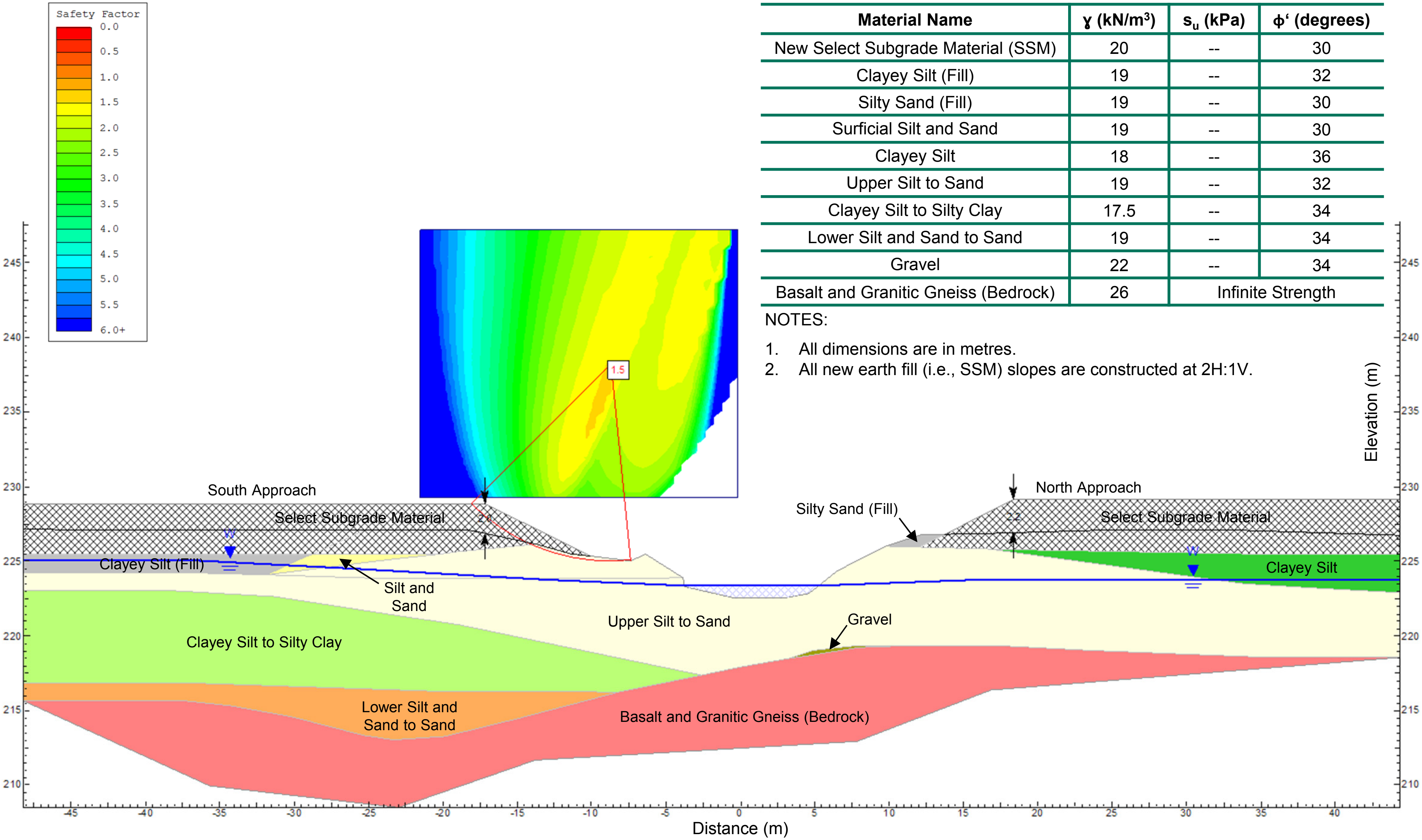
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Front Slope Stability
Base Case (Permanent Condition)

Figure 1B

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	--	32
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	--	36
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	--	34
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



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Project No: 1547670

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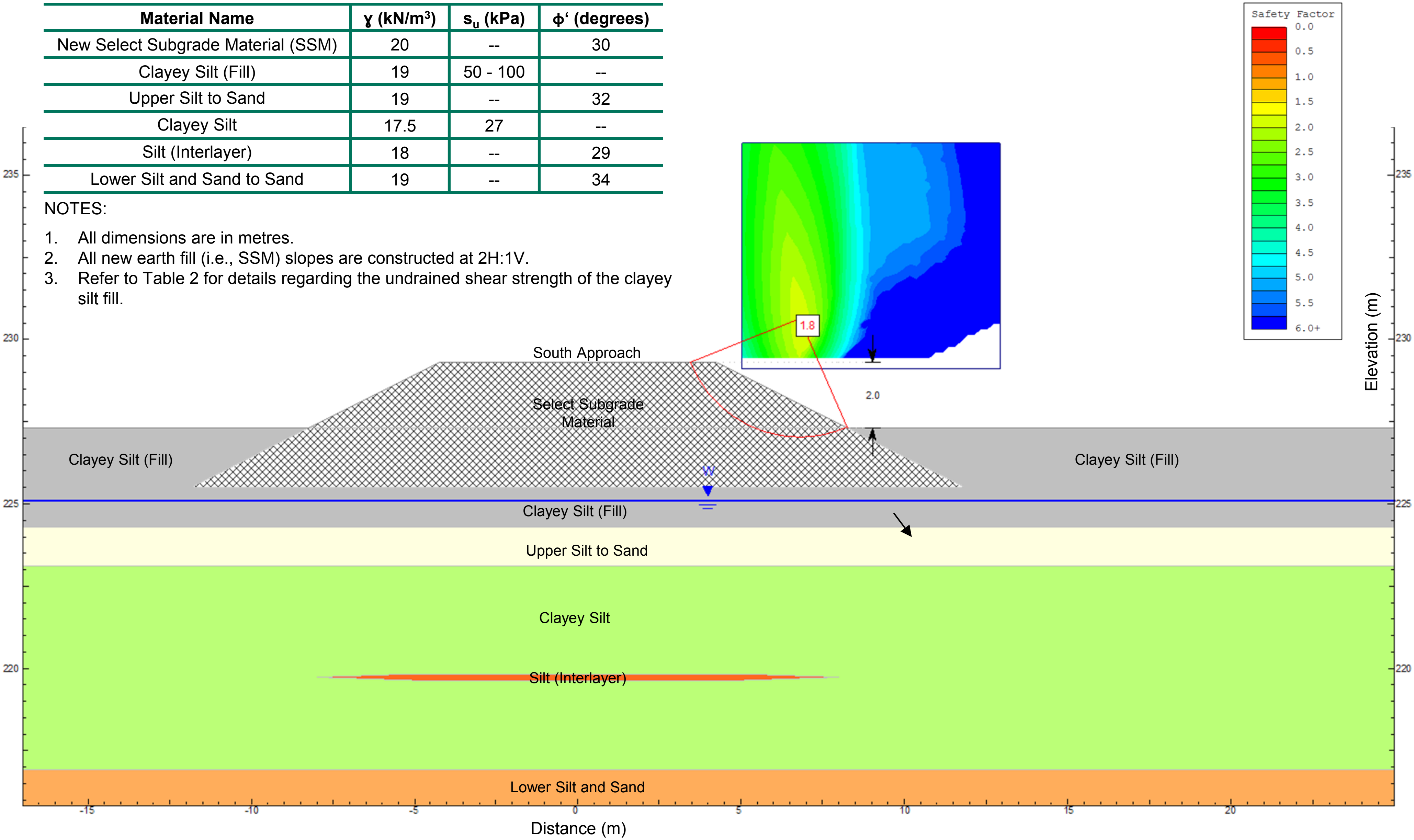
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Side Slope Stability
Base Case (Temporary Condition)

Figure 2A

Material Name	γ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	50 - 100	--
Upper Silt to Sand	19	--	32
Clayey Silt	17.5	27	--
Silt (Interlayer)	18	--	29
Lower Silt and Sand to Sand	19	--	34

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.
3. Refer to Table 2 for details regarding the undrained shear strength of the clayey silt fill.



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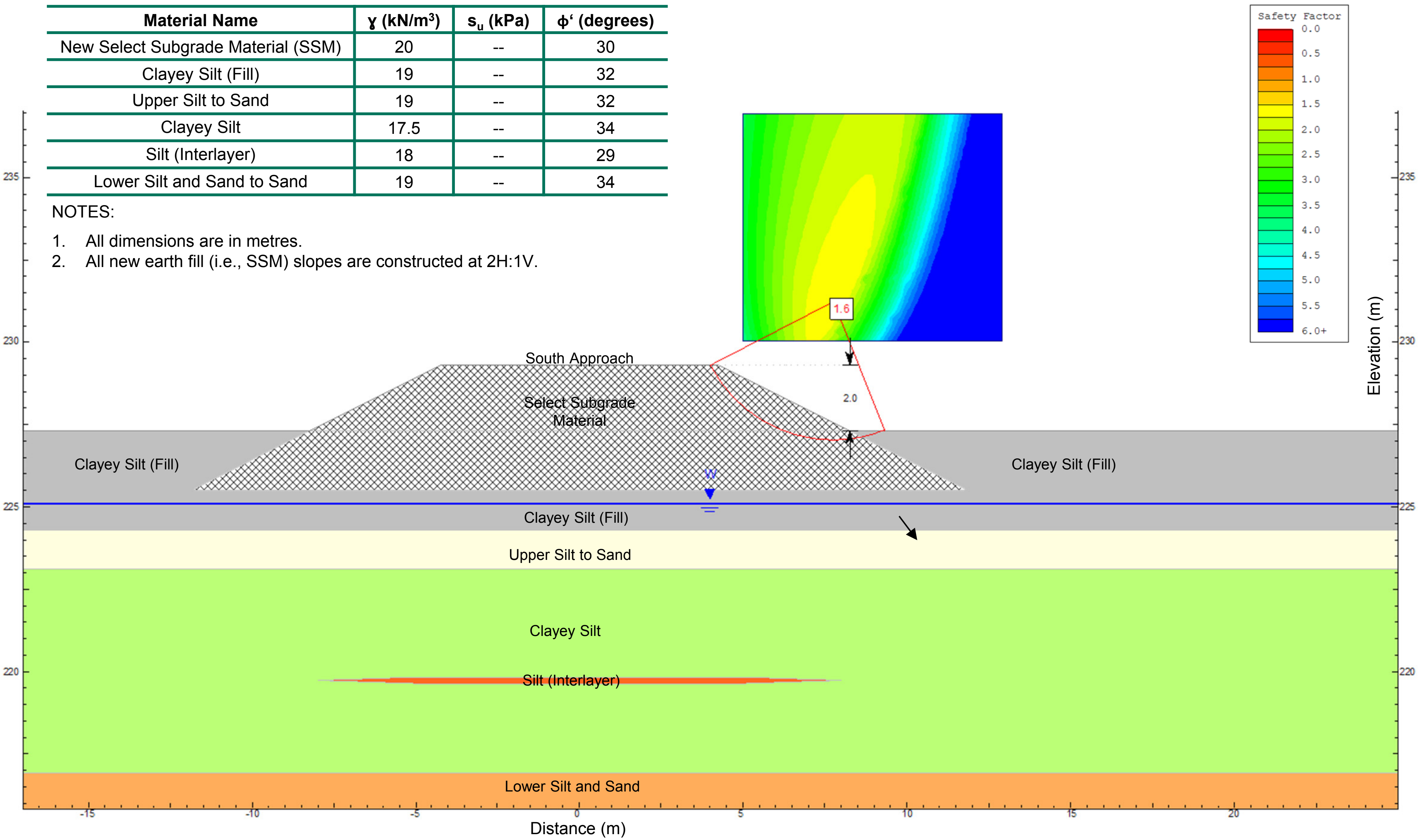
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Side Slope Stability
Base Case (Permanent Condition)

Figure 2B

Material Name	γ (kN/m³)	s_u (kPa)	ϕ‘ (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	--	32
Upper Silt to Sand	19	--	32
Clayey Silt	17.5	--	34
Silt (Interlayer)	18	--	29
Lower Silt and Sand to Sand	19	--	34

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



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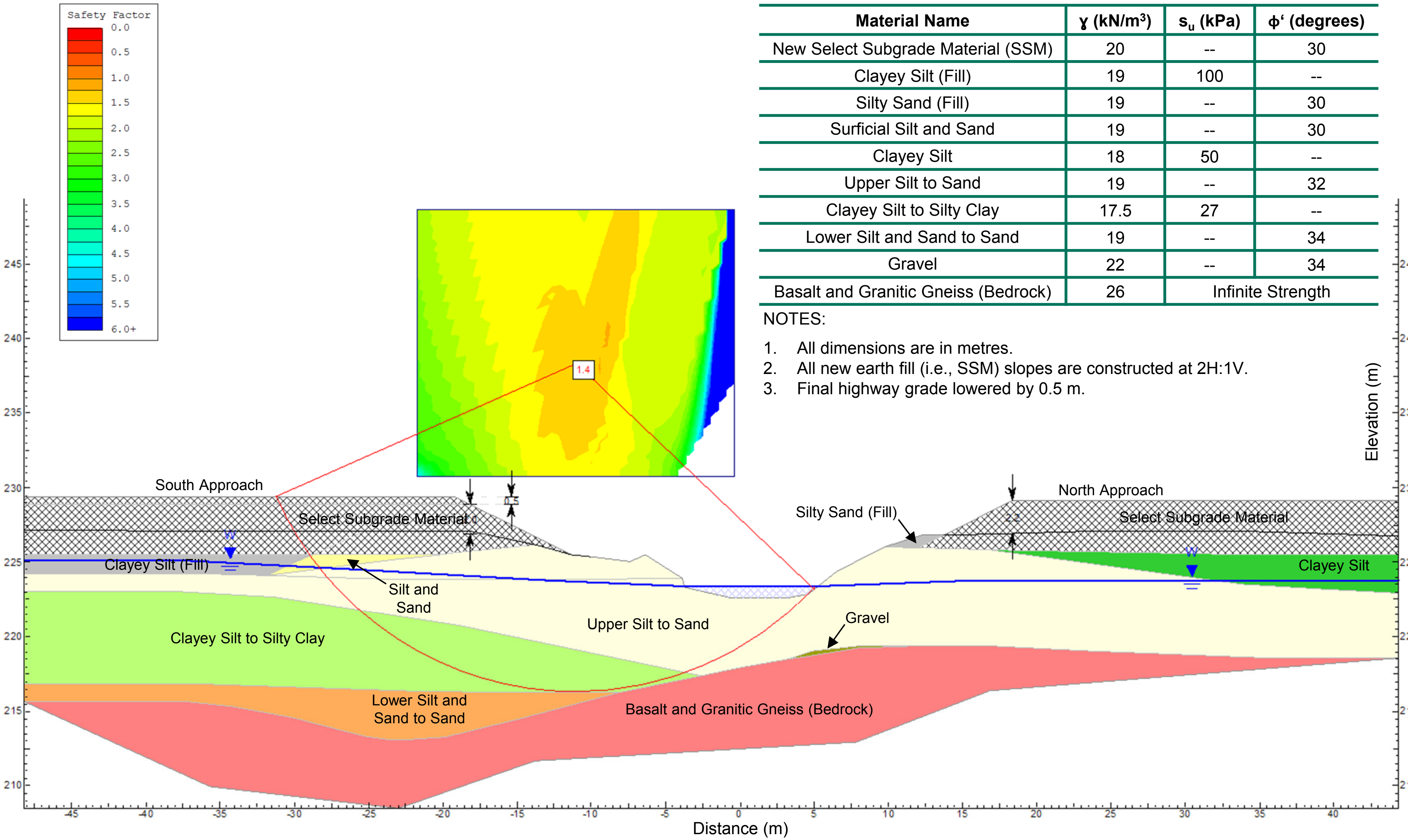
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Front Slope Stability
0.5 m High Surcharge (Temporary Condition)

Figure 3

Material Name	ɣ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	100	--
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	50	--
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	27	--
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.
3. Final highway grade lowered by 0.5 m.



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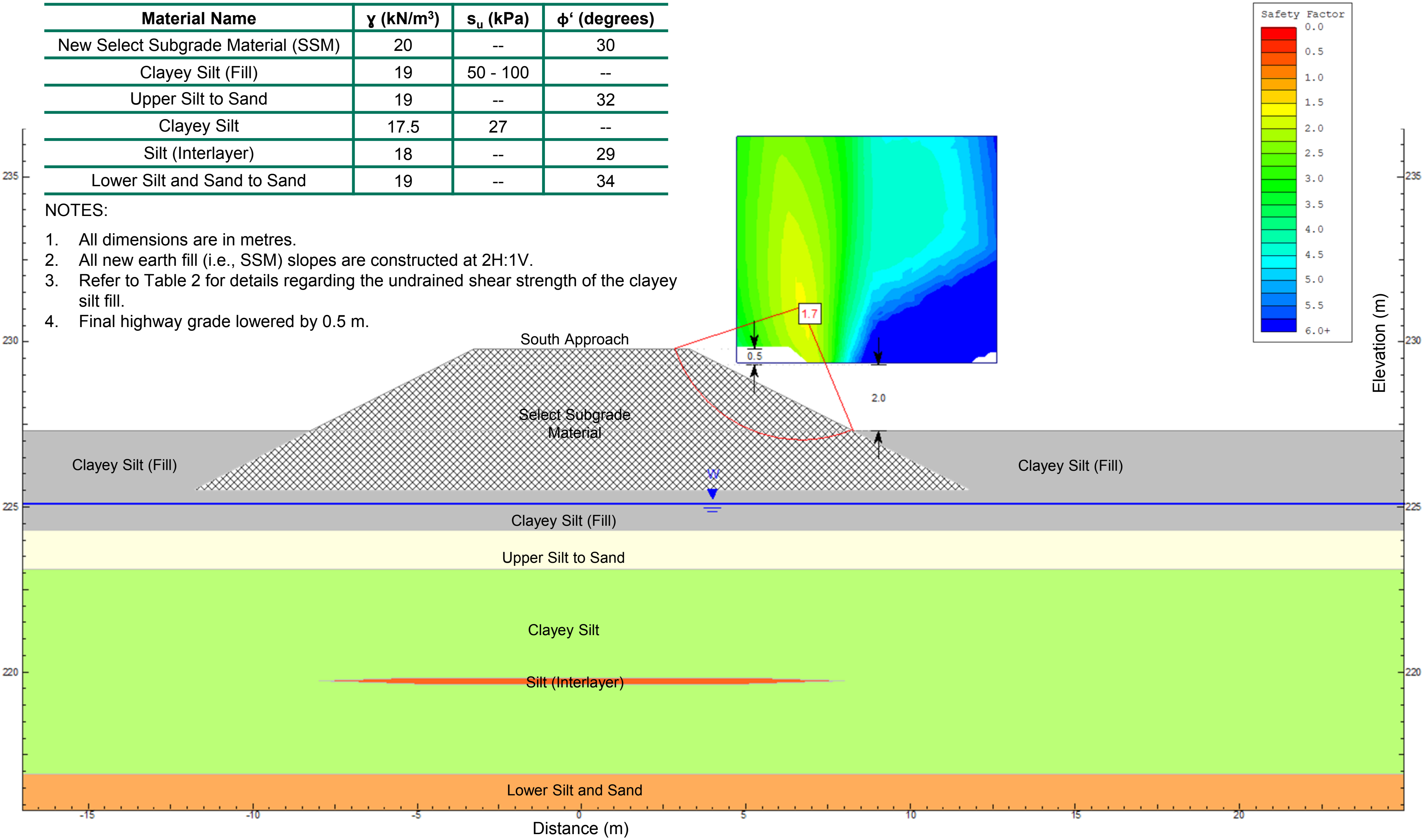
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Side Slope Stability
0.5 m High Surcharge (Temporary Condition)

Figure 4

Material Name	ɣ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	50 - 100	--
Upper Silt to Sand	19	--	32
Clayey Silt	17.5	27	--
Silt (Interlayer)	18	--	29
Lower Silt and Sand to Sand	19	--	34

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.
3. Refer to Table 2 for details regarding the undrained shear strength of the clayey silt fill.
4. Final highway grade lowered by 0.5 m.



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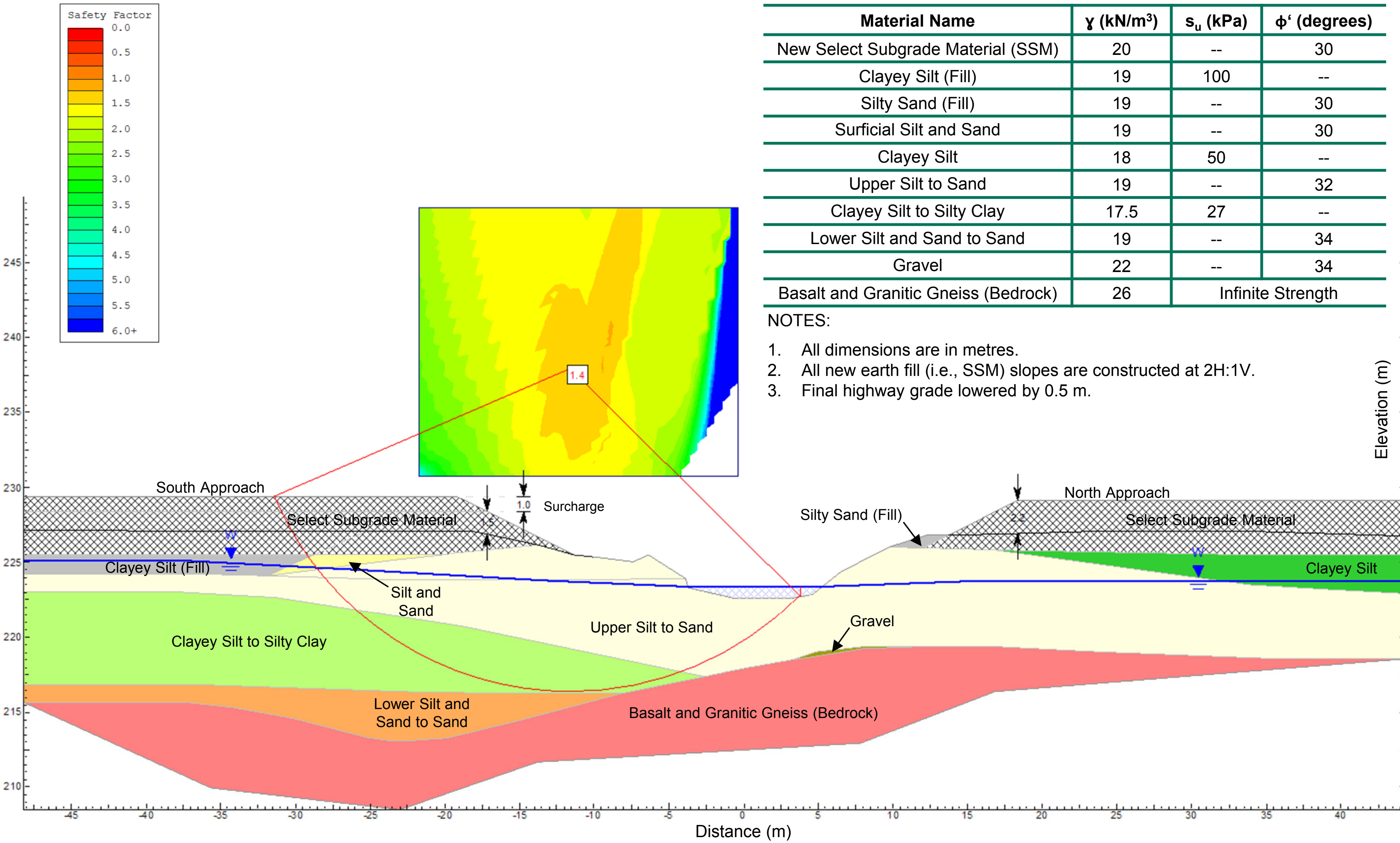
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Front Slope Stability
Lower Grade and 1 m High Surcharge (Temporary Condition)

Figure 5

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	100	--
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	50	--
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	27	--
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.
3. Final highway grade lowered by 0.5 m.



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Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

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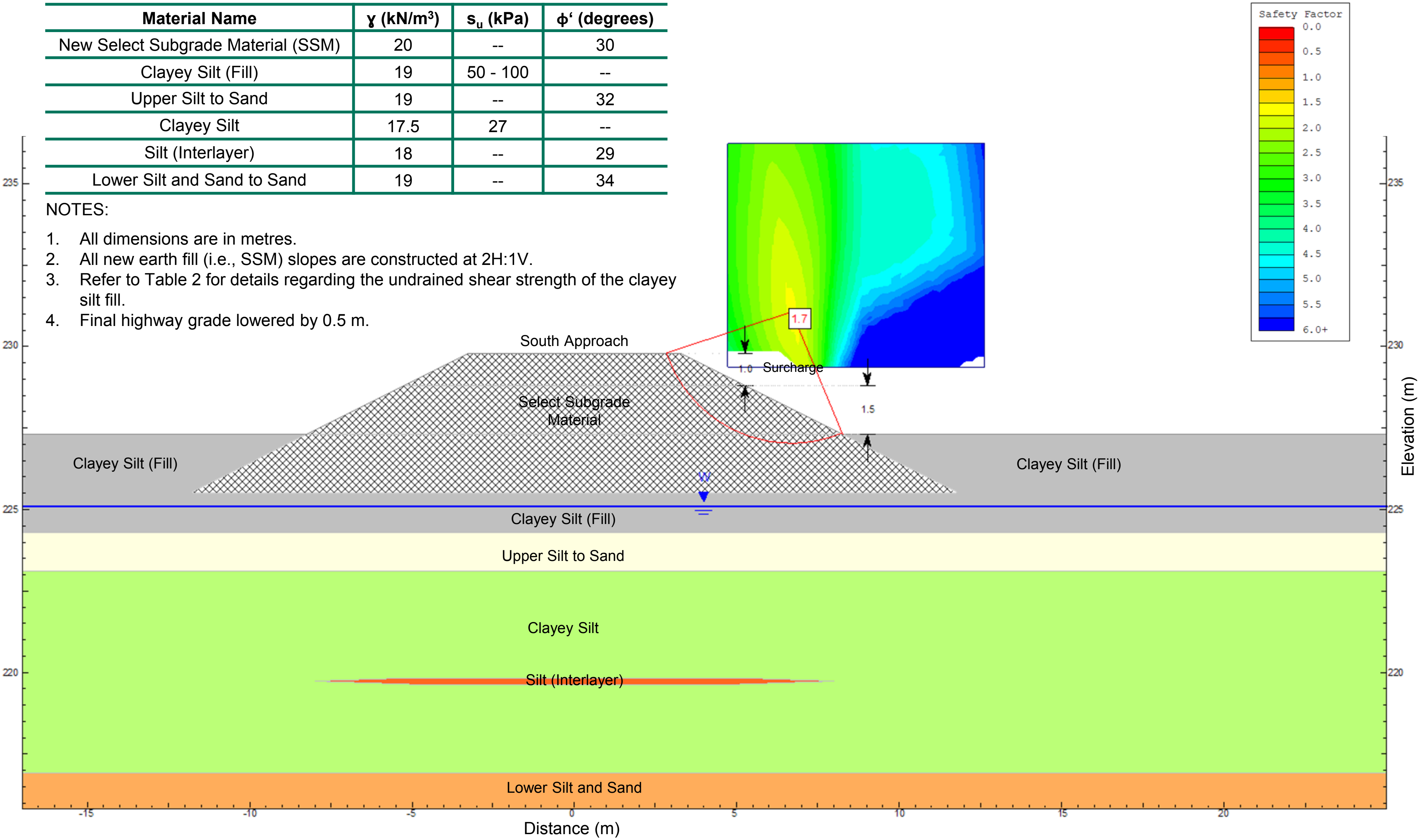
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Side Slope Stability
Lower Grade and 1 m High Surcharge (Temporary Condition)

Figure 6

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	50 - 100	--
Upper Silt to Sand	19	--	32
Clayey Silt	17.5	27	--
Silt (Interlayer)	18	--	29
Lower Silt and Sand to Sand	19	--	34

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.
3. Refer to Table 2 for details regarding the undrained shear strength of the clayey silt fill.
4. Final highway grade lowered by 0.5 m.



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Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

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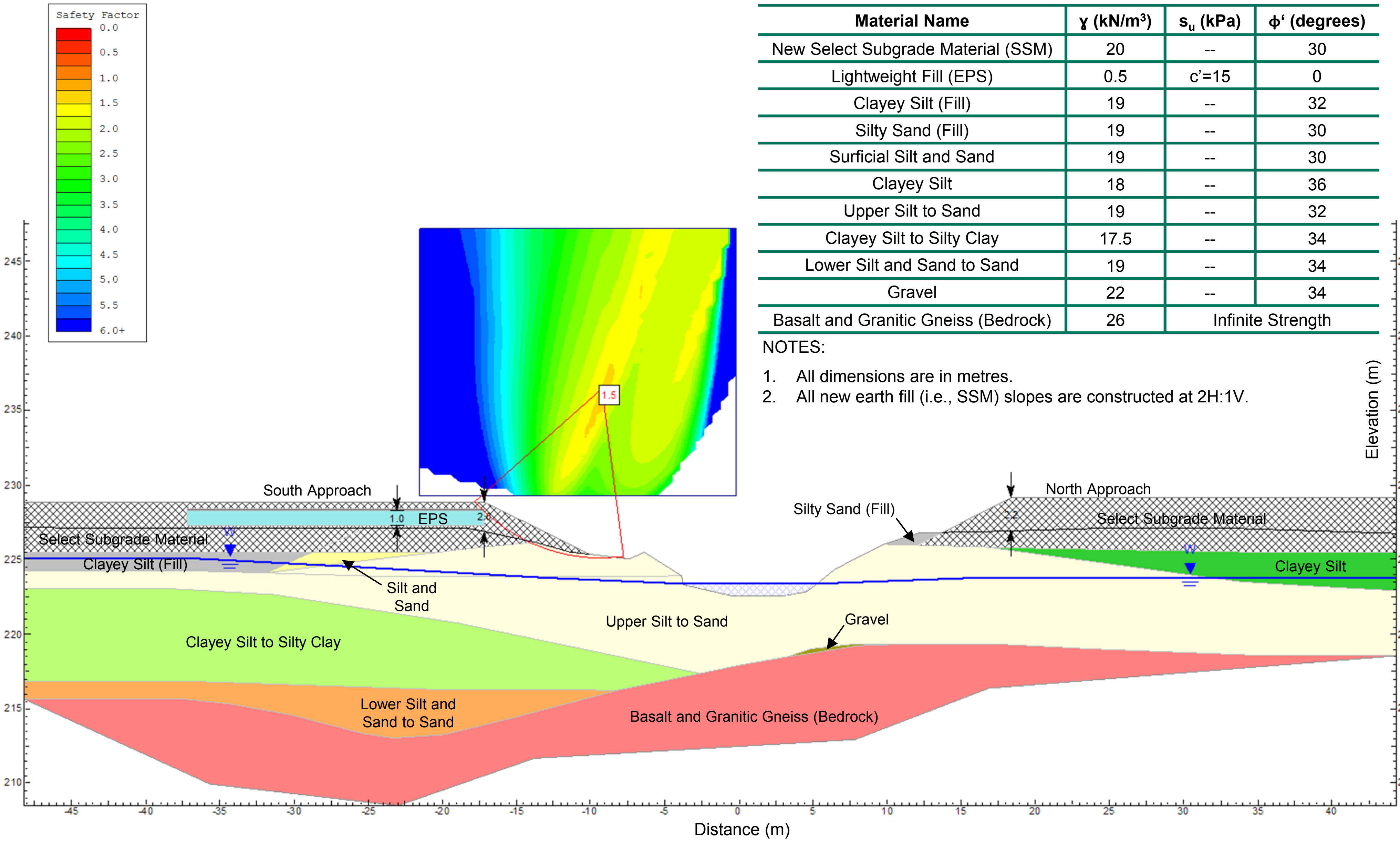
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Front Slope Stability
1 m Thick EPS Core (Permanent Condition)

Figure 7

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Lightweight Fill (EPS)	0.5	c'=15	0
Clayey Silt (Fill)	19	--	32
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	--	36
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	--	34
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017

Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

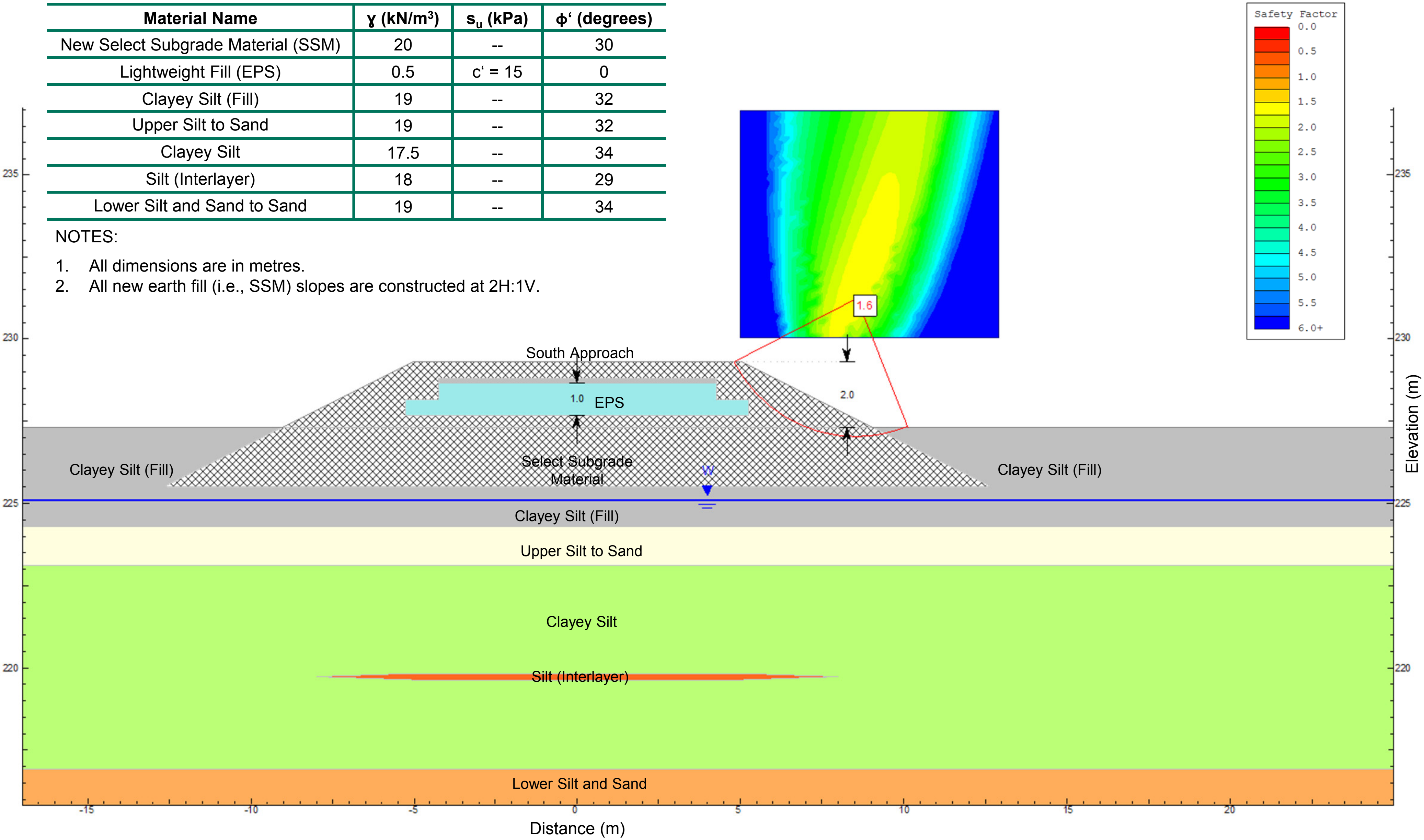
Highway 524 Commanda Creek Bridge Replacement
South Approach Embankment – Side Slope Stability
1 m Thick EPS Core (Permanent Condition)

Figure 8

Material Name	ɣ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Lightweight Fill (EPS)	0.5	c' = 15	0
Clayey Silt (Fill)	19	--	32
Upper Silt to Sand	19	--	32
Clayey Silt	17.5	--	34
Silt (Interlayer)	18	--	29
Lower Silt and Sand to Sand	19	--	34

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017

Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

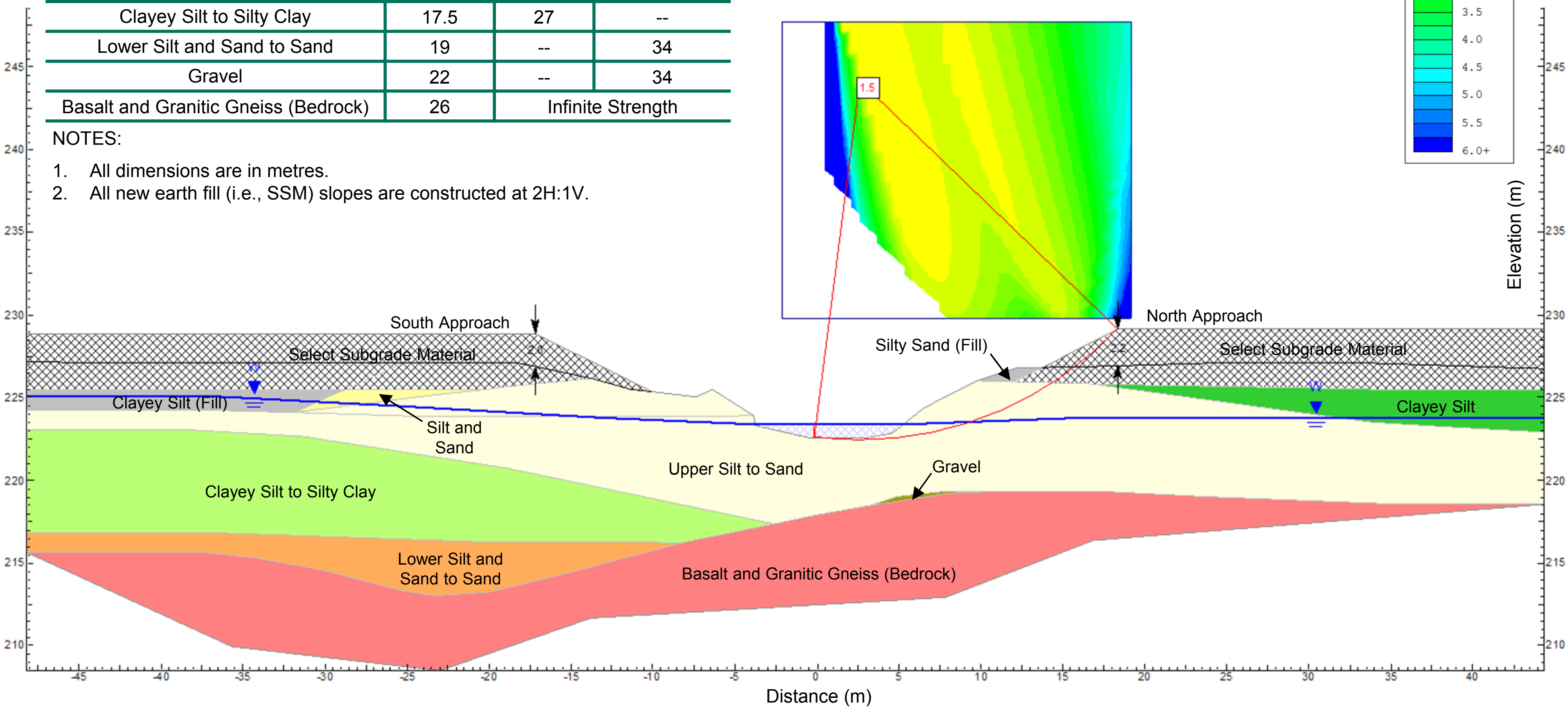
Highway 524 Commanda Creek Bridge Replacement
North Approach Embankment – Front Slope Stability
Base Case (Temporary Condition)

Figure 9A

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	100	--
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	50	--
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	27	--
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017

Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

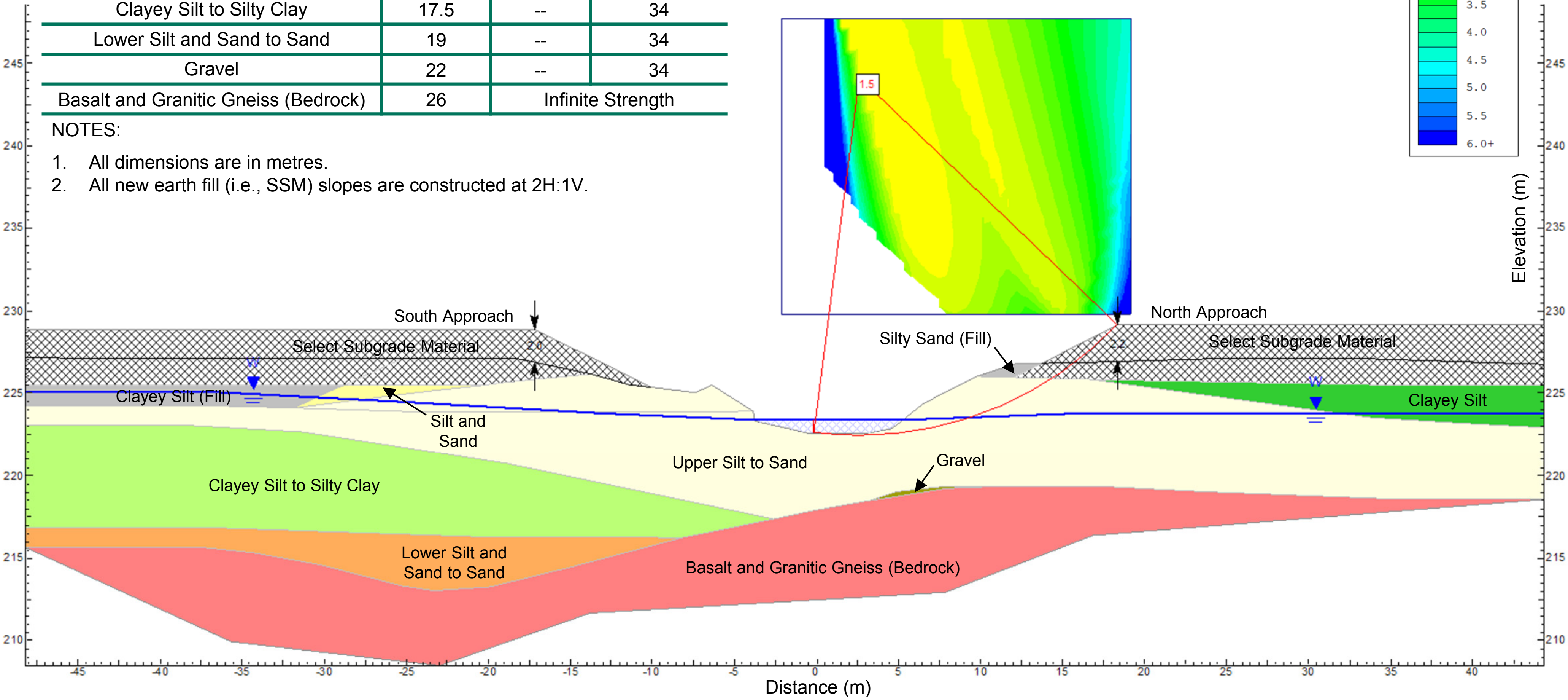
Highway 524 Commanda Creek Bridge Replacement
North Approach Embankment – Front Slope Stability
Base Case (Permanent Condition)

Figure 9B

Material Name	γ (kN/m^3)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Clayey Silt (Fill)	19	--	32
Silty Sand (Fill)	19	--	30
Surficial Silt and Sand	19	--	30
Clayey Silt	18	--	36
Upper Silt to Sand	19	--	32
Clayey Silt to Silty Clay	17.5	--	34
Lower Silt and Sand to Sand	19	--	34
Gravel	22	--	34
Basalt and Granitic Gneiss (Bedrock)	26	Infinite Strength	

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017
Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

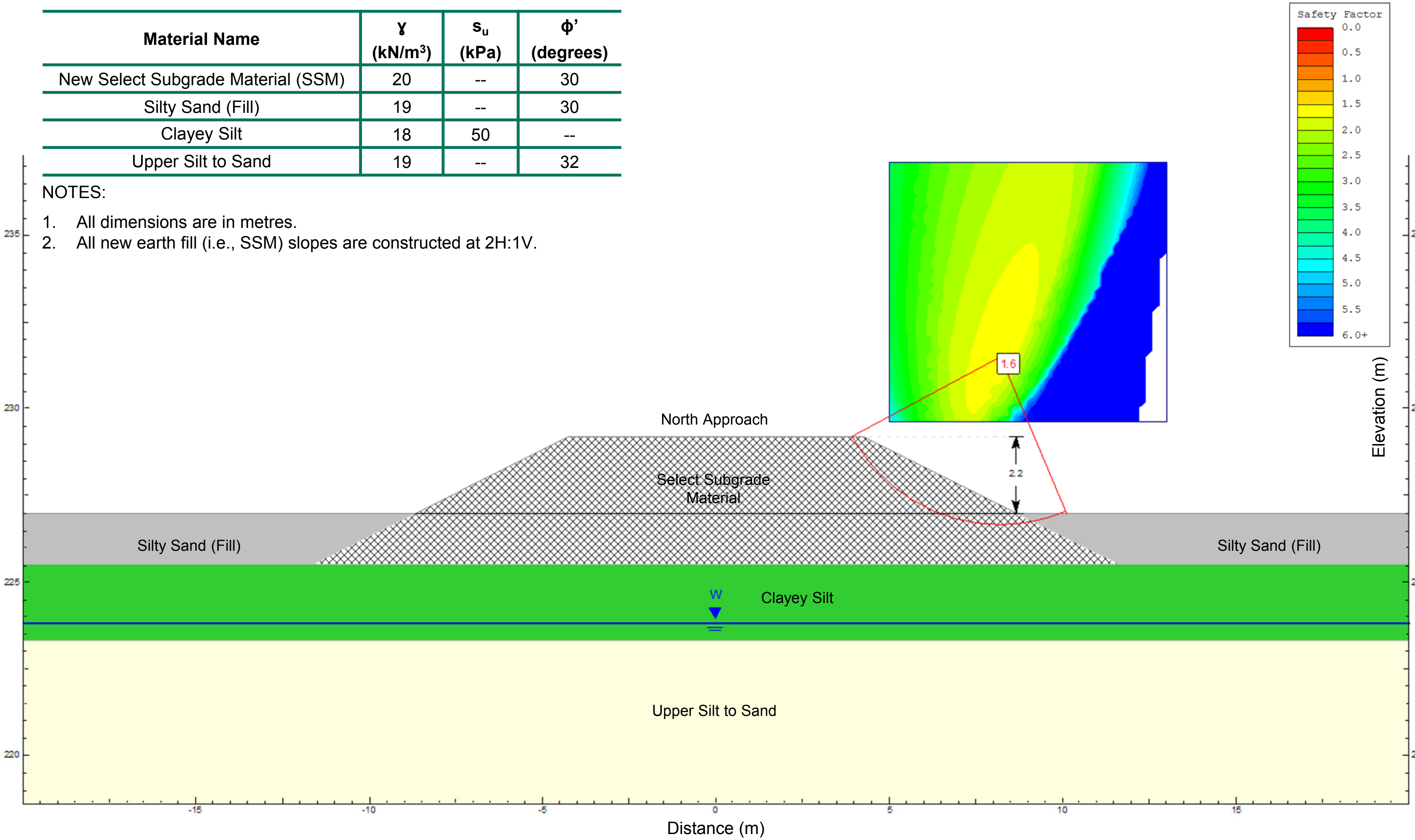
Highway 524 Commanda Creek Bridge Replacement
North Approach Embankment – Side Slope Stability
Base Case (Temporary Condition)

Figure 10A

Material Name	γ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Silty Sand (Fill)	19	--	30
Clayey Silt	18	50	--
Upper Silt to Sand	19	--	32

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017
Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

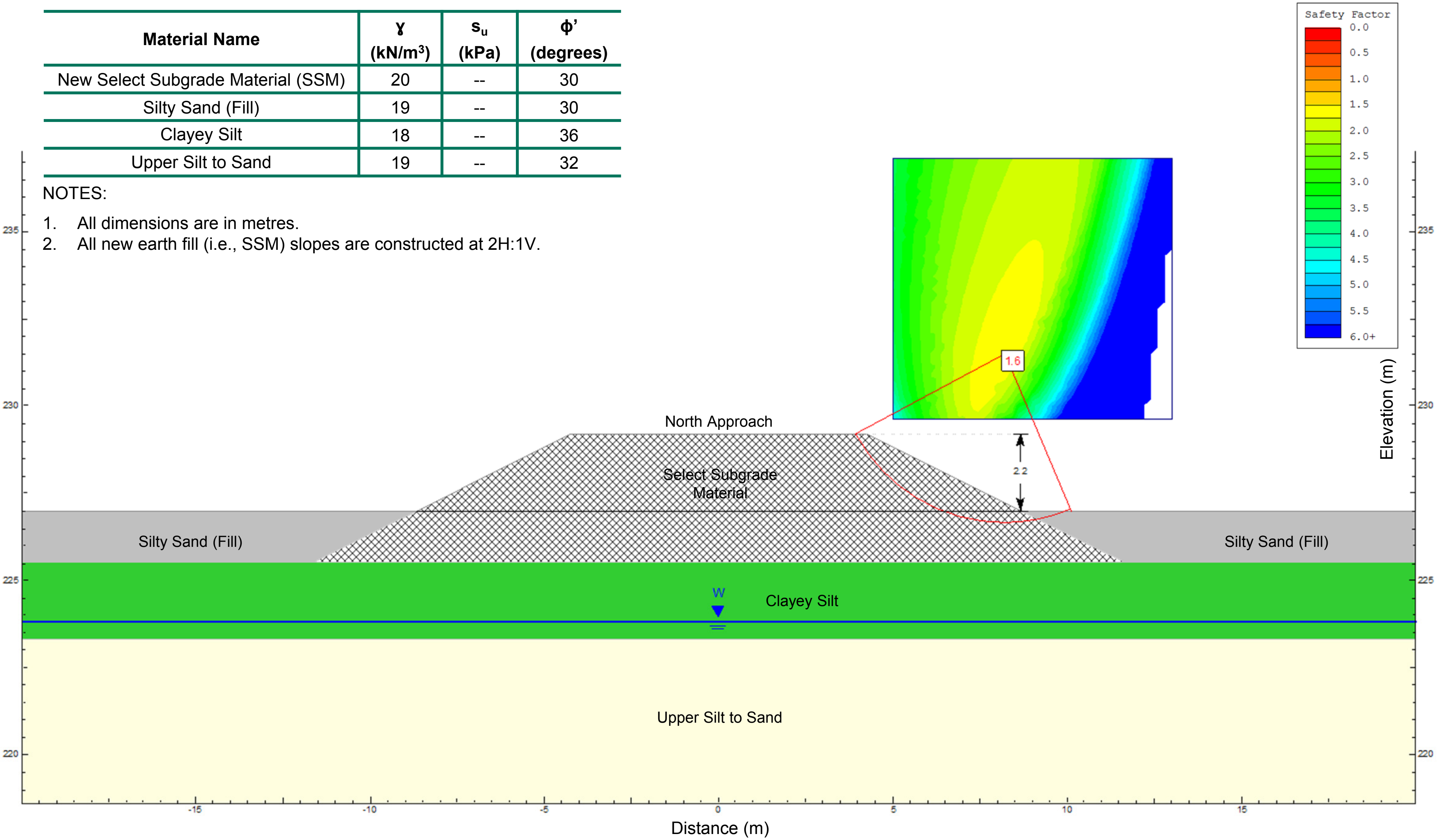
Highway 524 Commanda Creek Bridge Replacement
North Approach Embankment – Side Slope Stability
Base Case (Permanent Condition)

Figure 10B

Material Name	ɣ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Select Subgrade Material (SSM)	20	--	30
Silty Sand (Fill)	19	--	30
Clayey Silt	18	--	36
Upper Silt to Sand	19	--	32

NOTES:

1. All dimensions are in metres.
2. All new earth fill (i.e., SSM) slopes are constructed at 2H:1V.



Date: August 31, 2017
Project No: 1547670

Analysis By: TZ **Reviewed By:** CN/LCC

Golder Associates

DRAWINGS

MINISTRY OF TRANSPORTATION, ONTARIO

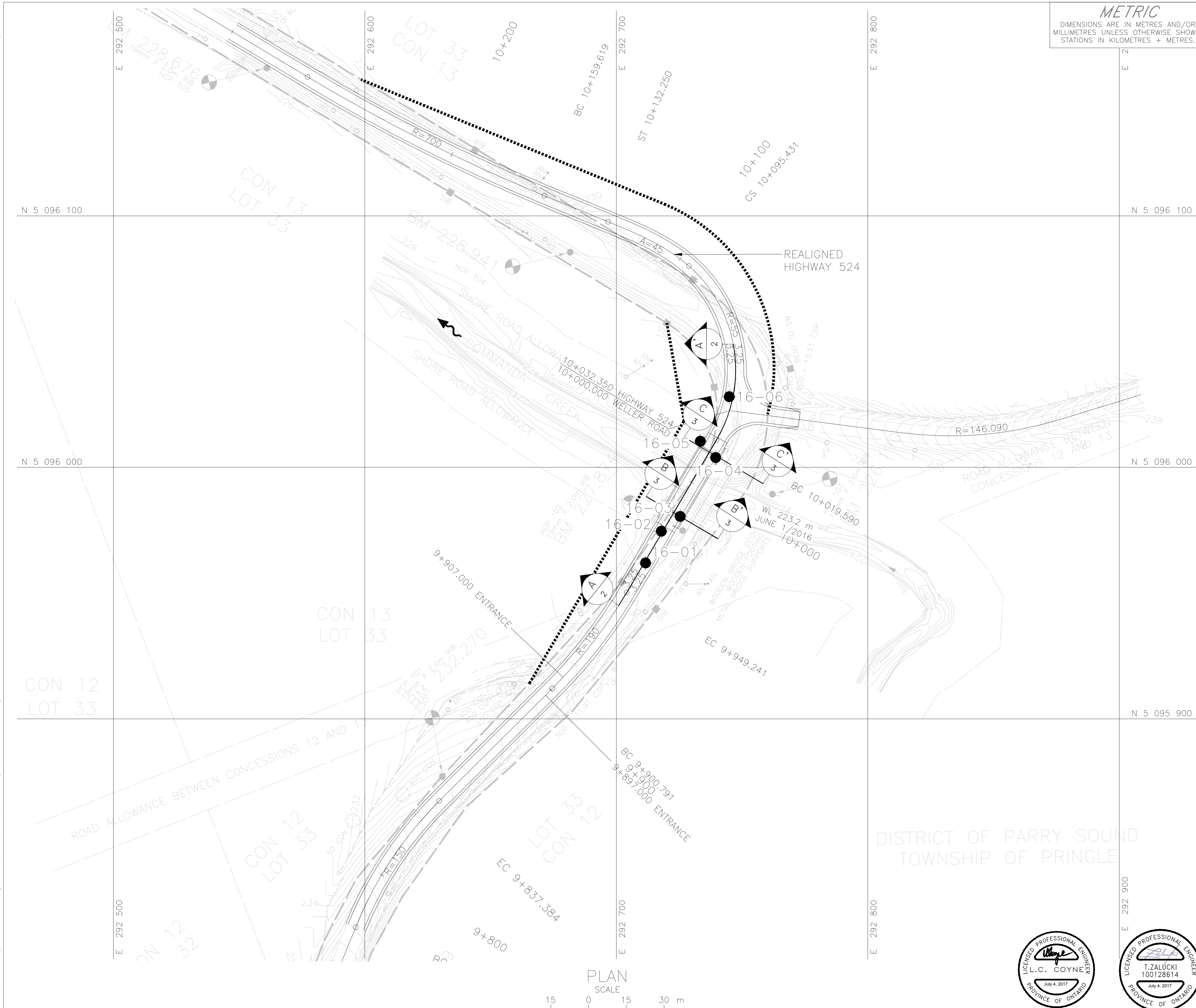
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.5260–13–00

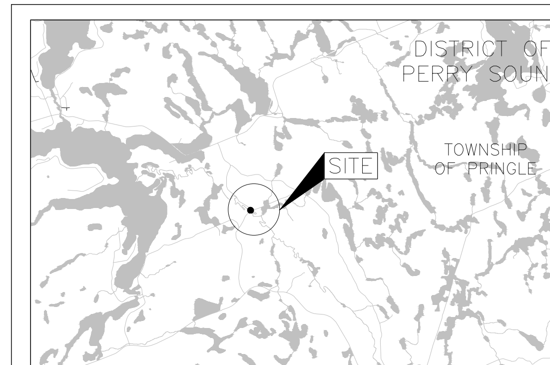
SHEET

HIGHWAY 524 COMMANDA CREEK
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS

Golder Associates



PLAN
SCALE
15 0 15 30 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

● Borehole – Current Investigation

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16–01	227.3	5095962.0	292711.6
16–02	227.1	5095974.6	292717.9
16–03	226.9	5095980.5	292725.4
16–04	227.0	5096003.9	292739.5
16–05	226.0	5096010.4	292733.4
16–06	227.0	5096028.1	292744.9

NOTES

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

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file Highway 524 Plan oct 7 2016.dwg and Highway 524_bgd.dwg, received OCT 07, 2016.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE July 4, 2017 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.ZALUCKI 100128614 July 4, 2017 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31L–205

HWY. 524	PROJECT NO. 1547670	DIST. NORTHEAST
SUBM'D. AK	CHKD. AK	DATE: 7/4/2017	SITE: 44–029
DRAWN: SMD	CHKD. TZ	APPD. JMAC	DWG. 1

PLOT DATE: July 4, 2017
FILENAME: S:\Clients\MTO\Hwy_524_CommandaCreekBridge\99_PROJ\1547670_40_PROD\0001_GeotechAnalysis\1547670_0003_BG_0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

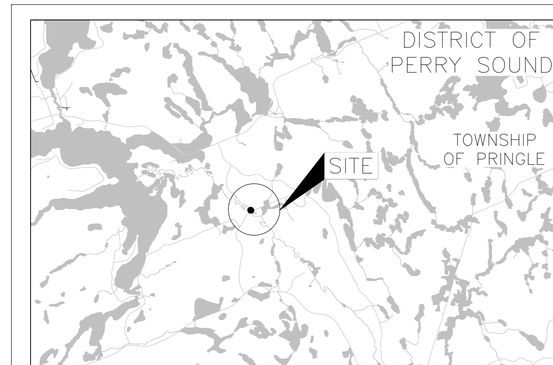
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.5260-13-00

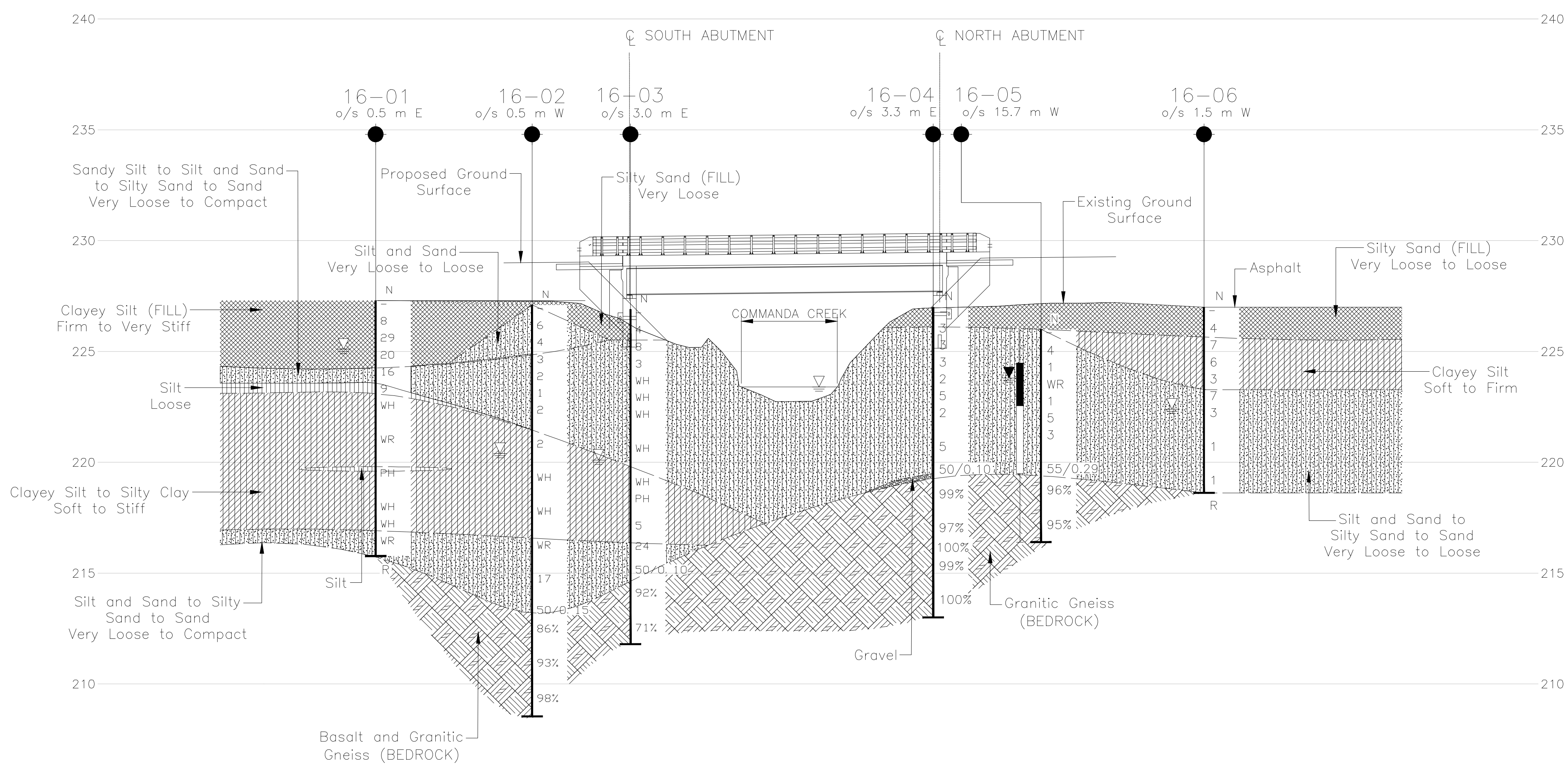
HIGHWAY 524 COMMANDA CREEK BRIDGE REPLACEMENT
SOIL STRATA

SHEET

Golder Associates



KEY PLAN
SCALE
2 0 2 4 km



A-A / 2 CENTRELINE PROFILE
HIGHWAY 524

HORIZONTAL SCALE
5 0 5 10 m

VERTICAL SCALE
2.5 0 2.5 5 m

LEGEND

- ● Borehole – Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on DEC 16, 2016
- WL upon completion of drilling
- R Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
16-01	227.3	5095962.0	292711.6
16-02	227.1	5095974.6	292717.9
16-03	226.9	5095980.5	292725.4
16-04	227.0	5096003.9	292739.5
16-05	226.0	5096010.4	292733.4
16-06	227.0	5096028.1	292744.9

NOTES

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LICENSED PROFESSIONAL ENGINEER L.C. COYNE July 4, 2017 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.ZALUCKI 100128614 July 4, 2017 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31L-205			
HWY. 524	PROJECT NO. 1547670		DIST. NORTHEAST
SUBM'D. AK	CHKD. AK	DATE: 7/4/2017	SITE: 44-029
DRAWN: SMD	CHKD. TZ	APPD. JMAC	DWG. 2

PLOT DATE: July 4, 2017

MINISTRY OF TRANSPORTATION, ONTARIO

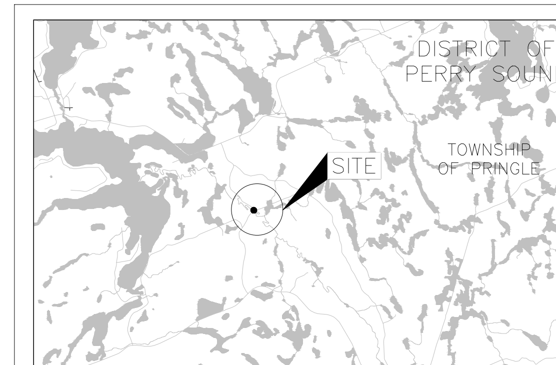
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.5260-13-00

SHEET

HIGHWAY 524 COMMANDA CREEK
BRIDGE REPLACEMENT
SOIL STRATA

Golder Associates



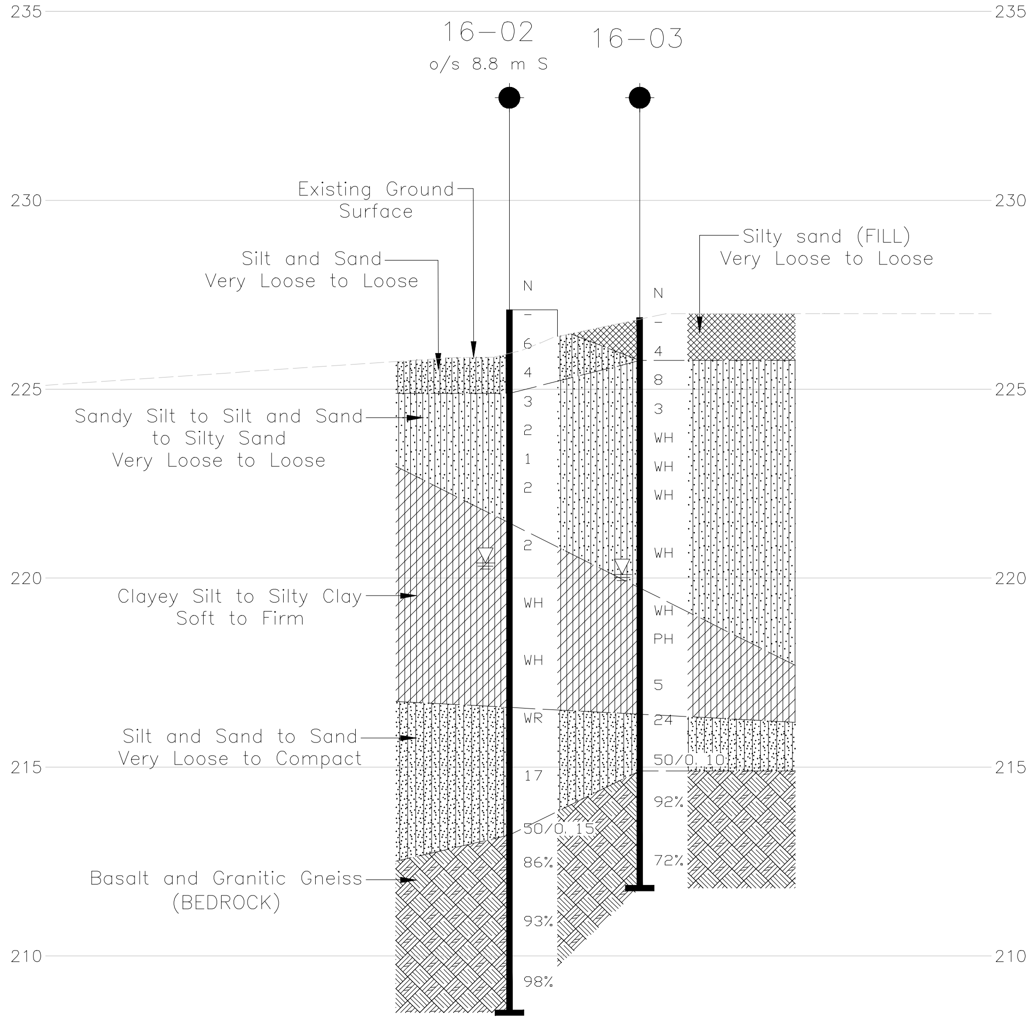
KEY PLAN
SCALE
2 0 2 4 km

LEGEND

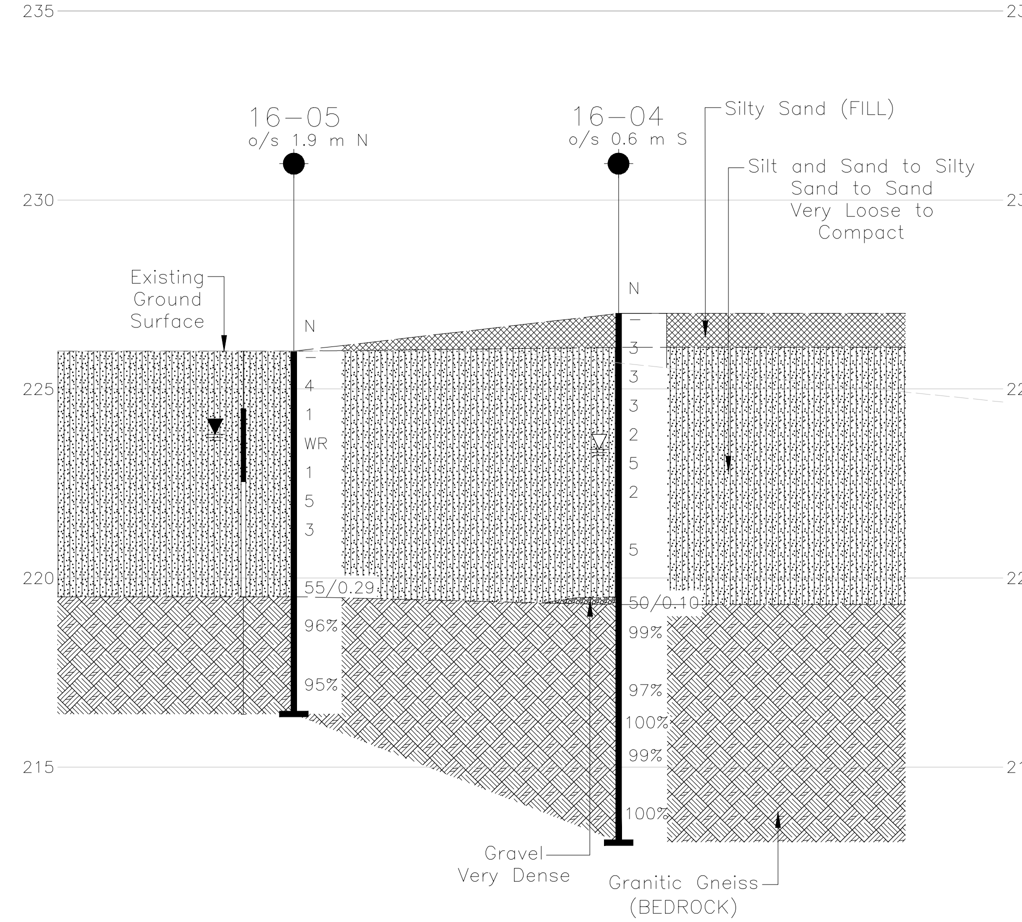
●	Borehole – Current Investigation
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on Dec. 16, 2016
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
16-02	227.1	5095974.6	292717.9
16-03	226.9	5095980.5	292725.4
16-04	227.0	5096003.9	292739.5
16-05	226.0	5096010.4	292733.4



B-B / 3 SOUTH ABUTMENT
HIGHWAY 524



C-C / 3 NORTH ABUTMENT
HIGHWAY 524

SCALE
2 0 2 4 m

NOTES

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LICENSED PROFESSIONAL ENGINEER L.C. COYNE July 4, 2017 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.ZALUCKI 100128614 July 4, 2017 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31L-205

HWY. 524	PROJECT NO. 1547670		DIST. NORTHEAST
SUBM'D. AK	CHKD. AK	DATE: 7/4/2017	SITE: 44-029
DRAWN: SMD	CHKD. TZ	APPD. JMAC	DWG. 3

PLOT DATE: July 4, 2017

APPENDIX A

Record of Borehole and Drillhole Sheets

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = 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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	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^o angle is horizontal.

Description and Notes

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

Abbreviations

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-01

SHEET 1 OF 2

METRIC

PROJECT 1547670

G.W.P. 5260-13-00

LOCATION N 5095962.0; E 292711.6 MTM ZONE 10 (LAT. 46.003372; LONG. -79.656082)

ORIGINATED BY ACK

DIST Northeast HWY 524

BOREHOLE TYPE 178 mm O.D. Continuous Flight Hollow Stem Augers, NW Casing and Wash Boring

COMPILED BY SMD

DATUM Geodetic

DATE November 24 and 26, 2016

CHECKED BY MCK/TZ

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
227.3	GROUND SURFACE							
0.0	Clayey silt, trace to some sand, trace organics, with wood pieces and rootlets to a depth of about 1.4 m (FILL) Firm to very stiff Brown mottled grey Moist	1	AS	-				
		2	SS	8				
		3	SS	29				
		4	SS	20	0	0	80	20
224.3 / 3.0	SILT and SAND Compact Brown Moist	5	SS	16				
223.6 / 3.7	SILT, trace to some clay, trace sand Loose Brown Moist	6A	SS	9	0	5	88	7
223.1 / 4.2	CLAYEY SILT, trace to some sand Firm to stiff Grey Moist to wet	6B						
		7	SS	WH				
		8	SS	WR*				
219.8	SILT, some sand, trace clay Grey Wet	9A	TO	PH	0	12	87	1
7.7	CLAYEY SILT, trace to some sand Firm Grey Moist to wet	9B						
		10	SS	WH				
	- Trace gravel below a depth of about 9.9 m	11A	SS	WH				
216.9 / 10.4	SILT and SAND, trace to some clay, trace gravel Very loose Grey Wet	11B						
		12	SS	WR**				
215.8 / 11.5	END OF BOREHOLE CASING REFUSAL							

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (227 to 216); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

Field vane sensitivity labels plotted: 9, 3, 9, 9, 14, 6

Continued Next Page

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 16-01

SHEET 2 OF 2

METRIC

PROJECT 1547670

G.W.P. 5260-13-00

LOCATION N 5095962.0; E 292711.6 MTM ZONE 10 (LAT. 46.003372; LONG. -79.656082)

ORIGINATED BY ACK

DIST Northeast HWY 524

BOREHOLE TYPE 178 mm O.D. Continuous Flight Hollow Stem Augers, NW Casing and Wash Boring

COMPILED BY SMD

DATUM Geodetic

DATE November 24 and 26, 2016

CHECKED BY MCK/TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										

NOTES:

* A Shelby tube was first pushed at a depth of about 6.1 m, however, the sample was not recovered upon retrieval of the Shelby tube. Consequently, a split-spoon sample was obtained at the same depth, but the SPT 'N'-value is considered unrepresentative due to sample disturbance.

** Unrepresentative SPT 'N'-value due to unbalanced hydrostatic head.

1. The original borehole was augered to a depth of about 1.5 m. As such, an additional borehole was advanced to a depth of about 0.5 m north of Borehole 16-01 to obtain a split-spoon sample and carry out a Standard Penetration Test at a depth of about 0.8 m.

2. Water level in casing measured at a depth of about 2.2 m below ground surface (Elev. 225.1 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No 16-02

SHEET 1 OF 2

METRIC

PROJECT 1547670

G.W.P. 5260-13-00 LOCATION N 5095974.6; E 292717.9 MTM ZONE 10 (LAT. 46.003486; LONG. -79.656001) ORIGINATED BY ACK

DIST Northeast HWY 524 BOREHOLE TYPE 178 mm O.D. Hollow Stem Augers, NW Casing, Wash Boring; NQ Rock Coring COMPILED BY SMD

DATUM Geodetic DATE November 25 and 26, 2016 CHECKED BY MCK/TZ

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
227.1	GROUND SURFACE					
0.0	SILT and SAND, some clay, trace gravel, trace organics; Very loose to loose; Brown; Moist	1	AS	-		
		2	SS	6		5 31 49 15
		3	SS	4		
224.9 / 2.2	SILTY SAND, trace clay, trace organics to a depth of about 3.2 m; Very loose; Brown; Moist; - An approximately 0.15 m thick layer of clayey silt encountered at a depth of about 3.1 m.	4	SS	3		
		5A / 5B	SS	2		
		6	SS	1		0 80 19 1
		7	SS	2		
221.5 / 5.6	CLAYEY SILT, trace gravel, trace sand; Soft to firm; Grey; Moist to wet	8	SS	2	▽	
		9	SS	WH		
		10	SS	WH		
216.6 / 10.5	SILT and SAND, trace clay to SAND, some silt, trace clay, trace gravel; Very loose to compact; Grey; Moist to wet; - Cobbles inferred below a depth of about 12.2 m due to difficulties in casing advancement.	11	SS	WR*		0 56 39 5
		12	SS	17		
213.2		13	SS	50/0.15		
13.9	SPLIT-SPOON AND CASING REFUSAL; GRANITIC GNEISS and BASALT (BEDROCK)	1	RC	REC 100%		RQD = 86%

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213

Field vane sensitivity numbers: 9, 9, 3, 8, 19, 4

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-02

METRIC

PROJECT 1547670

G.W.P. 5260-13-00

LOCATION N 5095974.6; E 292717.9 MTM ZONE 10 (LAT. 46.003486; LONG. -79.656001)

ORIGINATED BY ACK

DIST Northeast HWY 524

BOREHOLE TYPE 178 mm O.D. Hollow Stem Augers, NW Casing, Wash Boring; NQ Rock Coring

COMPILED BY SMD

DATUM Geodetic

DATE November 25 and 26, 2016

CHECKED BY MCK/TZ

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	GRANITIC GNEISS and BASALT (BEDROCK) Bedrock cored between depths of about 13.9 m and 18.6 m For bedrock coring details refer to Record of Drillhole 16-02.		1	RC	REC 100%		212				RQD = 86%
			2	RC	REC 93%		211				RQD = 93%
			3	RC	REC 100%		210 209				RQD = 98%
208.5 18.6	END OF BOREHOLE NOTES: * Unrepresentative SPT 'N'-value due to unbalanced hydrostatic head. 1. Water level in augers measured at a depth of about 6.7 m below ground surface (Elev. 220.4 m) prior to wash boring.										

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

PROJECT: 1547670

RECORD OF DRILLHOLE: 16-02

SHEET 1 OF 1

LOCATION: N 5095974.6 ;E 292717.9

DRILLING DATE: November 26, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850 (Track-Mounted)

DRILLING CONTRACTOR: Landcore Drilling Inc.

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.

Drilling record: Wireline Diamond Rock Coring, NQ Double Tube Core Barrel

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
	Continued from Record of Borehole 16-02	213.21					
14	Fresh, massive, black, fine-grained, non-porous, strong BASALT	13.89	1	BC,,			UCS=97.7 MPa
	Fresh, foliated, pink and black, medium grained, non-porous, very strong GRANITIC GNEISS	212.64 14.46	1	CO,CU,RO	3	1	
				CO,IR,RO	3	4	
				BC,,			
				CO,PL,RO	1.5	1	
				BC,,			
15				CO,PL,SM	1	1	
				JN,PL,SM	1	4	
				FO,IR,RO	3	1	
			2	BC,,			
				CO,PL,RO	1.5	1	
16							
		210.17					
17	Fresh, massive, black, fine grained, non-porous, strong to very strong BASALT	16.93	3	JN,PL,SM	1	2	
	Fresh, foliated, pink and black, medium grained, non-porous, very strong GRANITIC GNEISS	17.10		CO,UN,RO	3	1	
				CO,PL,RO	1.5	4	
				CO,PL,RO	1.5	4	
18				FO,CU,RO	3	1	
		208.54		CO,ST,RO	3	1	
	END OF DRILLHOLE	18.56					
19	NOTE: 1. Geographic Location: Latitiude: 46.003486° Longitude: -79.656001°						

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DEPTH SCALE 1 : 50

Golder Associates

LOGGED: ACK

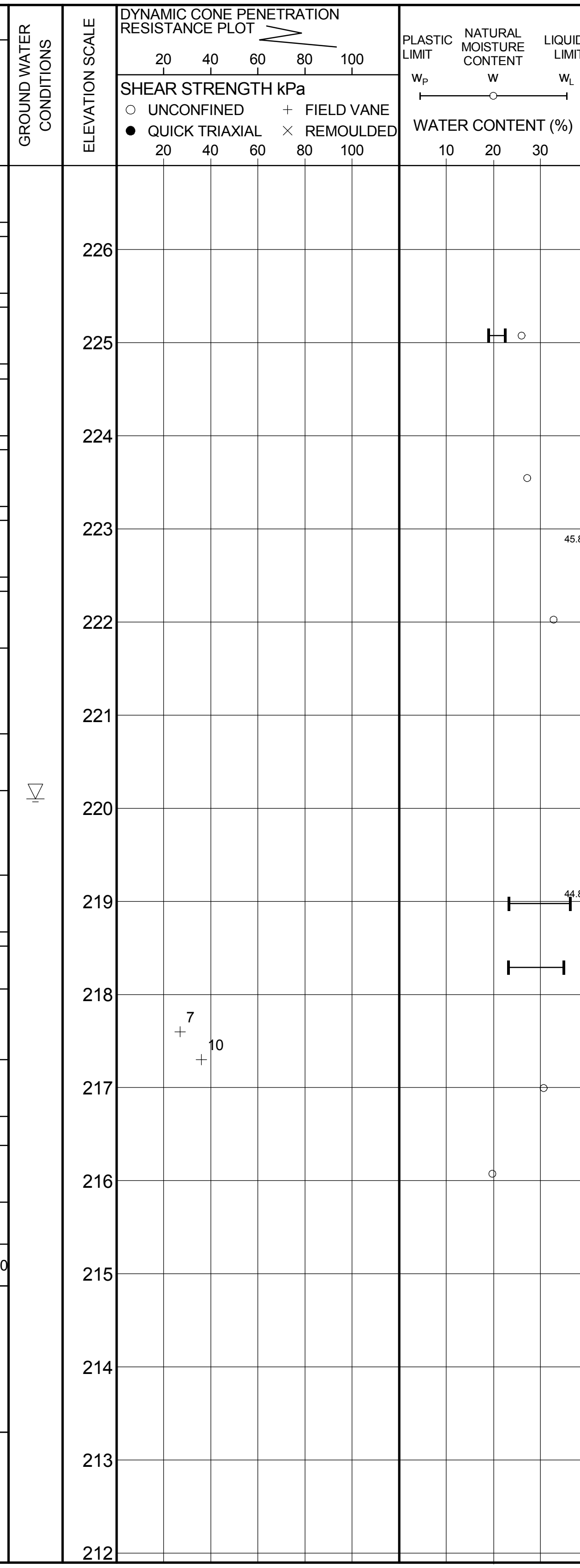
CHECKED: MCK/TZ

RECORD OF BOREHOLE No 16-03

SHEET 1 OF 2 **METRIC**

PROJECT 1547670
G.W.P. 5260-13-00
DIST Northeast HWY 524
DATUM Geodetic
LOCATION N 5095980.5; E 292725.4 MTM ZONE 10 (LAT. 46.003539; LONG. -79.655904)
BOREHOLE TYPE 178 mm O.D. Continuous Flight Hollow Stem Augers; NQ Rock Coring
DATE November 24 and 25, 2016
ORIGINATED BY ACK
COMPILED BY SMD
CHECKED BY MCK/TZ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT (kN/m³)	GR	SA	SI	CL
226.9	GROUND SURFACE								
0.0	Silty sand, trace gravel, trace organics (FILL) Very loose Brown Moist	1	AS	-					
		2A / 2B	SS	4					
225.8 / 1.1	Sandy SILT to SILT and SAND, trace to some clay Very loose to loose Brown to grey Moist to wet	3	SS	8		0	24	69	7
	- Trace organics encountered between depths of about 1.5 m and 2.1 m	4	SS	3	CHEM				
		5	SS	WH		0	48	50	2
	- Trace organics (wood fragments) encountered between depths of about 3.8 m and 5.2 m	6	SS	WH					
	- Grey and wet below a depth of about 4.6 m	7	SS	WH		0	60	36	4
		8	SS	WH					
219.7 / 7.2	SILTY CLAY Firm Grey Moist to wet - Trace organics encountered between depths of about 7.6 m and 8.2 m	9	SS	WH					
		10	TO	PH	17.5				
		11	SS	5					
216.4 / 10.5	SILT and SAND to SAND, some silt Compact Grey Wet	12	SS	24					
		13	SS	50/0.10					
214.9 / 12.0	GRANITIC GNEISS (BEDROCK) SPLIT-SPOON AND AUGER REFUSAL Bedrock cored between depths of about 12.0 m and 15.1 m For bedrock coring details, refer to Record of Drillhole 16-03.	1	RC	REC 95%		RQD = 92%			
		2	RC	REC 93%		RQD = 71%			



Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-03

SHEET 2 OF 2

METRIC

PROJECT 1547670
G.W.P. 5260-13-00
LOCATION N 5095980.5; E 292725.4 MTM ZONE 10 (LAT. 46.003539; LONG. -79.655904)
ORIGINATED BY ACK
DIST Northeast HWY 524
BOREHOLE TYPE 178 mm O.D. Continuous Flight Hollow Stem Augers; NQ Rock Coring
COMPILED BY SMD
DATUM Geodetic
DATE November 24 and 25, 2016
CHECKED BY MCK/TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
211.8 15.1	END OF BOREHOLE NOTE: 1. Water level in augers measured at a depth of about 6.8 m below ground surface (Elev. 220.1 m) upon completion of drilling.										

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

PROJECT: 1547670

RECORD OF DRILLHOLE: 16-03

SHEET 1 OF 1

LOCATION: N 5095980.5 ;E 292725.4

DRILLING DATE: November 25, 2016

DATUM: Geodetic

DRILL RIG: CME 850 (Track-Mounted)

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Landcore Drilling Inc.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 16-03	214.87					
	Wireline Diamond Rock Coring; NQ Double Tube Core Barrel	Fresh, foliated, pink and black, medium grained, non-porous, very strong GRANITIC GNEISS - Quarizite vein at a depth of about 12.5 m	12.03	1	BC,, FO,CU,RO FO,UN,RO BC,, VN,UN,RO VN,PL,RO	3 3 3 1.5	1 1 2 2	UCS=154.9 MPa
13								
14				2	BC,, JN,PL,SM JN,PL,SM JN,PL,SM FO,PL,RO JN,IR,RO BC,,	 1 1 1 1.5 3	 4 4 4 2 2	
15			211.77 15.13					
		END OF DRILLHOLE NOTE: 1. Geographic Location: Latitiude: 46.003539° Longitude: -79.655904°						
16								
17								
18								
19								
20								
21								
22								

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: ACK

CHECKED: MCK/TZ

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 16-04

SHEET 1 OF 2 **METRIC**

PROJECT 1547670
G.W.P. 5260-13-00
LOCATION N 5096003.9; E 292739.5 MTM ZONE 10 (LAT. 46.003539; LONG. -79.655723)
ORIGINATED BY ACK
DIST Northeast HWY 524
BOREHOLE TYPE 178 mm O.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Rock Coring
COMPILED BY SMD
DATUM Geodetic
DATE November 22, 2016
CHECKED BY MCK/TZ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
227.0	GROUND SURFACE								
0.0	Silty sand, some gravel, trace organics (FILL) Brown Moist	1	AS	-					
226.1 / 0.9	- An approximately 0.2 m thick layer of clayey silt encountered at a depth of about 0.8 m.	2A / 2B	SS	3		226	○		
	SILTY SAND to SAND, trace silt, trace organics Very loose Brown Moist	3	SS	3		225	○		
224.9 / 2.1	SILT and SAND, trace clay to SAND, trace to some gravel, trace silt, trace clay Loose Brown to grey Moist to wet	4	SS	3			○	Non - Plastic	0 60 35 5
	- Trace organics encountered at a depth of about 3.0 m	5	SS	2	∇	224			
	- Wet below a depth of about 3.3 m	6	SS	5		223	○		
	- Grey below a depth of about 5.0 m	7	SS	2		222		CHEM	
	An approximately 0.01 m thick layer of organics (rootlets and wood fragments) encountered at a depth of about 6.4 m	8	SS	5		221 / 220	○		
219.5 / 7.7	GRAVEL, trace to some sand Black and pink Wet	9	SS	50/0.10					
	SPLIT-SPOON AND CASING REFUSAL GRANITIC GNEISS (BEDROCK) Bedrock cored between depths of about 7.7 m and 14.0 m. For bedrock coring details refer to Record of Drillhole 16-04.	1	RC	REC 100%		219 / 218			RQD = 99%
		2	RC	REC 97%		217			RQD = 97%
		3	RC	REC 100%		216			RQD = 100%
		4	RC	REC 100%		215			RQD = 99%
		5	RC	REC 100%		214			RQD = 100%
213.0 / 14.0	END OF BOREHOLE								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 16-04

SHEET 2 OF 2

METRIC

PROJECT 1547670

G.W.P. 5260-13-00

LOCATION N 5096003.9; E 292739.5 MTM ZONE 10 (LAT. 46.003539; LONG. -79.655723)

ORIGINATED BY ACK

DIST Northeast HWY 524

BOREHOLE TYPE 178 mm O.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Rock Coring

COMPILED BY SMD

DATUM Geodetic

DATE November 22, 2016

CHECKED BY MCK/TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	NOTES: 1. Water level in augers measured at a depth of about 3.6 m below ground surface (Elev. 223.4 m) prior wash boring.										

+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

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RECORD OF DRILLHOLE: 16-04

PROJECT: 1547670

SHEET 1 OF 1

LOCATION: N 5096003.9 ;E 292739.5

DRILLING DATE: November 22, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850 (Track-Mounted)

DRILLING CONTRACTOR: Landcore Drilling Inc.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja
		Continued from Record of Borehole 16-04	219.30				
	Wireline Diamond Rock Coring; NQ Double Tube Core Barrel	Fresh, foliated, pink and black, medium-grained, non-porous, very strong GRANITIC GNEISS	7.70		BC,,		
8				1			
9							
10				2	FO,PL,RO	1.5	1
					BC,,		
11				3			
				4	BC,,		
12					JN,PL,RO	1.5	2
13				5	FO,PL,RO	1.5	1
14			213.02 / 13.98				
		END OF DRILLHOLE					

NOTE:

1. Geographic Location:

Latitiude: 46.003539°
Longitude: -79.655723°

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: ACK
CHECKED: MCK

GTA-RCK 004 S:\CLIENTS\MTO\HWY_524_COMMANDACREEKBRIDGE\02_DATA\GINT\1547670.GPJ GAL-MISS.GDT 04/07/17

RECORD OF BOREHOLE No 16-05

SHEET 1 OF 1 **METRIC**

PROJECT 1547670
G.W.P. 5260-13-00
LOCATION N 5096010.4; E 292733.4 MTM ZONE 10 (LAT. 46.003809; LONG. -79.655802)
ORIGINATED BY ACK
DIST Northeast HWY 524
BOREHOLE TYPE 178 mm O.D. Hollow Stem Augers, NW Casing, Wash Boring; NQ Rock Coring
COMPILED BY SMD
DATUM Geodetic
DATE November 23, 2016
CHECKED BY MCK/TZ

ELEV DEPTH (m)	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
226.0	GROUND SURFACE										
0.0	SILT and SAND, trace clay, trace organics to a depth of about 0.6 m; Very loose to loose; Brown; Moist	1	AS	-							
		2	SS	4	~28						
		3	SS	1	~28	Non - Plastic	0	65	33	2	
	- Wet below a depth of about 2.3 m	4A / 4B	SS	WR*							
223.2 / 2.8	Silty SAND, trace organics to a depth of about 5.2 m; Very loose to compact; Grey; Wet	5	SS	1	~28		0	76	24	0	
		6	SS	5							
		7	SS	3	~29	Non - Plastic OC=4.1%					
	- Trace gravel below a depth of about 6.1 m	8	SS	55/0.29							
219.5 / 6.5	SPLIT-SPOON AND CASING REFUSAL; GRANITIC GNEISS (BEDROCK); Bedrock cored between depths of about 6.5 m and 9.6 m depth. For bedrock coring details refer to Record of Drillhole 16-05.	1	RC	REC 100%							RQD = 96%
		2	RC	REC 100%							RQD = 95%
216.4 / 9.6	END OF BOREHOLE										

Groundwater: water level symbol at elevation about 223.9 m.

Elevation scale: 225, 224, 223, 222, 221, 220, 219, 218, 217

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

NOTES:

*Unrepresentative SPT 'N'-value due to unbalanced hydrostatic head.

1. Water level measurements in standpipe piezometer:

Date	Depth(m)	Elev.(m)
23/11/16	2.1	223.9
24/11/16	2.1	223.9
25/11/16	2.1	223.9
26/11/16	2.1	223.9
16/12/16	2.2	223.8

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

RECORD OF DRILLHOLE: 16-05

PROJECT: 1547670
SHEET 1 OF 1

LOCATION: N 5096010.4 ;E 292733.4
DRILLING DATE: November 16, 2016
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG: CME 850 (Track-Mounted)
DRILLING CONTRACTOR: Landcore Drilling Inc.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 16-05	219.46					
7	Wireline Diamond Rock Coring, NQ Double Tube Core Barrel	Fresh, foliated, pink and black, medium-grained, non-porous, very strong GRANITIC GNEISS	6.54	1	BC,,			UCS=107.6 MPa
		- Basalt vein at a depth of about 7.5 m						
8				2	JN,PL,RO	1.5	2	Filter Sand
					JN,PL,RO	1.5	1	
9					FO,PL,SM	1	2	
					JN,IR,RO	3	2	
					BC,,			
			216.39		FO,PL,SM	1	2	
		END OF DRILLHOLE	9.61					

NOTES:

1. For complete standpipe piezometer installation details and water level measurements, refer to Record of Borehole 16-05.

2. Geographic Location:

Latitiude: 46.003809°
Longitude: -7

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: ACK
CHECKED: MCK/TZ

GTA-RCK 004 S:\CLIENTS\MTO\HWY_524_COMMANDACREEKBRIDGE\02_DATA\GINT\1547670.GPJ GAL-MISS.GDT 04/07/17

Golder Associates

RECORD OF BOREHOLE No 16-06

SHEET 1 OF 1 **METRIC**

PROJECT 1547670
G.W.P. 5260-13-00
LOCATION N 5096028.1; E 292744.9 MTM ZONE 10 (LAT. 46.00968; LONG. -79.655654)
ORIGINATED BY ACK
DIST Northeast HWY 524
BOREHOLE TYPE 178 mm O.D. Continuous Flight Hollow Stem Augers
COMPILED BY SMD
DATUM Geodetic
DATE November 21, 2016
CHECKED BY MCK/TZ

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
227.0	GROUND SURFACE					
0.0	ASPHALT					
0.2	Silty sand, trace to some clay, trace gravel, trace organics (FILL) Very loose Brown Moist	1	AS	-		
		2	SS	4		
225.5 / 1.5	CLAYEY SILT, some sand Soft to firm Brown moist	3	SS	7		0 13 77 10
		4	SS	6		
		5	SS	3		
223.3 / 3.7	SILT and SAND, trace clay to SAND, trace gravel, trace clay Very loose to loose Brown to grey Wet - Trace organics encountered between depths of about 4.6 m and 5.2 m	6	SS	7		
		7	SS	3	▽	0 66 33 1
		8	SS	1		
		9	SS	1		
218.6 / 8.4	END OF BOREHOLE AUGER REFUSAL					

ELEVATION SCALE: 226, 225, 224, 223, 222, 221, 220, 219

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

NOTES:

1. Water level in open borehole measured at a depth of about 4.6 m below ground surface (Elev. 222.4 m) upon completion of drilling.

2. Borehole caved to a depth of about 5.5 m below ground surface (Elev. 221.5 m) upon removal of augers.

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

RECORD OF DRILLHOLE: 16-06

PROJECT: 1547670

SHEET 1 OF 1

LOCATION: N 5096028.1 ;E 292744.9

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850 (Track-Mounted)

DRILLING CONTRACTOR: Landcore Drilling Inc.

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	Diametral Point Load Index (MPa) 2 4 6	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		227.00																	
0				0.00																	
1																					
2																					
3																					
4																					
5																					
6																					
7																					
8																					
9																					
10																					

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: ACK

CHECKED: MCK

APPENDIX B

Geotechnical Laboratory Test Results

TABLE B1
SUMMARY OF UNIAXIAL COMPRESSIVE STRENGTH TEST RESULTS

Borehole Number (Core Run)	Approximate Sample Depth (m)	Approximate Sample Elevation (m)	Rock Type	Core Diameter (mm)	Uniaxial Compressive Strength (MPa)
16-02 (Run No. 1)	14.1	213.0	Basalt	47.5	97.7
16-03 (Run No. 1)	12.3	214.6	Granitic Gneiss	47.5	154.9
16-05 (Run No. 1)	6.8	219.2	Granitic Gneiss	47.5	107.6

Compiled By: AK

Reviewed By: TZ

TABLE B2
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1547670	RUN NUMBER	1
PROJECT NAME	Hwy 524 Commanda Creek Bridge Replacement	SAMPLE DEPTH, m	14.02-14.21
BOREHOLE NUMBER	16-02	DATE:	Jan. 26, 2017

TEST CONDITIONS

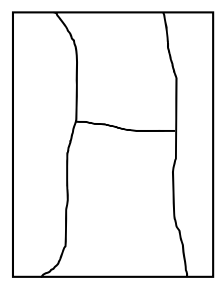
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.65	WATER CONTENT, (specimen) %	0.00
SAMPLE DIAMETER, cm	4.76	UNIT WEIGHT, kN/m^3	29.38
SAMPLE AREA, cm^2	17.77	DRY UNIT WT., kN/m^3	29.38
SAMPLE VOLUME, cm^3	189.11	SPECIFIC GRAVITY	-
WET WEIGHT, g	566.71	VOID RATIO	-
DRY WEIGHT, g	566.71		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	97.7

REMARKS:

CHECKED BY: TZ

TABLE B3
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1547670	RUN NUMBER	1
PROJECT NAME	Hwy 524 Commanda Creek Bridge Replacement	SAMPLE DEPTH, m	12.20-12.33
BOREHOLE NUMBER	16-03	DATE:	Jan. 26, 2017

TEST CONDITIONS

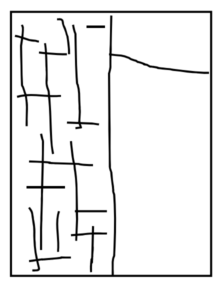
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.13

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.15	WATER CONTENT, (specimen) %	0.00
SAMPLE DIAMETER, cm	4.76	UNIT WEIGHT, kN/m^3	25.62
SAMPLE AREA, cm^2	17.80	DRY UNIT WT., kN/m^3	25.62
SAMPLE VOLUME, cm^3	180.72	SPECIFIC GRAVITY	-
WET WEIGHT, g	472.29	VOID RATIO	-
DRY WEIGHT, g	472.29		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	154.9

REMARKS:

CHECKED BY: TZ

TABLE B4
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1547670	RUN NUMBER	1
PROJECT NAME	Hwy 524 Commanda Creek Bridge Replacement	SAMPLE DEPTH, m	6.7-6.94
BOREHOLE NUMBER	16-05	DATE:	Jan. 26, 2017

TEST CONDITIONS

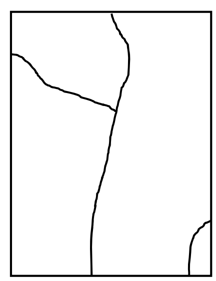
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.52	WATER CONTENT, (specimen) %	0.00
SAMPLE DIAMETER, cm	4.76	UNIT WEIGHT, kN/m^3	25.71
SAMPLE AREA, cm^2	17.78	DRY UNIT WT., kN/m^3	25.71
SAMPLE VOLUME, cm^3	187.07	SPECIFIC GRAVITY	-
WET WEIGHT, g	490.70	VOID RATIO	-
DRY WEIGHT, g	490.70		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

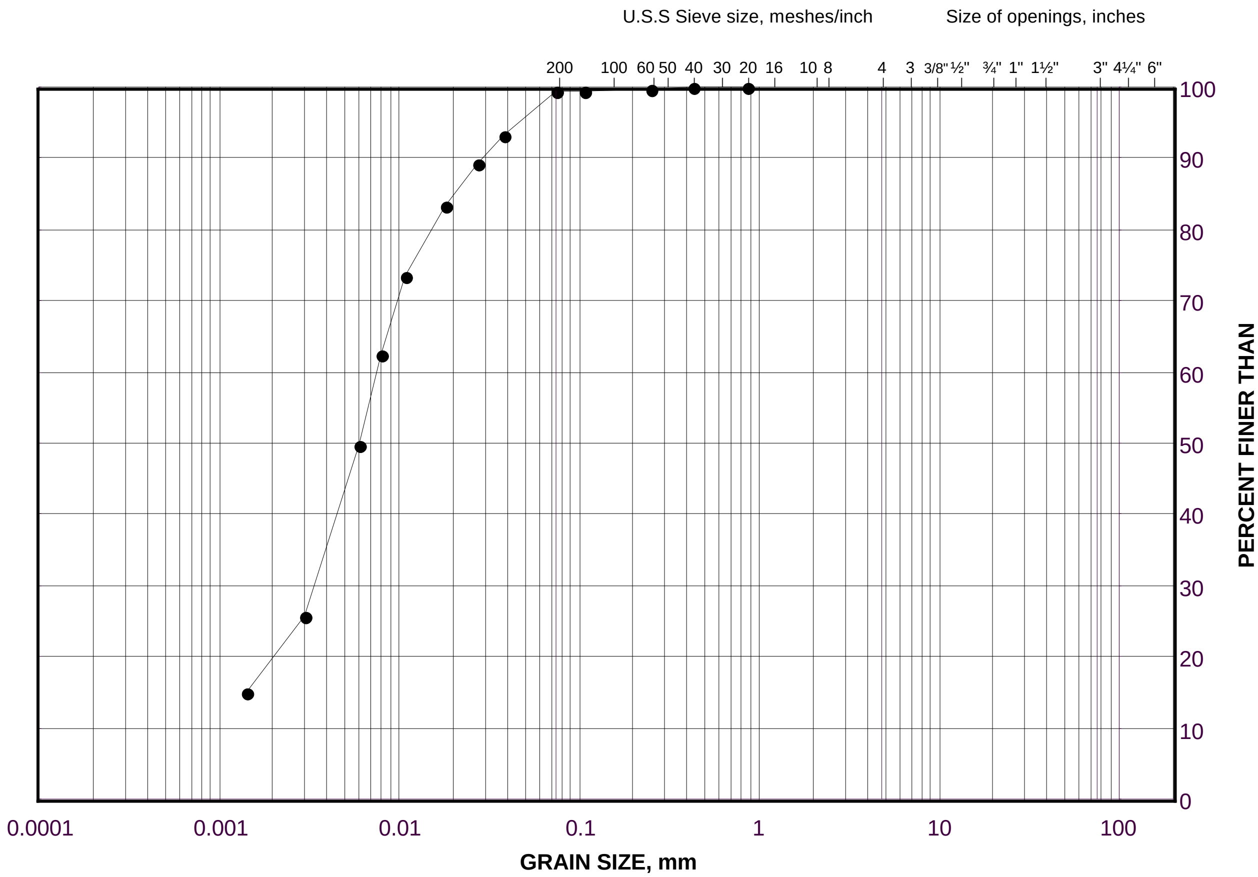
STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	107.6

REMARKS:

CHECKED BY: TZ

Golder Associates Ltd.

GRAIN SIZE DISTRIBUTION Clayey Silt (Fill)	FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-01	4	224.7

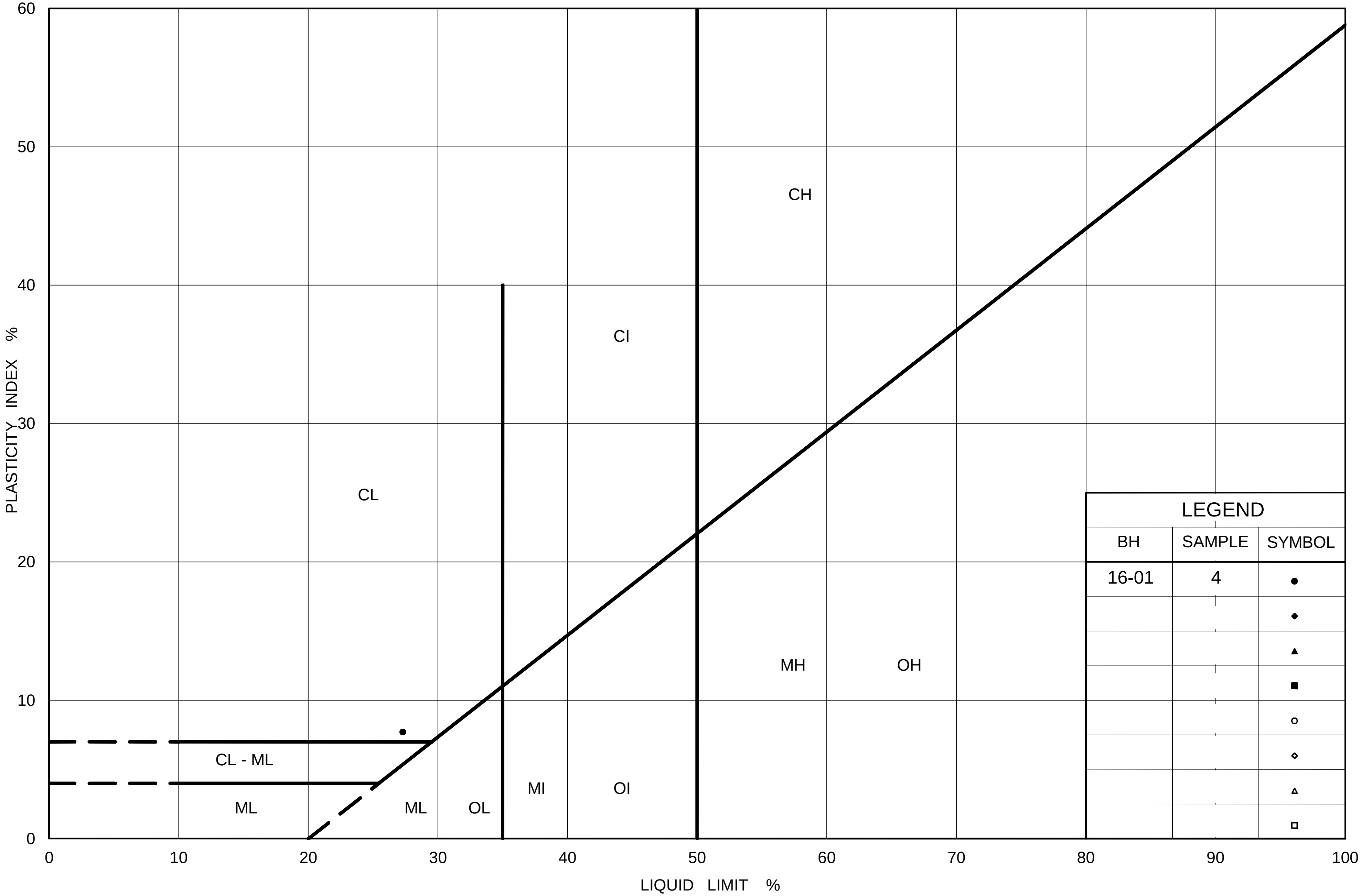
Project Number: 1547670

Checked By: TZ

Golder Associates

Date: 21-Feb-17

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt (Fill)

Figure No. B2

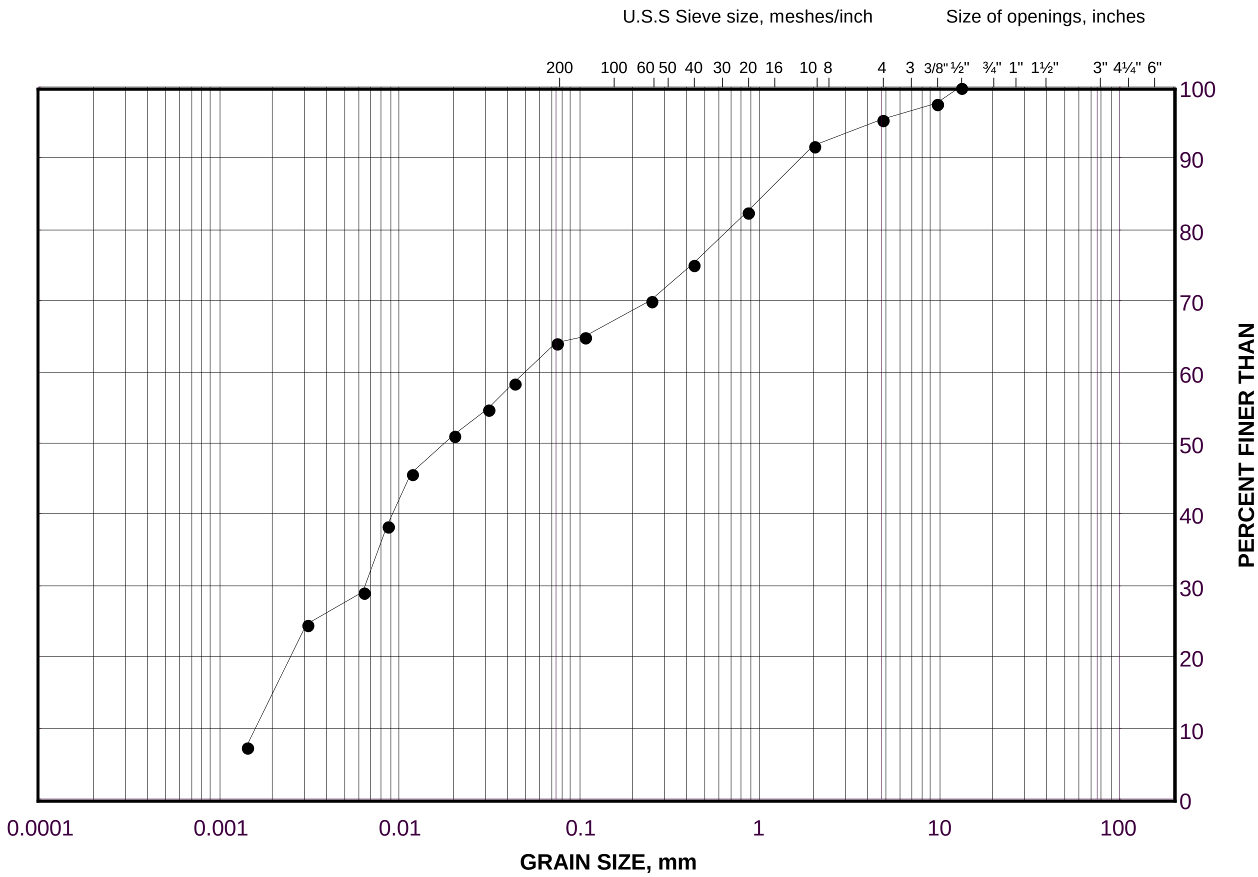
Project No. 1547670

Checked By: TZ

GRAIN SIZE DISTRIBUTION

Silt and Sand

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-02	2	226.0

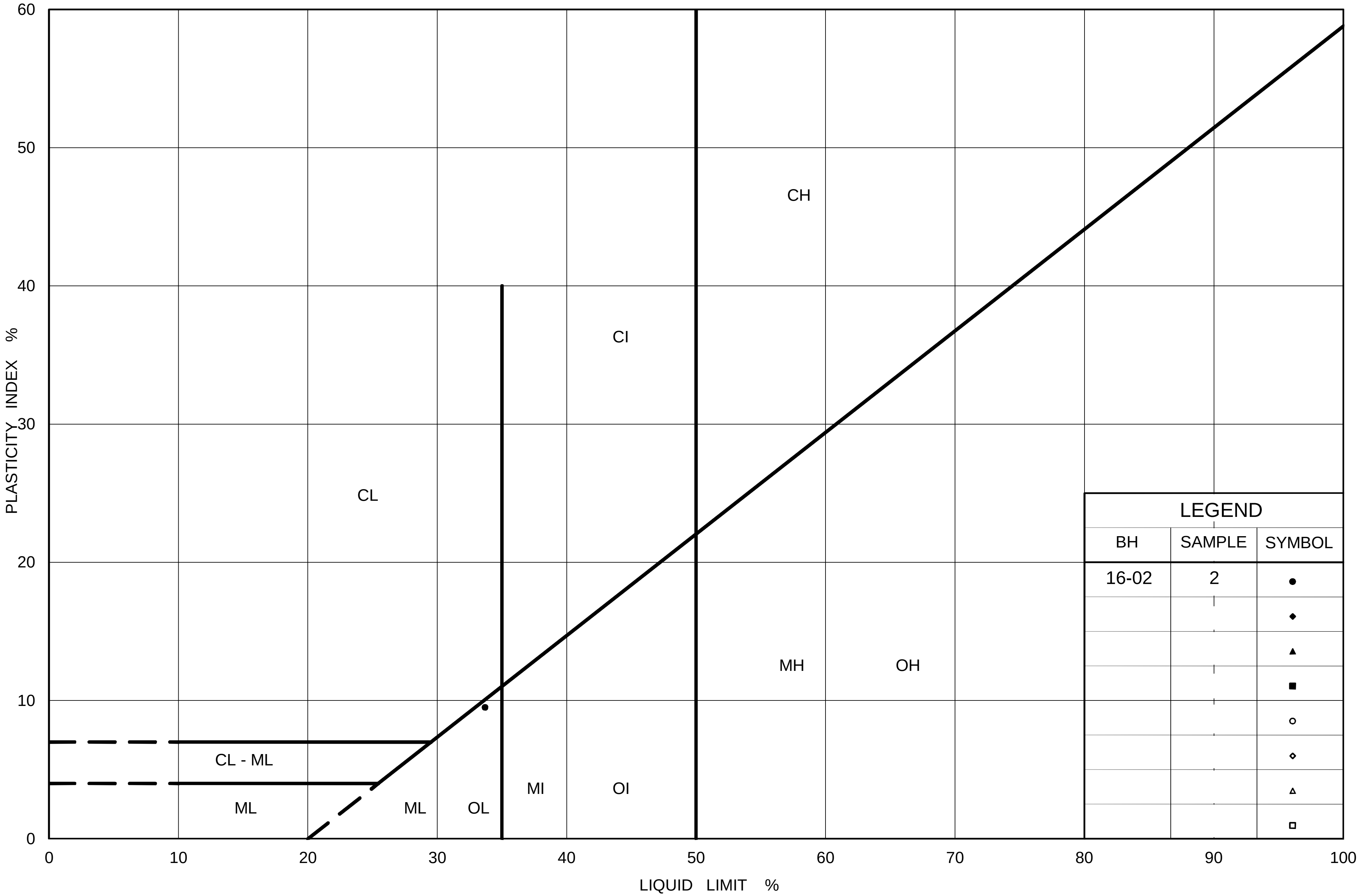
Project Number: 1547670

Checked By: TZ

Golder Associates

Date: 21-Feb-17

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHAR
Silt and Sand (Slight Plasticity)

Figure No. B4

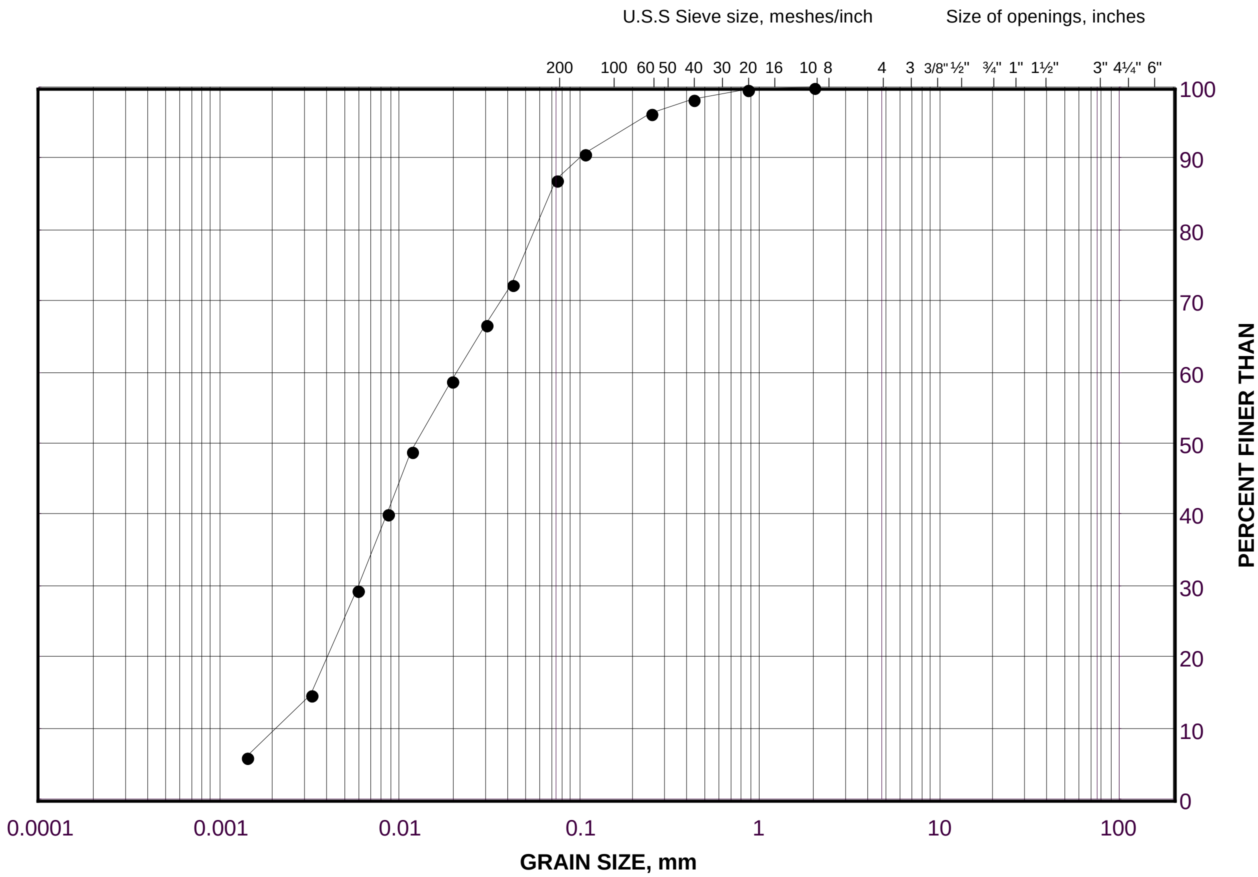
Project No. 1547670

Checked By: TZ

GRAIN SIZE DISTRIBUTION

Clayey Silt

FIGURE B5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-06	3	225.2

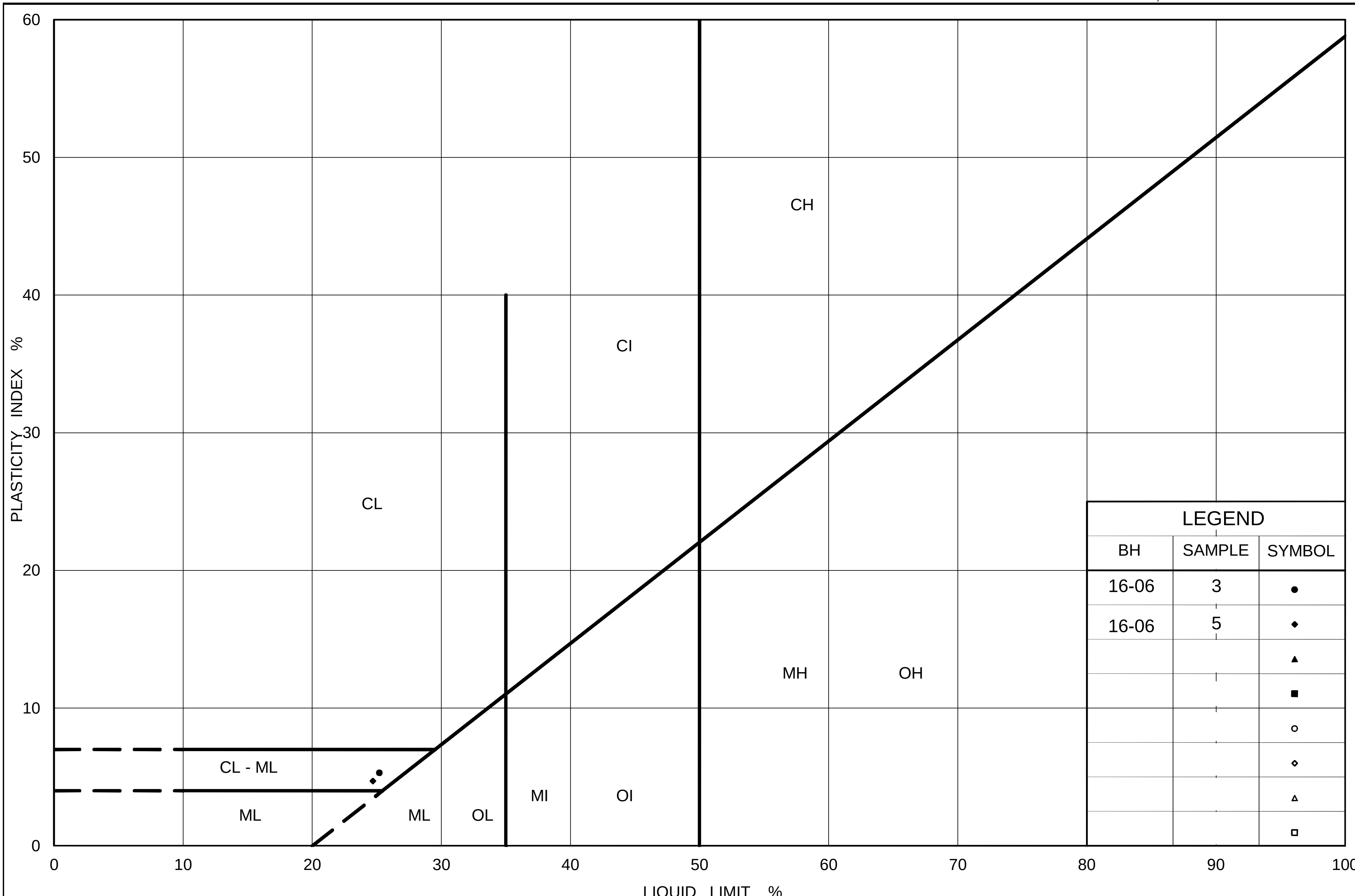
Project Number: 1547670

Checked By: TZ

Golder Associates

Date: 21-Feb-17

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
16-06	3	●
16-06	5	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

Figure No. B6

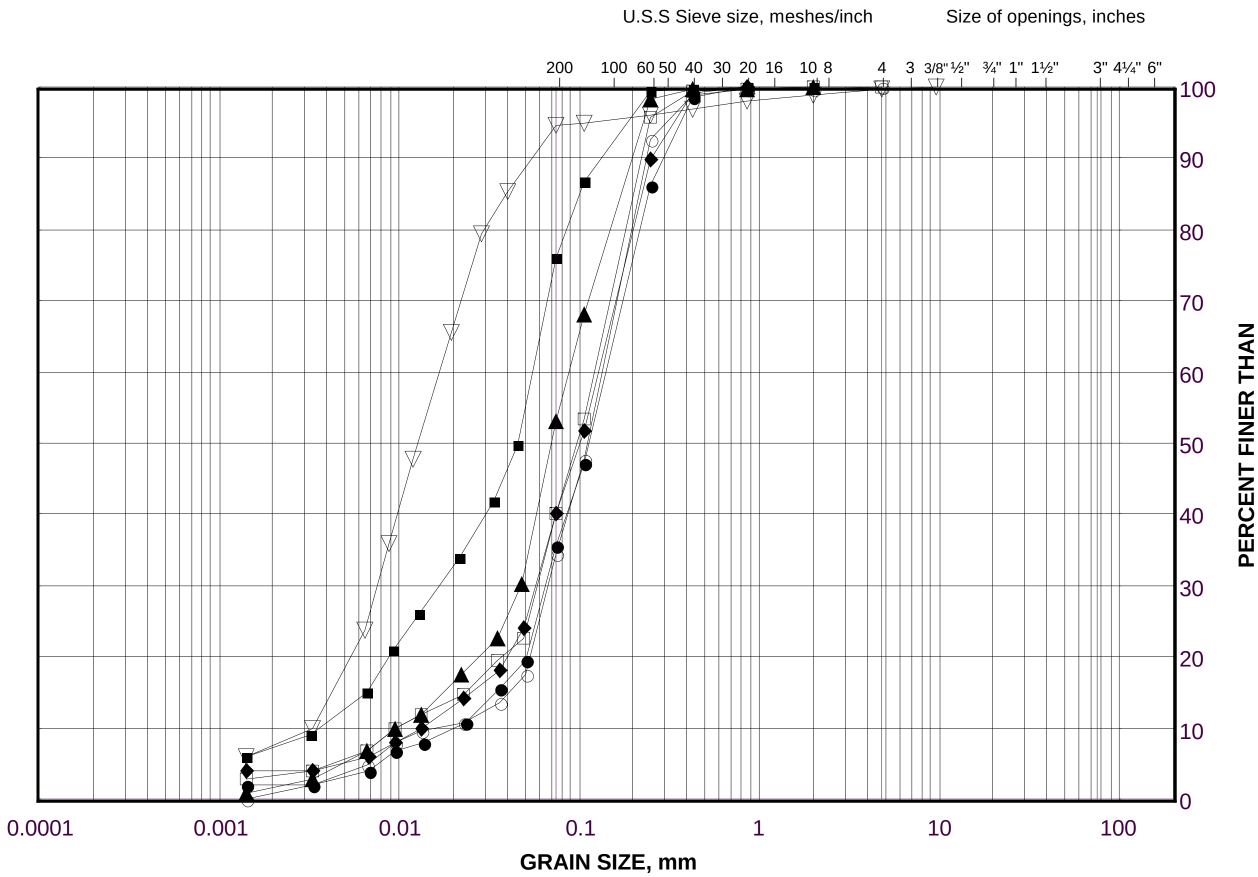
Project No. 1547670

Checked By: TZ

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt to Silt and Sand (Upper)

FIGURE B7A



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-05	3	224.2
■	16-03	3	225.1
◆	16-04	4	224.4
▲	16-03	5	223.5
▽	16-01	6A	223.3
❍	16-06	7	222.1
☐	16-03	7	222.0

Project Number: 1547670

Checked By: TZ

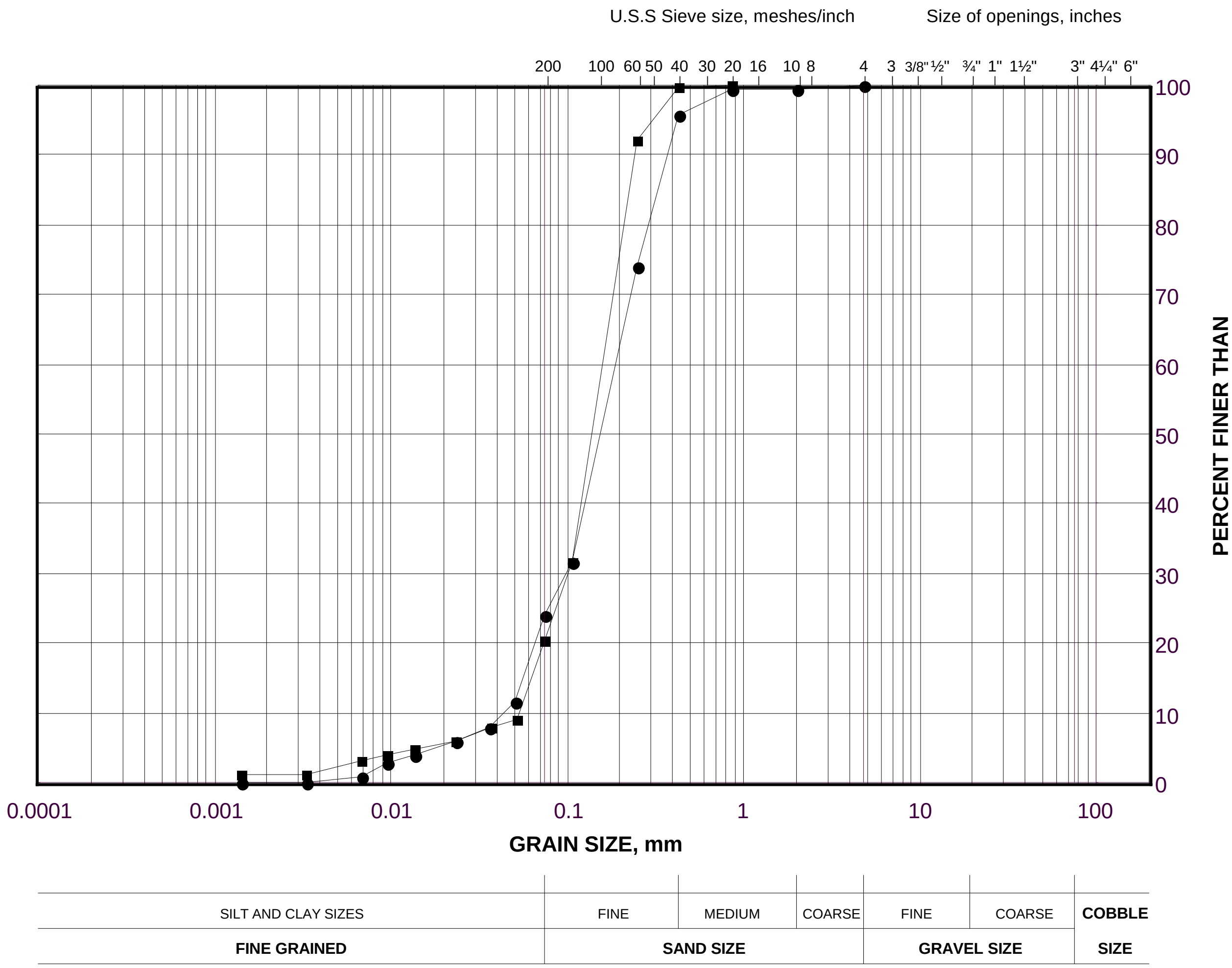
Golder Associates

Date: 21-Feb-17

GRAIN SIZE DISTRIBUTION

Silty Sand (Upper)

FIGURE B7B



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-05	5	222.6
■	16-02	6	223.0

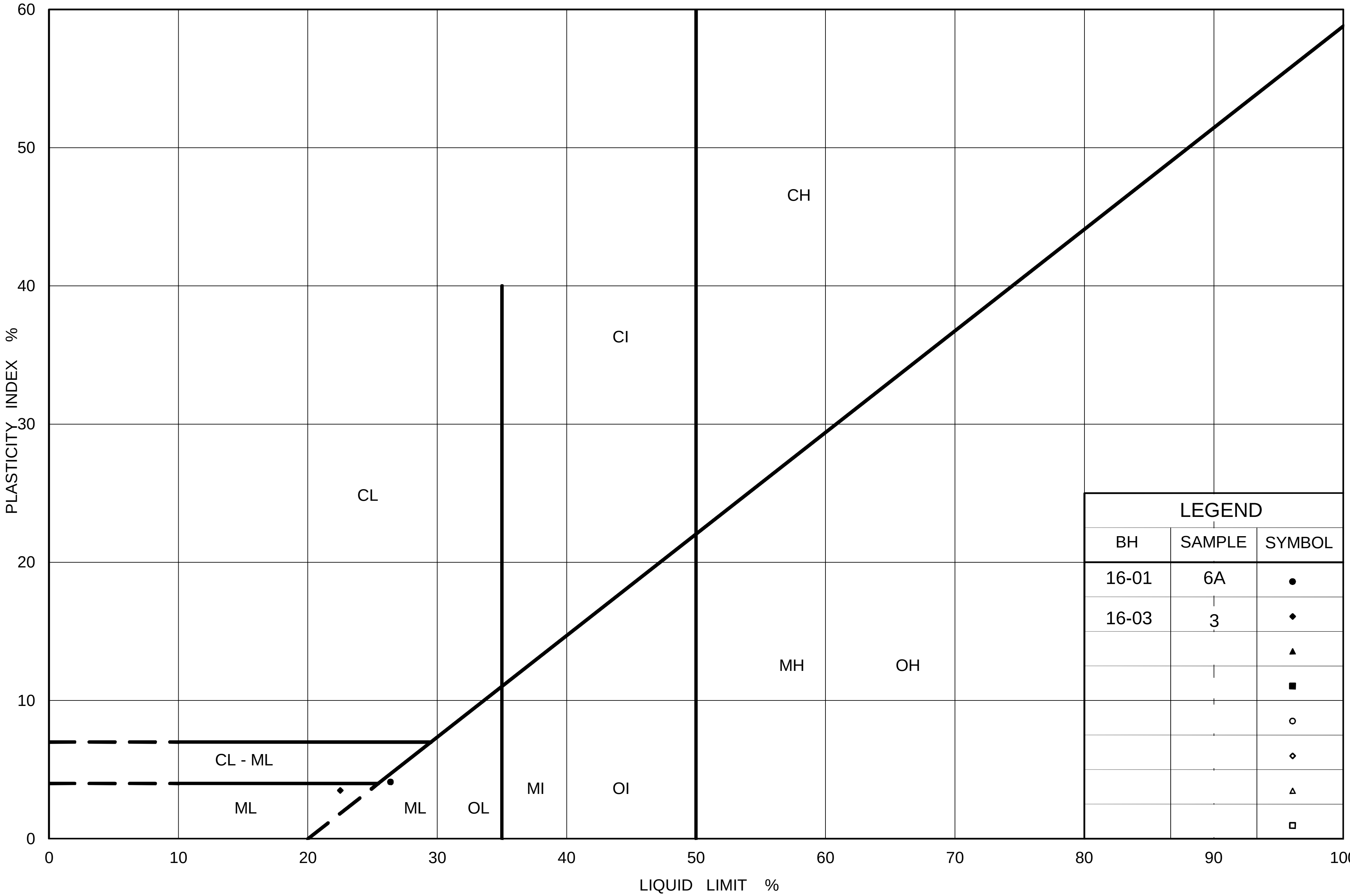
Project Number: 1547670

Checked By: TZ

Golder Associates

Date: 21-Feb-17

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt and Sandy Silt (Upper)
(Slight Plasticity)

Figure No. B8

Project No. 1547670

Checked By: TZ

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 1 OF 3		FIGURE B9 Page 1 of 3
TEST STAGE	A	B
BOREHOLE NUMBER	16-05	
SAMPLE NUMBER	6	
SAMPLE DEPTH, (m)	-	
SAMPLE HEIGHT, (mm)	26.50	26.86
SAMPLE LENGTH, (mm)	60.00	60.00
WATER CONTENT, BEFORE TEST, (%)	28.42	28.42
NORMAL (CONSOLIDATION) STRESS, (kPa)	30.00	60.00
WATER CONTENT, AFTER TEST, (%)	25.03	23.30
DISPLACEMENT RATE, mm/min	0.0048	0.0048
TIME TO FAILURE, HOURS	14	15
PEAK SHEAR STRESS[1], (kPa)	30.56	59.00
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	4.09	4.20
DRY DENSITY, initial, Mg/m^3	1.46	1.44
WET DENSITY, initial, Mg/m^3	1.87	1.85

TEST NOTES:

1. In the absence of a peak, the shear stress reported is at 10 percent relative horizontal displacement (ASTM D3080).
2. Specimens were lightly pressed into the box at as is moisture content; visible organics were removed from the sample.

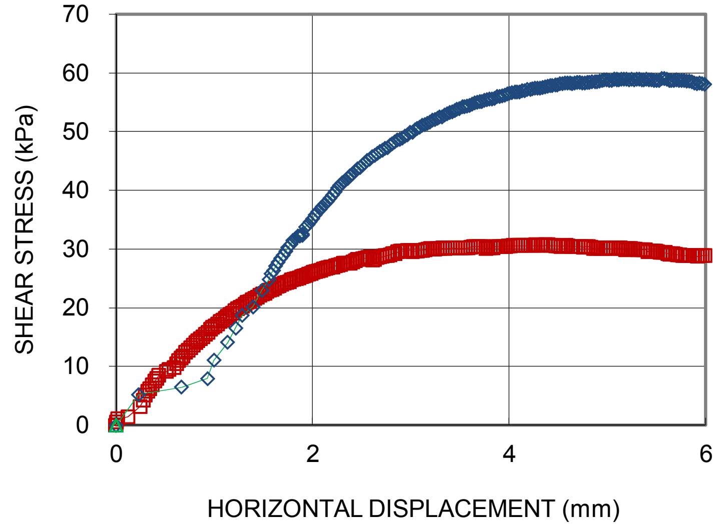
Date: 2/28/2017 Prepared By: LH

Project No. 1547670 **Golder Associates** Checked By: MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 2 OF 3	FIGURE B9 Page 2 of 3

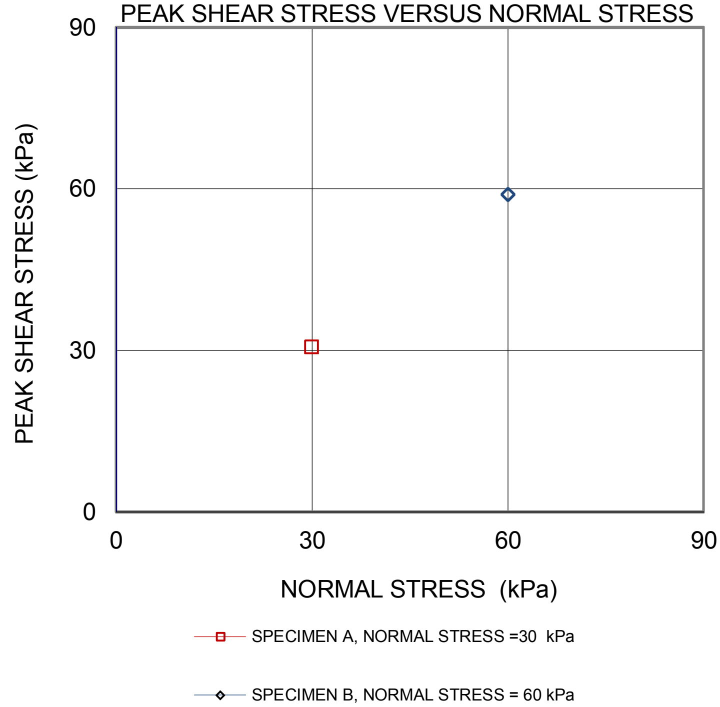
16-05 SA6

SHEAR STRESS VERSUS HORIZONTAL DISPLACEMENT



16-05 SA6

PEAK SHEAR STRESS VERSUS NORMAL STRESS

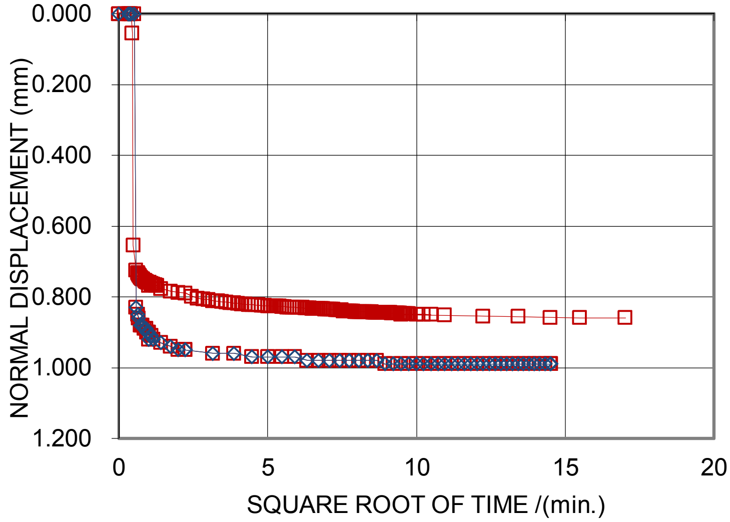


Date: 2/28/2017		Prepared By:	LH
Project No. 1547670	**Golder Associates**	Checked By:	MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 3 OF 3	FIGURE B9 Page 3 of 3

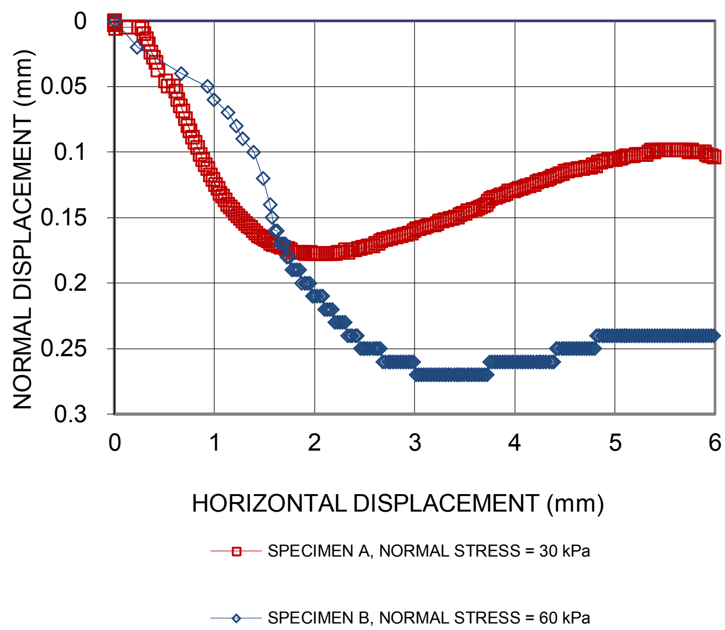
16-05 SA6

NORMAL DISPLACEMENT VERSUS SQUARE ROOT OF TIME



16-05 SA6

NORMAL DISPLACEMENT VERSUS HORIZONTAL DISPLACEMENT

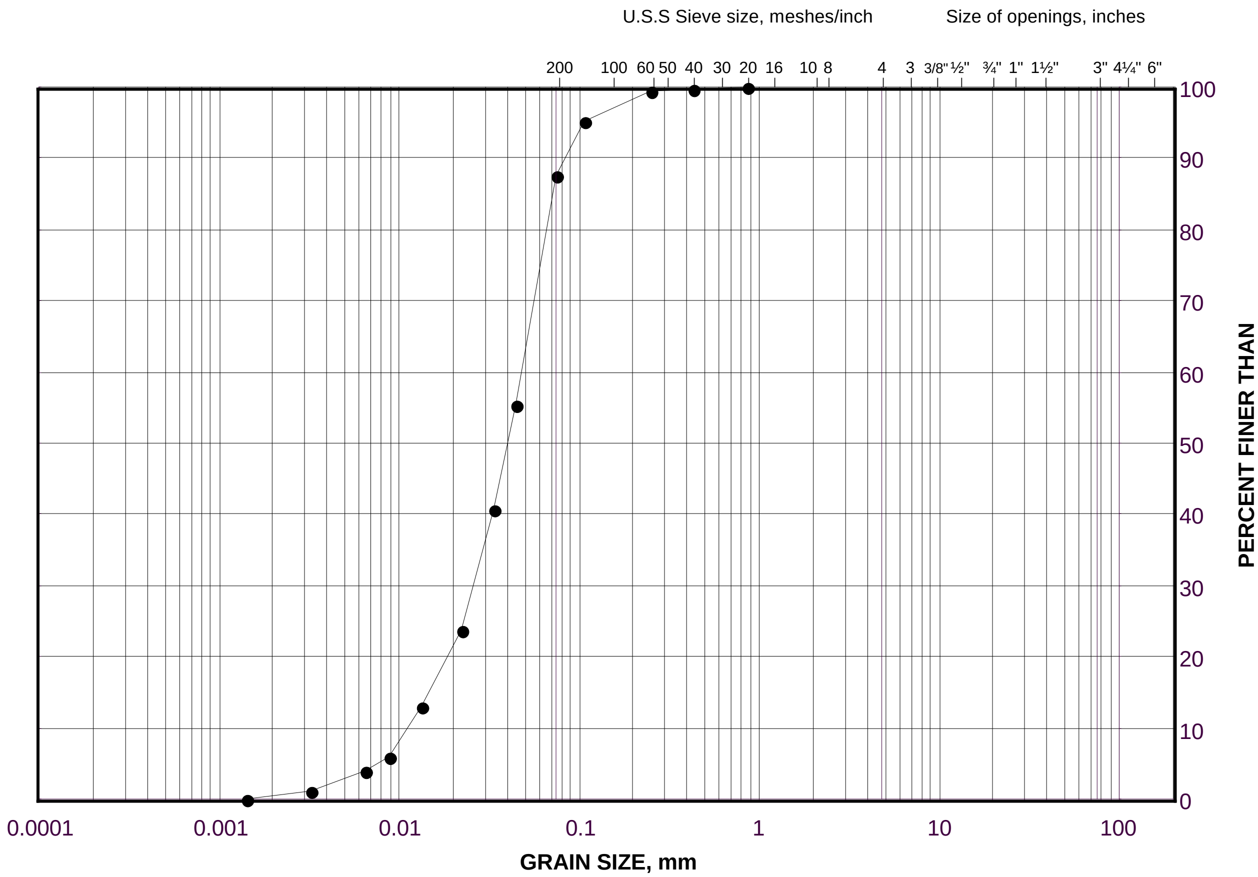


Date:	2/28/2017		Prepared By:	LH
Project No.	1547670	**Golder Associates**	Checked By:	MM

GRAIN SIZE DISTRIBUTION

Silt (Interlayer)

FIGURE B10



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-01	9A	219.6

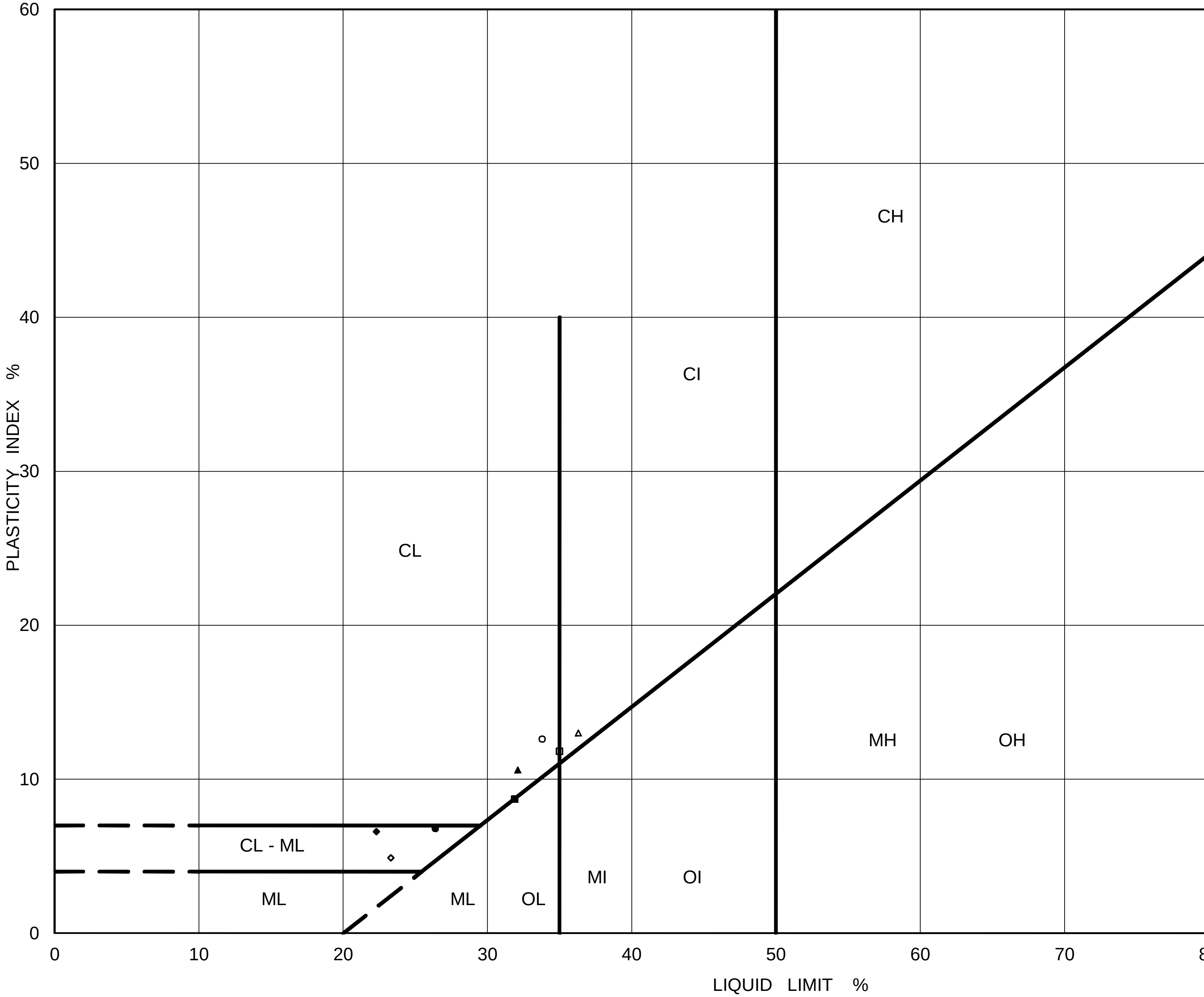
Project Number: 1547670

Checked By: TZ

Golder Associates

Date: 21-Feb-17

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
16-01	7	●
16-01	8	◆
16-01	9B	▲
16-01	10	■
16-02	9	○
16-02	10	◇
16-03	9	△
16-03	10	□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. B11

Project No. 1547670

Checked By: TZ

CONSOLIDATION TEST SUMMARY

SAMPLE IDENTIFICATION

Project Number	1547670	Sample Number	10
Borehole Number	16-03	Sample Depth, m	8.38 - 8.84

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	12/09/2016		
Date Completed	01/02/2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m3	17.45
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m3	12.05
Area, cm2	31.42	Specific Gravity, measured	2.74
Volume, cm3	59.23	Solids Height, cm	0.85
Water Content, %	44.81	Volume of Solids, cm3	26.56
Wet Mass, g	105.38	Volume of Voids, cm3	32.67
Dry Mass, g	72.77	Degree of Saturation, %	99.82

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0	1.885	1.230	1.885				
6	1.877	1.221	1.881	47	1.60E-02	7.13E-04	1.12E-06
11	1.870	1.212	1.873	70	1.06E-02	8.28E-04	8.62E-07
21	1.858	1.198	1.864	71	1.04E-02	6.17E-04	6.27E-07
40	1.843	1.181	1.851	83	8.75E-03	4.04E-04	3.46E-07
79	1.821	1.154	1.832	113	6.30E-03	3.02E-04	1.86E-07
123	1.805	1.136	1.813	187	3.73E-03	1.91E-04	6.98E-08
40	1.814	1.146	1.810				
11	1.821	1.155	1.818				
40	1.814	1.146	1.818	46	1.52E-02	1.28E-04	1.92E-07
123	1.799	1.129	1.807	47	1.47E-02	9.61E-05	1.39E-07
157	1.788	1.115	1.793	113	6.03E-03	1.84E-04	1.09E-07
313	1.693	1.003	1.740	130	4.94E-03	3.21E-04	1.56E-07
625.18	1.608	0.902	1.651	113	5.11E-03	1.44E-04	7.23E-08
1248.92	1.524	0.803	1.566	130	4.00E-03	7.17E-05	2.81E-08
2495.15	1.444	0.708	1.484	107	4.36E-03	3.41E-05	1.46E-08
1248.92	1.453	0.719	1.448				
312.81	1.477	0.747	1.465				
79.06	1.498	0.772	1.487				
20.59	1.524	0.803	1.511				
5.92	1.546	0.829	1.535				

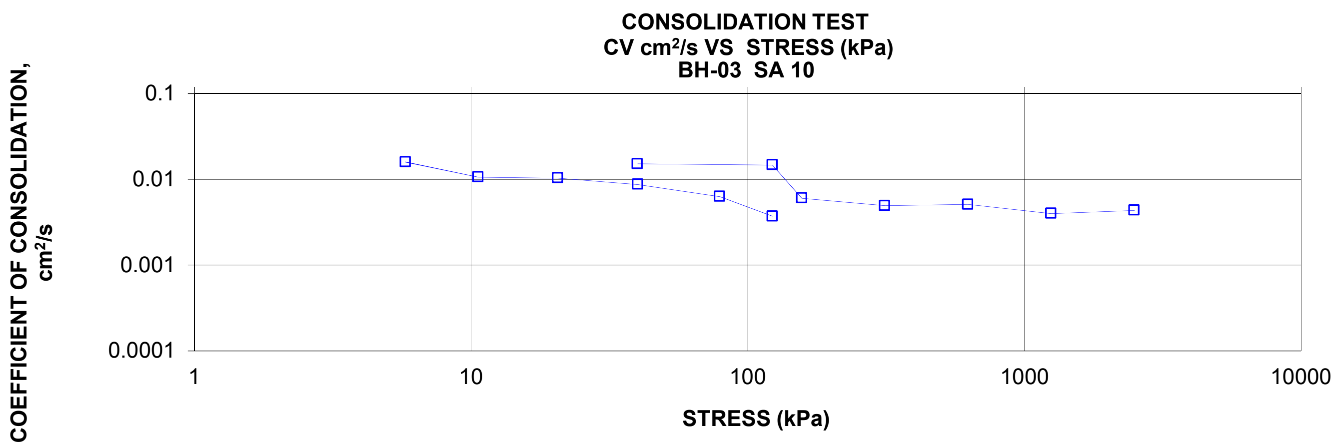
Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t90 estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen taken 19-28cm from top of the tube.

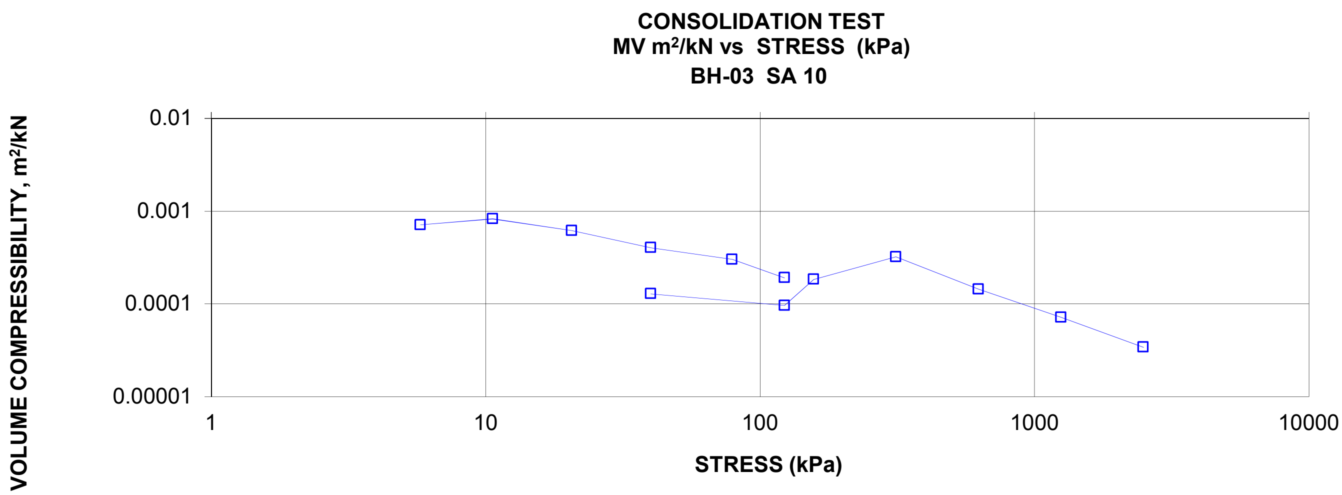
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

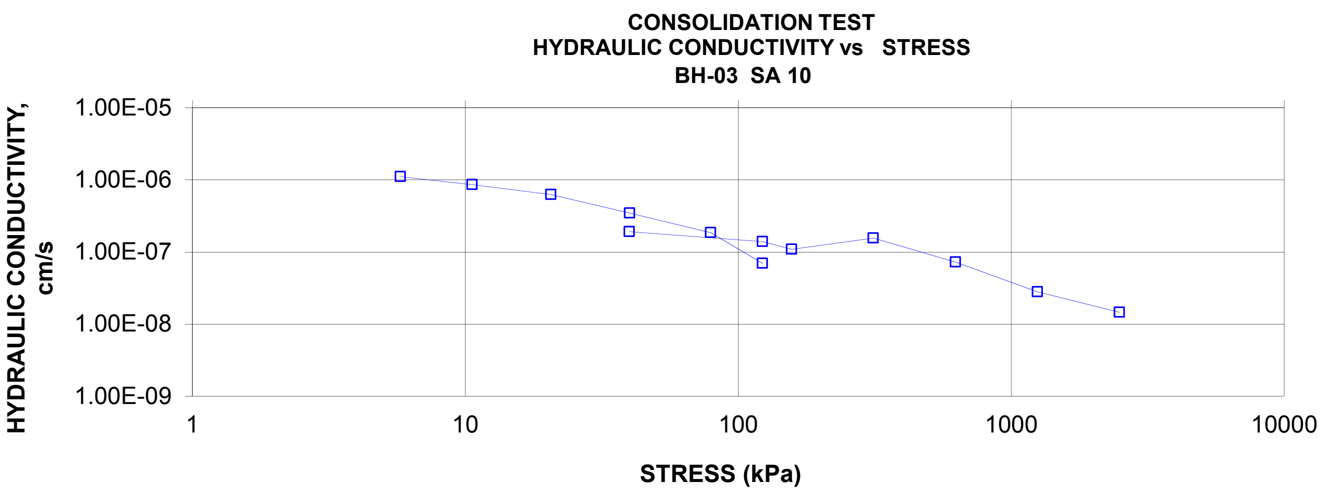
Sample Height, cm	1.55	Unit Weight, kN/m3	19.46
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m3	14.69
Area, cm2	31.42	Specific Gravity, measured	2.74
Volume, cm3	48.57	Solids Height, cm	0.845
Water Content, %	32.44	Volume of Solids, cm 3	26.56
Wet Mass, g	96.38	Volume of Voids, cm 3	22.01
Dry Mass, g	72.77		

Prepared By: TG **Golder Associates** Checked By: MT

CONSOLIDATION TEST SUMMARY	FIGURE B12 Page 2 of 4







Project No. 15447670

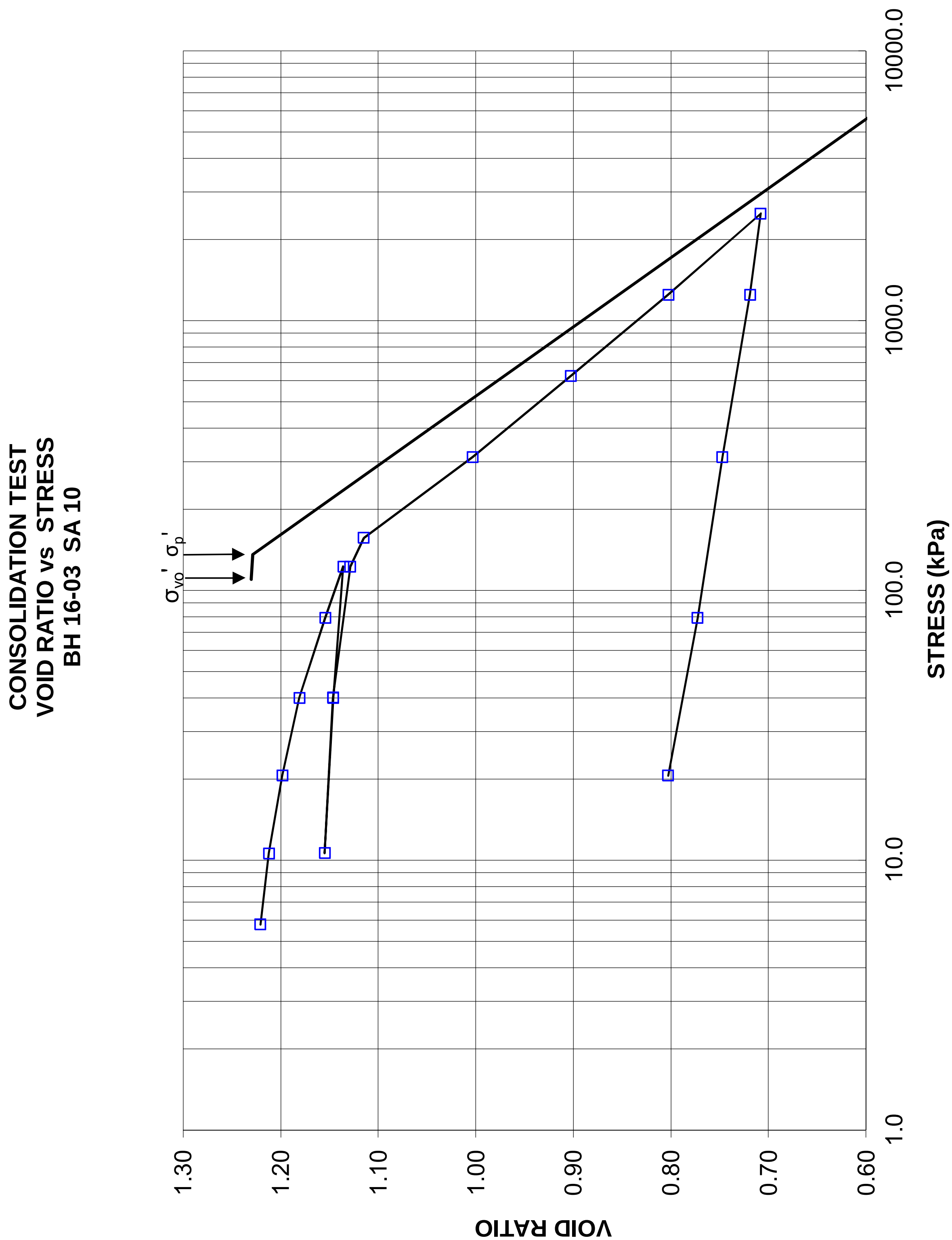
Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B12
Page 3 of 4

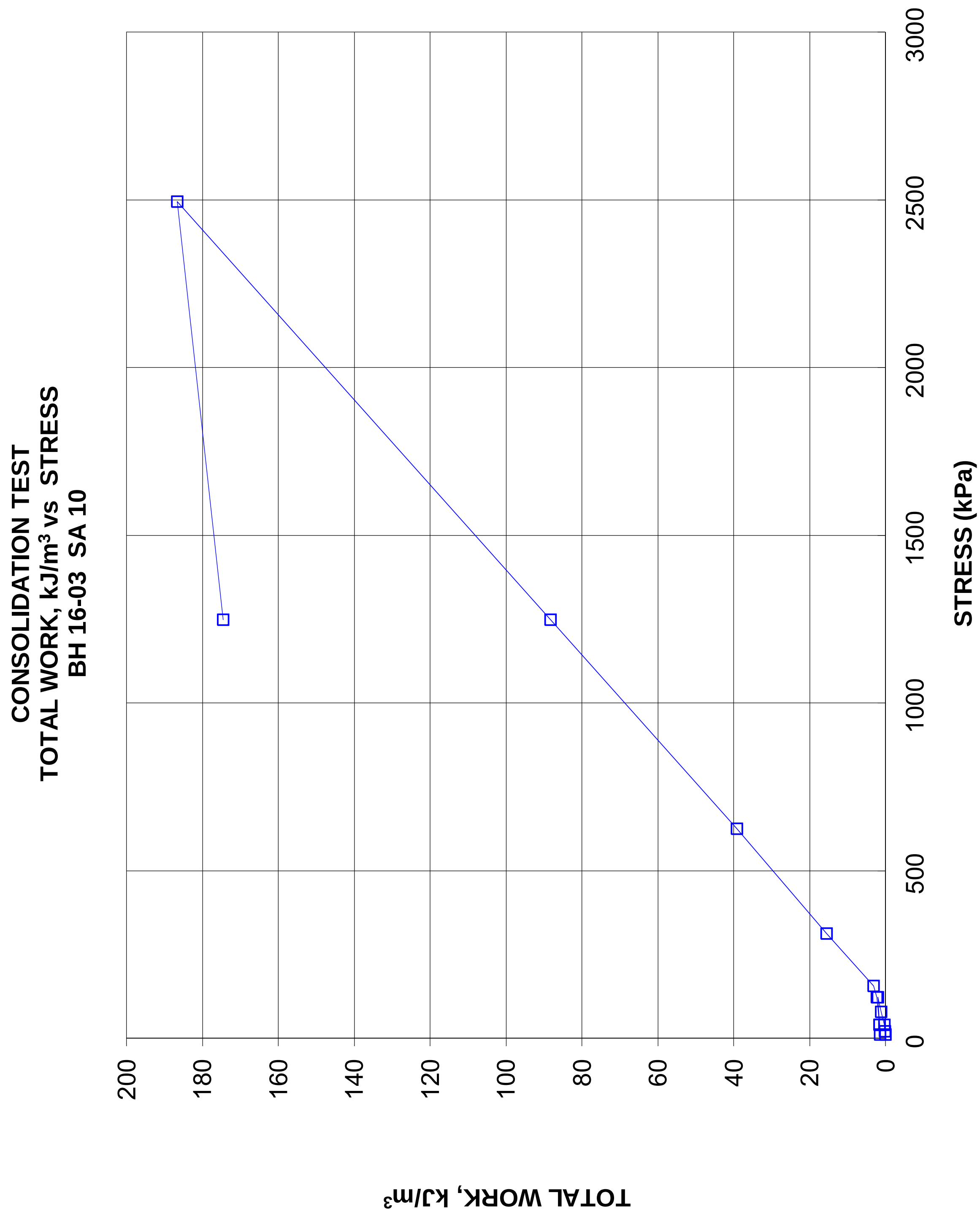


Project No. 1535723

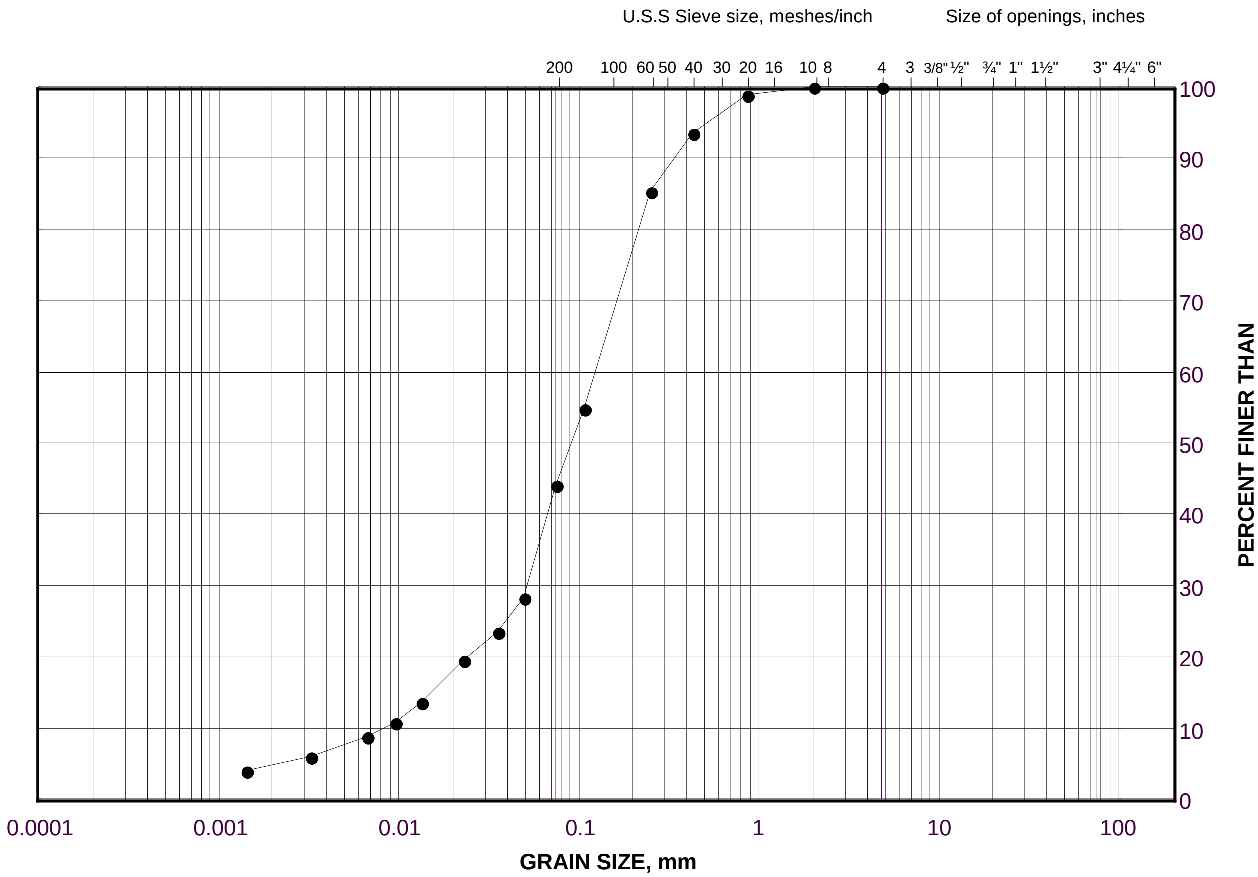
Prepared By: LH

Golder Associates

Checked By: MM


CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH 16-03 SA 10
STRESS (kPa)
0
500
1000
1500
2000
2500
3000
TOTAL WORK, kJ/m³
200
180
160
140
120
100
80
60
40
20
0

GRAIN SIZE DISTRIBUTION Silt and Sand (Lower)	FIGURE B13



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-02	11	216.1

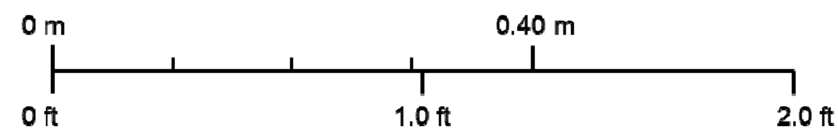
Project Number: 1547670

Checked By: TZ

Golder Associates

Date: 21-Feb-17

Box 1: 13.89 m to 18.56 m



PROJECT	Hwy 524 Commanda Creek Bridge Replacement (Site No. 44-029)					
TITLE	Bedrock Core Photograph Borehole 16-02					
Golder Associates	PROJECT No. 1547670			FILE No. ----		
	DESIGN	ACK	20170217	SCALE	NTS	REV.
	CADD	--		FIGURE B14		
	CHECK	TZ	20170221			
	REVIEW	CN	20170221			

Box 1: 12.03 m to 15.13 m

Scale

0 m — 0.40 m

0 ft — 1.0 ft — 2.0 ft

PROJECT	**Hwy 524 Commanda Creek Bridge Replacement (Site No. 44-029)**
TITLE	**Bedrock Core Photograph Borehole 16-03**

Golder Associates

PROJECT No. 1547670			FILE No. ----		
DESIGN	ACK	20170217	SCALE	NTS	REV.
CADD	--		**FIGURE B15**		
CHECK	TZ	20170221			
REVIEW	CN	20170221			

Box 1: 7.70 m to 12.17 m

Box 2: 12.17 m to 13.98 m

Scale

0 m 0.40 m

0 ft 1.0 ft 2.0 ft

PROJECT	Hwy 524 Commanda Creek Bridge Replacement (Site No. 44-029)					
TITLE	Bedrock Core Photographs Borehole 16-04					
Golder Associates	PROJECT No. 1547670			FILE No. ----		
	DESIGN	ACK	20170217	SCALE	NTS	REV.
	CADD	--		FIGURE B16		
	CHECK	TZ	20170221			
	REVIEW	CN	20170221			

Box 1: 6.54 m to 9.61 m

Scale

0 m — 0.40 m

0 ft — 1.0 ft — 2.0 ft

PROJECT
Hwy 524 Commanda Creek Bridge Replacement (Site No. 44-029)

TITLE
Bedrock Core Photographs
Borehole 16-05

Golder Associates

PROJECT No. 1547670			FILE No. ----		
DESIGN	ACK	20170217	SCALE	NTS	REV.
CADD	--		**FIGURE B17**		
CHECK	TZ	20170221			
REVIEW	CN	20170221			

APPENDIX C

Analytical Laboratory Test Results

Your Project #: 1547670
Your C.O.C. #: 76780

Attention:Alysha Kobylinski

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/02/06
Report #: R4350556
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B720656
Received: 2017/01/31, 15:45

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2017/02/03	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2017/02/03	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2017/02/02	2017/02/02	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2017/01/31	2017/02/03	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2017/02/03	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods. Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

Your Project #: 1547670
Your C.O.C. #: 76780

Attention:Alysha Kobylinski

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/02/06
Report #: R4350556
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B720656
Received: 2017/01/31, 15:45

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B720656
Report Date: 2017/02/06

Golder Associates Ltd
Client Project #: 1547670
Sampler Initials: AK

RESULTS OF ANALYSES OF SOIL

Maxxam ID		DVL567	DVL568	DVL568		
Sampling Date		2017/01/23	2017/01/24	2017/01/24		
COC Number		76780	76780	76780		
	UNITS	**BH-04 SA7**	**BH-03 SA4**	**BH-03 SA4 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters						
Resistivity	ohm-cm	3800	3300			4846388
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	100	170	180	20	4850517
Conductivity	umho/cm	266	306		2	4850667
Available (CaCl2) pH	pH	4.68	4.77			4848781
Soluble (20:1) Sulphate (SO4)	ug/g	60	<20	<20	20	4850519

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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TEST SUMMARY

Maxxam ID: DVL567
Sample ID: BH-04 SA7
Matrix: Soil

Collected: 2017/01/23
Shipped:
Received: 2017/01/31

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4850517	N/A	2017/02/03	Alina Dobreanu
Conductivity	AT	4850667	N/A	2017/02/03	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4848781	2017/02/02	2017/02/02	Neil Dassanayake
Resistivity of Soil		4846388	2017/02/03	2017/02/03	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4850519	N/A	2017/02/03	Alina Dobreanu

Maxxam ID: DVL568
Sample ID: BH-03 SA4
Matrix: Soil

Collected: 2017/01/24
Shipped:
Received: 2017/01/31

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4850517	N/A	2017/02/03	Alina Dobreanu
Conductivity	AT	4850667	N/A	2017/02/03	Neil Dassanayake
pH CaCl2 EXTRACT	AT	4848781	2017/02/02	2017/02/02	Neil Dassanayake
Resistivity of Soil		4846388	2017/02/03	2017/02/03	Cristina Carriere
Sulphate (20:1 Extract)	KONE/EC	4850519	N/A	2017/02/03	Alina Dobreanu

Maxxam ID: DVL568 Dup
Sample ID: BH-03 SA4
Matrix: Soil

Collected: 2017/01/24
Shipped:
Received: 2017/01/31

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4850517	N/A	2017/02/03	Alina Dobreanu
Sulphate (20:1 Extract)	KONE/EC	4850519	N/A	2017/02/03	Alina Dobreanu

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	4.0°C

Results relate only to the items tested.

Maxxam Job #: B720656
Report Date: 2017/02/06

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1547670
Sampler Initials: AK

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
4848781	Available (CaCl2) pH	2017/02/02			98	97 - 103			0.45	N/A
4850517	Soluble (20:1) Chloride (Cl)	2017/02/03	NC	70 - 130	107	70 - 130	<20	ug/g	2.8	35
4850519	Soluble (20:1) Sulphate (SO4)	2017/02/03	115	70 - 130	108	70 - 130	<20	ug/g	NC	35
4850667	Conductivity	2017/02/03			99	90 - 110	<2	umho/cm	0.68	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (one or both samples < 5x RDL).

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VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Cristina Carriere

Cristina Carriere, Scientific Services

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IMMEDIATE TEST

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
CAM FCD-01191/2

CHAIN OF CUSTODY RECORD 76780

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required
Company Name:	GOLDER ASSOCIATES	Company Name:		Quotation #:		☑ Regular TAT (5-7 days) Most analyses
Contact Name:	ALYSHA KOBYLINSKI	Contact Name:		P.O. #/ AFE#:		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS
Address:	6925 CENTURY AVE, #100 MISSISSAUGA, ONTARIO	Address:		Project #:	1547670	Rush TAT (Surcharges will be applied) ☐ 1 Day ☐ 2 Days ☐ 3-4 Days
				Site Location:		
Phone:	(905) 567-4444 Fax: (905) 567-6561	Phone:	Fax:	Site #:		Date Required:
Email:	Alysha_Kobylinski@golder.com	Email:		Sampled By:		Rush Confirmation #:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153: ☐ Table 1 ☐ Res/Park ☐ Med/ Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/ Other ☐ Table ____

FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ MISA ☐ Storm Sewer Bylaw ☐ PWQO Region ____ ☐ Other (Specify) ____ ☐ REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

LABORATORY USE ONLY: CUSTODY SEAL Y / N (N circled); Present; Intact; COOLER TEMPERATURES 3/4/5; COOLING MEDIA PRESENT: Y / N (Y circled)

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REG 153 METALS & INORGANICS	REG 153 ICPMS METALS	REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	PH	SULPHATE	CHLORIDE	RESISTIVITY	ELECTRICAL CONDUCTIVITY				HOLD- DO NOT ANALYZE	COMMENTS
1	BH-04 SA 7	2016/11/23	AM	SOIL	2								X	X	X	X	X					2 jars (120 mL x 2)
2	BH-03 SA 4	2016/11/24	AM	SOIL	2								X	X	X	X	X					2 jars (120 mL x 2)
3																						
4																						
5																						
6																						
7																						
8																						
9																						
10																						

(REFER TO BACK OF COC)

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)
Alysha Kobylinski ALYSHA KOBYLINSKI	2017/01/31	03:42 PM	[signature]	2017/01/31	15:45

31-Jan-17 15:45
Ema Gitej
B720656
MAF ENV-670

COC-1004 (10/14) - ENV. ENG.

White: Maxxam ~ Yellow: Client

APPENDIX D

Contract Specifications

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT – ITEM NO.

Non-Standard Special Provision

1.0 SCOPE

This Special Provision contains the requirements for the supply and installation of Temporary Survey Benchmarks (TBM) and Settlement Plates (SP) to monitor the settlement of the foundation soils during construction of the Highway 524 Commanda Creek Bridge south approach embankment.

The purpose of the SPs is to monitor settlements of the embankment base. The settlement readings shall help to establish the timing for removal of the surcharge. Settlement is measured by survey of the top of the rod with reference to stable, non-settling TBMs.

The timing for the removal of the surcharge shall be controlled by the instrumentation readings.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 905 Steel Reinforcement for Concrete

Ontario Provincial Standards Specifications, Material

OPSS.PROV 1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production

OPSS.PROV 1205 Clay Seal

OPSS 1301 Cementing Materials

OPSS 1801 Corrugated Steel Pipe (CSP) Products

Ontario Water Resources Act RRO 1990:

Regulation 903 Wells

3.0 DEFINITIONS

Contractor means the Contractor and his Geotechnical Consultant.

Geotechnical Engineering Consultant means a consultant with MTO classification of “Geotechnical (Structures and Embankments) - Medium Complexity”, to undertake the supply and installation of geotechnical instruments.

Temporary Survey Benchmark means a non-yielding, deep-seated survey reference point.

Monitoring Program means the monitoring readings conducted by others as part of the Contract Administration Assignment.

Settlement Plate means a plate installed at the defined level with a series of rods attached to a plate for the purposes of settlement monitoring.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

4.01.01 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to overhead and underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

4.01.02 Boreholes

The Contractor shall document subsurface conditions at the locations of instruments and prepare Record of Borehole sheets (borehole logs).

4.01.03 Marking and Labelling

The location of any above-ground monitoring fixtures shall be made clearly visible to nearby traffic before, during and after embankment construction. Markings shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for the duration of the surcharge period.

4.01.04 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Owner or Contract Administrator.

4.02 Submission Requirements

4.02.01 Notification

The Contract Administrator shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of instruments.

4.02.02 Installation Methods

The Contractor shall submit details of the proposed installation methods including locations, temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

5.0 MATERIALS

5.01 General

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless noted otherwise.

5.02 Temporary Benchmarks (TBM)

5.02.01 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 6.02.03.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.02.02 Sand

The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.02.03 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS.PROV 1205), 143 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.02.04 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 14 kg of bentonite (OPSS.PROV 1205), 49 litres of water and 40 kg of cement (Type GU – OPSS 1301).

5.02.05 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve for the full length of rod consisting of Schedule 40 – 50.8 mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

5.03 Settlement Plates (SP)

5.03.01 Plate

The Contractor shall supply a steel plate with a thickness of at least 6.35 mm. The plate shall be at least 0.5 m wide by 0.5 m long.

5.03.02 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 6.02.04.

The top end of the full length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

5.03.03 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 – 50.8 mm O.D. PVC pipe cut perpendicular to the axis of the pipe.

5.03.04 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the embankment.

The surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with medium to coarse sand.

6.0 CONSTRUCTION

6.01 Subsurface Conditions

The subsurface conditions at the site are described in Foundation Investigation Report as specified elsewhere in the Contract Documents.

6.02 Instrumentation Installation

6.02.01 Instrument Locations

The quantity and location of instruments are as shown in the Contract Documents and in Table 1A below.

Table 1A – Instrument Quantities and Locations

Monitoring Section			
Location	***Station***	***TBM***	***SP***
South Approach Embankment	9+956	1	0
	9+966	0	1
	9+978	0	1
	TOTAL	**1**	**2**

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

The locations of the monitoring instruments should be adjusted in the field such that they will not be damaged by the sub-excavation procedures for the new embankment, by highway maintenance equipment on the existing highway, or by earth moving equipment.

6.02.02 Installation Program

Table 1B presents a summary of the installation schedule requirements.

Table 1B – Installation Program

Location	Type	Start of Installation	Completion of Installation
South Approach Embankment	TBM	Before installation of settlement plates (SPs) and backfilling	Before installation of settlement plates (SPs) and backfilling
	SP	After sub-excavation and before backfilling	At completion of embankment construction, including placement of surcharge

6.02.03 Temporary Survey Benchmarks

6.02.03.01 General

The locations of the TBMs are given in Table 2A. The TBMs shall be installed prior to sub-excavation operations. The TBMs shall consist of a steel rod anchored to the bottom of a borehole.

The number and locations of TBMs shall be such that direct sighting is possible from all geotechnical instruments to at least one (1) TBM. The Contractor shall establish the geodetic elevation of each such TBM.

Table 2A – Approximate Temporary Benchmark Locations [1]

Location	Station	Offset from Proposed Centreline	Approximate Elevation of the Bottom of Rod Anchor [2] (m)	Estimated Final Length of Steel Rod including 1 m Stickup [2] (m)
South Approach Embankment	9+956	6 m Rt	213.8	14.5

NOTES:
1. Location to be agreed upon by Contractor and Contract Administrator prior to installation.
2. The rod anchor elevations shown are approximate and should be adjusted in the field so that the rod anchor is installed a minimum of 2 m into the bedrock.

6.02.03.02 Borehole Installation

The borehole shall be advanced to the rod anchor elevations provided in Table 2A using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction reducing sleeve and rod anchor grout. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

6.02.03.03 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

6.02.03.04 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole using the rod anchor grout mix to form a concrete/soil anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.5 m length of the borehole above the concrete/soil anchor to create a base for the end of the friction reducing sleeve to rest on.

6.02.03.05 Friction Reducing Sleeve

The friction reducing sleeve shall be over the entire length of the rod above the rod anchor and sand.

6.02.03.06 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

6.02.04 Settlement Plates

6.03.04.01 General

The locations of the SPs are given in Table 2B. As embankment construction proceeds the rods shall be extended above the top of the surcharge embankment. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

The SPs shall be placed on properly prepared subgrade following sub-excavation and replacement of existing fill/silt and sand deposit to Elevation 225.5 m. As embankment construction proceeds the settlement measuring rods shall be extended above the top of the surcharge embankment.

Table 2B – Settlement Plate Locations

Location	Station	Offset from Proposed Centreline	Approximate Founding Elevation of SP (m)	Estimated Thickness of Embankment Fill Above Properly Prepared Subgrade [1] (m)
South Approach Embankment	9+966	0 m	225.5	3.5
	9+978	0 m	225.5	3.5

NOTE: 1. Embankment fill thickness excludes surcharge.

The elevation, easting and northing of the centre of the base of the plate and top of the rod shall be surveyed after installation.

The total distance from the base of the plate to the top of the rod shall be measured to an accuracy of ± 2 mm, or better.

6.02.04.02 Plate

The SPs shall be installed horizontally on top of the backfill after sub-excavation and replacement of existing fill / organic deposits

6.02.04.03 Rod

The SP rod shall be fixed to the centre of the plate and perpendicular to the plate. The coupling of the rods shall be such that all sections have the same axis and that no separation or contraction will occur at the couplings.

6.02.04.04 Friction Reducing Sleeve

The friction reducing sleeve shall be over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the SP rod shall extend 25 mm above the top of the friction sleeve at all times.

6.02.04.05 Extension of Rod

The SP rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

6.02.04.06 Protective Surround

The CSP, Friction Reducing Sleeve and sand protective surround shall be extended concurrent with the rods. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

6.03 Coordination with Monitoring Program

6.03.01 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days the completion of installation of TBMs and SPs.

6.03.02 Reporting

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within three (3) days of completion of installation of each instrument.

6.03.02.01 Temporary Survey Benchmarks

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- TBM Northing and Easting in MTM NAD 83 coordinate system and latitude/longitude coordinates;

- Elevation of the rod anchor bottom rod anchor length and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions it encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

6.03.02.02 Settlement Plates

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP Northing and Easting in MTM NAD 83 coordinate system and latitude/longitude coordinates;
- Elevation of base of plate and top of rod in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

6.03.03 Monitoring

6.03.03.01 Temporary Survey Benchmarks

Monitoring of settlements with reference to the TBMs shall be done by others. Monitoring shall be conducted during the embankment construction. The Contractor shall provide installation information as specified above and provide access to the TBMs during the monitoring program.

6.03.03.02 Settlement Plates

Monitoring of the SPs shall be done by others. Monitoring shall be conducted during the embankment and surcharge construction. The Contractor shall provide installation information as specified above and provide access to the SPs for monitoring.

6.04 Decommissioning of Instruments

6.04.01 General

The Contractor shall decommission all the TBMs and SPs at the end of the monitoring program unless advised otherwise by the Contract Administrator. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources Act, R.R.O. 1990, Regulation 903, as applicable.

7.0 PAYMENT

7.0.1 Measurement for Payment

Measurement for Payment will be made on the basis of the number of units of survey TBMs and SPs installed.

7.02 Basis of Payment

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, monitoring equipment and material to do the work.

FOUNDATION MONITORING PROGRAM

1.0 GENERAL

The requirements for the monitoring of the following geotechnical instruments are required:

- Settlement Plates (SP)
- Temporary Survey Bench Marks (TBMs)

The instrumentation monitoring services include: data collection; data reduction and reporting; and adherence to criteria used to assess the embankment performance based on the monitoring data collected from the instruments installed by others.

1.0.1 Specialist Qualifications

The Foundation Engineering Consultant services required for this assignment have been categorized as "Geotechnical Specialty – Medium Complexity".

The Foundation Engineering Consultants that are registered in MTO's consultant registry acquisition system (RAQS) at the complexity rating in the required specialty that meets the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Consultant collecting, assessing and reporting the monitoring data shall not be the same Foundation Consultant retained by the Contractor for the supply and installation of embankment monitoring equipment.

The Foundation Engineer shall have a minimum of five (5) years of experience in the monitoring assessment of data and reporting for settlement plates and survey benchmarks data or alternatively demonstrate expertise through providing satisfactory monitoring services for the instrumentation specified for a minimum of two (2) projects in which the work was similar in scope to that in the contract.

1.0.2 Services, Deliverables and Records

The Foundation Engineering Consultant shall:

- Review the monitoring program and, if deemed necessary, submit in writing to the Contract Administrator recommendations for modifications to the Monitoring Program;
- Supply all materials and equipment that are required for the Monitoring Program;
- Calibrate and maintain monitoring equipment;
- Take instrument readings, reduce data, prepare reports;
- Provide transmittal of instrumentation readings and reports to the Contract Administrator;
- Interpret instrumentation readings as needed for the purpose of on-going construction;
- Notify the Contract Administrator of required modifications to the construction procedures accordingly, if necessary. Interpretation shall include making

correlations between instrumentation data and specific construction activities; and

- Notify the Contract Administrator if critical instrument readings (Review and Alert Levels), as specified herein, for any instrumentation are reached. Discuss as soon as possible (within 48 hours) with the Contract Administrator response action(s), and submit a plan of actions, to prevent the instrument readings from exceeding the critical levels.

Progress reports shall be submitted to the Contract Administrator, the MTO Contract Services Administrator and the MTO Foundations Engineer. Weekly reports shall be issued from the beginning of construction monitoring to the end of the one month period immediately after the top of the surcharge fill is reached. Thereafter, one report shall be submitted after each set of readings is taken. As a minimum, progress reports shall be submitted on a monthly basis. The progress reports shall discuss the Contractor's operations with respect to the installation of instrumentation, extent of embankment fill placed and a summary of the monitoring completed.

The Foundation Engineering Consultant shall maintain a Foundations Monitoring Diary. The diary shall document original conditions, work in progress, including any unusual or problem situations that arise, record of actions taken by the Contractor to rectify the situation, and restored conditions. The diary shall be supported by photographs of these conditions.

1.0.3 Submission of Foundation Monitoring Plan

The Foundation Engineering Consultant shall, in a brief narrative, discuss the applicable experience and qualifications of specialist staff, the role each will play in administration of the Contract, the authority to be assumed, and the reporting relationships with the construction administration staff.

The Consultant shall also complete the Foundation Monitoring Plan table in the format provided below.

Foundation Monitoring Plan		
Major Inspection Tasks	***Level of Inspection***	***Deliverable Record(s)***
List major inspection tasks associated with foundation monitoring.	State frequency/level of inspection.	List associated Deliverable Records for each task.

1.0.4 Purpose

The purpose of this Monitoring Program is to monitor settlement of the foundation soils during construction of the Highway 524 Commanda Creek Bridge south approach embankment.

The timing for the removal of the surcharge shall be controlled by the instrumentation readings.

1.0.5 Subsurface Conditions

The subsurface Conditions at the site are described in the following report:

- Foundation Investigation Report; Highway 524 Commanda Creek Bridge Replacement (Site No. 44-029), Ministry of Transportation, Ontario, Pringle township, Parry Sound District, Ontario, GWP 5260-13-00; Geocres No. 31L-205, August 31, 2017, by Golder Associates Ltd.

1.0.6 Equipment Operation

All monitoring equipment shall be maintained and rendered operational throughout the monitoring period.

Any equipment malfunction shall be investigated and attempts shall be made to remedy the malfunction. Notification of any equipment malfunction and equipment that cannot be repaired shall be made to the Contract Administrator. Documentation of the possible causes and suggested remedial measures shall be forwarded to the Contract Administrator.

1.0.7 Reading Schedule and Frequency

The Foundation Engineering Consultant shall save and archive raw data in electronic format.

Monitoring shall commence immediately after the installation of an instrument. Monitoring is to continue during a period from the start of embankment construction to the removal of surcharge. The actual length of the monitoring period is subject to the construction schedule and the results of monitoring amongst other factors.

The minimum monitoring frequencies along with the anticipated number of readings for the embankments in this contract are given in Table 1 and Table 2. The monitoring frequency is the same for each individual instrument indicated in the following tables. Instruments shall be read more or less frequently if judged to be required by the Contract Administrator.

It should be noted that the number of readings given in Table 1 are estimates and may vary depending on the actual construction schedule.

Table 1 – Minimum Monitoring Frequency for the Construction of Highway 524 Commanda Creek Bridge South Approach Embankment

Stage	Frequency	Anticipated Number of Readings Per Monitoring Section [1]
Baseline Readings [2]	Three readings on 3 consecutive days, no sooner than 7 days following installation	3
Immediately prior to start of embankment construction	Once	1
During embankment construction	Once every fill lift within 20 m of the monitoring section	5

Surcharge Period (anticipated duration: 4 months)	Weekly: First month Bi-weekly: Second month to end of surcharge	10

NOTE: 1. Due to uncertainty of the construction schedule, the anticipated number of readings per monitoring section is not equivalent to the number of site visits required to carry out the monitoring program described herein.
2. Baseline Readings: value of instrumentation readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of the ground and changes in piezometric head.

2.0 INSTRUMENTATION SPECIFIC REQUIREMENTS

2.0.1 Settlement Plates (SP)

Surveying

The elevations of Settlement Plates shall be surveyed to an accuracy of ± 2 mm or better and shall be reported to the nearest millimetre.

Surveying for settlement monitoring shall be conducted by a registered surveyor with appropriate equipment and experience. The surveyor shall be retained by the Foundation Engineering Consultant.

Reporting

A brief interpretation of the updated monitoring data shall be reported to the Contract Administrator within five (5) working days after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports.

As a minimum, the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the SPs:

- A plot of settlement of the base of the embankments (SPs) versus time on an arithmetic plot and semi-log plot;
- Fill height at the instruments versus time;
- Plan view, cross section and profile sketches showing the top of fill location while the SPs were being surveyed.

Review and Alert Levels

Typically, embankment failures result in an acceleration of settlements after placement of a lift of fill. If this condition is observed or the maximum settlement measured exceeds the Review Levels in Table 3, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and discuss response action(s). The Foundation Monitoring Consultant shall submit a plan of action(s) to prevent Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached.

If the measured total settlement exceeds the Alert Levels in Table 3, the Foundation Monitoring Consultant shall immediately inform the Contract Administrator and the

Contract Administrator shall inform the Foundation Design Engineer that instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on the affected embankment until all the following conditions are satisfied:

- The cause of the accelerated settlement has been identified and analyzed by the Foundation Design Engineer;
- Any corrective action deemed necessary by the Foundation Design Engineer has been implemented;
- The Contract Administrator deems it safe to proceed.

Table 3 – Review and Alert Levels for Instruments Monitoring Settlements

Instrument Type	Location	Station	Offset from Centreline	Settlement Response Levels (mm)	
				Review	Alert
Settlement Plates (SPs)	South Approach Embankment	9+966	0 m	75	100
		9+978	0 m	75	100

3.0 CONTROL MONITORING LEVELS

General

The monitoring program will provide input for the removal of the surcharge.

Stabilization of Settlements due to Primary Consolidation

Settlement data monitored at the SPs allow for an approximate assessment of the total settlement that will occur due to primary consolidation and the approximate time required for settlements due to primary consolidation to stabilize.

The anticipated total settlement that will occur and the required time for settlements due to primary consolidation to stabilize shall be assessed for each of the SPs using an appropriate method.

4.0 FINAL REPORT

At the completion of the monitoring program, a final monitoring report shall be issued to the Contract Administrator. The monitoring results shall be presented in tabular and graphical form as described above for the instruments. Interpretation of the monitoring data shall be included in the report.

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