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DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT

Embankment Widening - STA 14+300 TO STA 14+725

Township of Denison

Highway 17 and Municipal Road 55 West Junction Intersection
Improvements

Ministry of Transportation, Ontario

Agreement No. 5019-E-0026, GWP 5032-19-00

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PART A

FOUNDATION INVESTIGATION REPORT
EMBANKMENT WIDENING – STA 14+300 to 14+725
TOWNSHIP OF DENISON
HIGHWAY 17 AND MUNICIPAL ROAD 55 WEST JUNCTION INTERSECTION
IMPROVEMENTS
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5032-19-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the widening of Highway 17 east bound lane (EBL) alignment between STA 14+300 and 14+725 in the Township of Denison. The proposed work is part of the Highway 17 and Municipal Road 55 West Junction Intersection Improvements. The general location of the widening is shown on the Key Plan on Drawing 1.

The Terms of Reference and the Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal (RFP), dated May 13, 2020, and subsequent addenda. Golder's proposal for the associated foundation engineering services is contained in Section 7.7 of AECOM Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Project - Specific Supplementary QC Plan for foundation engineering services for this project, dated January 12, 2021. The base plan showing the existing horizontal alignment and a drawing showing the existing vertical alignment for Highway 17 were provided to Golder by AECOM in April 2021. The proposed widening options were provided to Golder by AECOM on May 4, 2021.

This report addresses the investigation carried out for the highway widening between stations 14+300 and 14+725 to accommodate the proposed acceleration lane along Highway 17. Separate reports address the foundation investigations for the culvert extensions.

Preliminary subsurface information for the widening section is available in the previous Foundation Investigation Report for High Fill Embankments Over Swamps, Highway 17 Four-Laning Extension, prepared by Golder under report number 11-1191-0007-01, dated June 6, 2015, GWP 156-98-00, Geocres No. 411-323 (Golder, 2015) and in the Preliminary Investigation for the Fairbanks Creek culvert prepared by MTO in January 1975 (MTO, 1975).

2.0 SITE DESCRIPTION

The overall project consists of improvements to the intersection of Highway 17 at the west junction of Sudbury Municipal Road 55. A new acceleration lane along the Highway 17 eastbound lane highway embankment is proposed that will require an embankment widening between about 3.5 m and 5 m. Based on the topographic survey provided by AECOM on March 8, 2021, the highway grade in this section of the EBL is between approximately Elevations 243.0 m and 244.0 m. The existing embankment slopes are varied in inclination between about 3 Horizontal and 1 Vertical (3H:1V) and 1H:1V (immediately adjacent to the Fairbanks Creek Culvert). At the time of the subsurface exploration field work, the embankment side slopes were generally snow covered; however, no signs of embankment / roadway slope instability were observed in the area of the proposed widening.

In general, the topography of this area consists of rolling terrain, numerous bedrock outcrops separated by low-lying swamps with areas of standing water and various vegetation types and organic soils. The land use in the general area includes residential developments with scattered rural farm use. The ground surface within the limits of the embankment widening varies between about Elevations 243 m and 241 m. The ground surface conditions along the footprint of the proposed embankment widening during the field investigation are shown on Photographs 1 to 4.

3.0 INVESTIGATION PROCEDURES

The investigation for the Highway 17 widening between STA 14+300 and 14+725 was carried out between February 2 and February 9, 2021, during which time eight sampled boreholes (designated 21-3 to 21-10) and two boreholes for vane testing (designated 21-05A and 21-09A) were advanced on the shoulder of the highway and at the toe of the existing embankment. The locations of the boreholes from the current investigation and relevant boreholes from the past investigations (designated H2-25 and 2) in the area of the proposed widening are shown on Drawing 1.

The field investigation was carried out using a track mounted CME-55, a truck mounted CME-55 drill rig, and portable equipment supplied and operated by Landcore Drilling (Landcore) of Sudbury, Ontario. The boreholes were advanced using 108 mm inner diameter hollow stem augers and NW casing (with portable equipment). Soil samples were obtained in the boreholes at 0.75 m and 1.5 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic or cathead hammer (for portable equipment) in general accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). Select samples of the cohesive soils were obtained using 76 mm O.D. thin-walled Shelby Tubes (ASTM D1587). In-situ vane shear tests were carried out in cohesive soils for determination of undrained shear strengths in accordance with Standard Test Method for Field Vane Shear Test in Saturated Fine Grained Soils (ASTM 2573), using an MTO standard 'N'-size vane.

The groundwater level inside the augers was observed during and upon completion of drilling operations and a standpipe piezometer was installed in Borehole 21-09 to monitor the groundwater level. The piezometer consisted of a 50 mm diameter polyvinyl chloride (PVC) pipe, with a slotted screen, sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite to create a seal above the screen. The piezometer installation details, and water level readings are shown on the borehole records contained in Appendix A. The boreholes and piezometer were backfilled in general accordance with Ontario Regulation 903 (as amended). The borehole (21-03) drilled through the roadway was capped at the roadway surface using cold patch asphalt.

Field work was supervised on a full-time basis by a member of Golder's technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes; and examined the soil samples. The soil samples were identified in the field, placed in labelled containers, and transported to Golder's geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions, and Atterberg limits tests were carried out on selected soil samples. In addition, incrementally loaded consolidation testing (ASTM D2435) was carried out on four samples of the clay deposit. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable.

The as-drilled borehole locations, in station and offset, were measured in reference to the centreline alignment staked on the shoulder and was subsequently converted into MTM NAD 83 coordinates in AutoCAD. The ground surface elevation at the borehole locations were surveyed by Golder, relative to the highway EBL centreline where benchmark elevations were provided by AECOM. The northing and easting, latitude/longitude coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the borehole records in Appendix A and summarized below.

Borehole	¹ Location (MTM NAD 83 Zone 17)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
21-03	5137564.6	278167.4	243.8	15.9
21-04	5137605.9	278203.5	243.6	25.0
21-05 / 05A ²	5137645.0	278244.8	242.2	9.9
21-06	5137676.9	278261.9	243.4	15.9
21-07	5137704.9	278285.0	243.3	15.9
21-08	5137709.2	278297.9	241.0	10.9
21-09 / 09A ²	5137748.0	278319.6	243.3	15.9
21-10	5137780.3	278354.1	241.2	10.7 ³

¹ Latitude and longitude co-ordinates referenced on borehole records.

² Boreholes 21-05A and 21-09A were advanced 2 m south and 5 m north of original borehole locations respectively and continuous vane testing was performed within the silty clay to clayey silt layers (refer to borehole record for 21-05 and 21-09).

³ DCPT driven from 8.8 m to 10.7 m refusal depth (Elev. 230.5 m).

Borehole 2 (MTO, 1975) and the Borehole H3-25 (Golder, 2015 report [drilled in 2012]) are also shown on Drawing 1. The borehole locations, Geodetic ground surface elevation, and drilled depth are shown on the borehole records in Appendix A and summarized as follows:

Borehole	Location (MTM NAD 83 Zone 17)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
H3-25	5137516.0	278139.2	242.2	23.5
2	5137573.4	278193.0	240.9	43.4*

*DCPT driven from 37.2 (Elev. 203.7 m) to 43.4 below ground surface (Elev. 197.5 m).

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in the NOEGTS¹ Mapping, the ground terrain in this section of Highway 17 is comprised of bedrock knobs, outcrops, and ridges within an undulating to rolling glaciolacustrine plain and alluvial plain containing areas of primarily silt with organic soil deposits. In the lower-lying glaciolacustrine plain and alluvial plain areas, the primary materials consist of wet silts, sands and clays, and the organic terrain deposit primarily consists of peat. The surface water drainage in the area varies from dry to wet, corresponding to areas of moderate to low relief.

Based on geological mapping by the Ministry of Natural Resources (Map 2542)², the site is underlain by rocks belonging to the Huronian Supergroup and Elliot Lake Group consisting of siltstone, wacke, and argillite. Areas of mafic and related intrusive rocks comprised of diabase sills, dykes, and related granophyre are also present in the vicinity of the site. Based on geological mapping by the Ontario Department of Mines (Map 2170)³ this site area is characterized by extensive faults including the Murray Fault, which has been identified to run parallel to the alignment of Highway 17.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation together with the results of the laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets in Appendix A. The results of the in-situ field tests (i.e., SPT 'N'-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheet and in Section 4.2 are uncorrected, unless otherwise noted. The Record of Borehole sheets from previous geotechnical investigations (Golder, 2015 and MTO, 1975) are also provided in Appendix A. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress, and the results of SPTs and in-situ testing. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change.

The subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented on the Record of Borehole sheets governs any interpretation of the site conditions. A summary description of the soil deposits and groundwater conditions encountered in the boreholes is provided below. It should be noted that the interpreted stratigraphy shown on Drawing 2 is a simplification of the subsurface conditions.

4.2.1 Asphalt

In Borehole 21-03, 100 mm of asphalt was encountered at ground surface at Elevation 243.8 m.

4.2.2 Topsoil

In Borehole 21-05, 50 mm of topsoil was encountered at ground surface at Elevation 242.2 m.

During the previous investigation, a 0.3 m thick layer of topsoil was encountered in Borehole H3-25 at ground surface. The surface of the topsoil was encountered at Elevation 242.2 m.

¹Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society Digital Map Reference Number 41ISW.

² Ministry of Natural Resources, 1991. Bedrock Geology of Ontario – West Central Sheet, Ontario Geological Survey - Map 2542

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

4.2.3 Fill Material

4.2.3.1 *Sandy Silt to Sand and Gravel Embankment Fill*

A 0.7 m to 4.4 m thick layer of sandy silt to sand and gravel embankment fill was encountered below the asphalt in Borehole 21-03 and from ground surface in Boreholes 21-04 and 21-06 to 21-09 between Elevations 240.7 and 243.7 m. Auger grinding was observed at depths ranging from 0.8 m to 2.9 m and 0.0 m to 2.1 m in Boreholes 21-03 and 21-09, respectively, suggesting potential obstructions within the embankment fill (e.g., cobbles and/or boulders).

The SPT 'N'-values measured within the cohesionless embankment fill range from 2 blows to 105 blows per 0.3 m of penetration, indicating a very loose to very dense state of compactness. One SPT did not penetrate the full test length in Borehole 21-03, encountering an obstruction after penetrating 0.1 m.

Grain size distribution testing was carried out on three samples of the cohesionless embankment fill and the results are presented on Figure B-1 in Appendix B. The natural moisture content measured on samples of the embankment fill ranged between 5% and 16%.

During the previous investigation a 2.3 m thick deposit of sand, some gravel, some silt embankment fill was encountered in Borehole H3-25 at Elevation 241.9 m. One SPT 'N'-value measured within the sand embankment fill was 12 blows per 0.3 m of penetration, indicating a compact state of compactness.

4.2.3.2 *Silty Clay Fill*

A 1.6 m thick layer of silty clay, trace organics fill was encountered below the topsoil in Borehole 21-05 at Elevation 242.1 m.

Two SPT 'N'-values measured within the cohesive embankment fill were 5 blows and 8 blows per 0.3 m of penetration, suggesting a firm to stiff consistency. Two field vane tests performed in the silty clay fill measured shear strengths between 70 kPa and 80 kPa, consistent with the stiff consistency measured from the SPT measurements.

4.2.4 Peat

A 50 mm thick layer of peat was encountered in Borehole 21-10 from ground surface at Elevation 240.7 m.

During the previous investigation, a 0.9 m thick layer of peat (muskeg) was encountered from ground surface in Borehole 2 at Elevation 240.9 m.

4.2.5 Organic Silt

A 1.3 m and 0.9 m thick deposit of organic silt was encountered below the fill in Boreholes 21-04 and 21-05 at Elevations 239.9 m and 240.5 m, respectively.

The SPT 'N'-values measured within this deposit range from 1 blow to 4 blows per 0.3 m of penetration indicating a very loose to loose state of compactness.

Organic content tests were carried out on two samples and the results were 6.7% and 8.8 %. The natural moisture content measured on two samples of the organic silt were 54% and 61%.

An Atterberg limits test carried out on one sample of the organic silt yielded a non-plastic result.

4.2.6 Upper Gravelly Sand

A 1.1 m thick deposit of gravelly sand was encountered below the organic silt in Boreholes 21-05 at Elevation 239.6 m.

Two SPT 'N'-values measured within this deposit were 11 blows and 2 blows per 0.3 m of penetration indicating the layer is very loose to compact.

Grain size distribution testing was carried out on one sample of the gravelly sand and the results are shown on Figure B-2 in Appendix B. The natural moisture content measured on one sample of the gravelly sand was 17%.

4.2.7 Clayey Silt to Silty Clay

A 2.8 m to 12.8 m thick deposit of clayey silt to silty clay was encountered below the embankment fill in Boreholes 21-03 and 21-06 to 21-09, below the organic silt in Borehole 21-04, below the upper gravelly sand in Borehole 21-05 and below the peat in Borehole 21-10. The top of the deposit was encountered between Elevations 240.6 m and 238.5 m. In Boreholes 21-03, 21-05 and 21-08 the deposit was not fully penetrated with the boreholes being terminated in the deposit after exploring for between 5.9 m and 11.4 m. The deposit was generally observed to be varved (alternating silty clay and clayey silt layers) below about Elevation 235 m in Boreholes 21-03, 21-07 and 21-08.

The SPT 'N'-values measured within the clayey silt to silty clay ranged from 0 blows (i.e., weight of rods) to 8 blows per 0.3 m of penetration, suggesting a very soft to firm consistency. In-situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 19 kPa to greater than 100 kPa, with calculated sensitivity between about 1 and 8. The field vane test results indicate that the deposit has a soft to very stiff consistency.

An organic content test was carried out on one sample of the deposit near the interface with the overlying fill deposit in Borehole 21-06 and the result was 1.8%.

Grain size distribution testing was carried out on one sample of the clayey silt layer and the result is presented on Figure B-3 in Appendix B

Atterberg limits tests were carried out on twenty samples of the deposit, which indicate liquid limits between about 25% and 48%, plastic limits between about 15% and 23% and plasticity indices between about 8% and 26%. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B-4 in Appendix B and indicate that the material is classified as clayey silt to silty clay of low to intermediate plasticity.

Four consolidation (oedometer) tests were carried out on selected specimens of the silty clay from samples obtained using Shelby tubes. Two of the tests were carried out in the horizontally trimmed orientation to evaluate the deformation parameters of the cohesive deposit, whereas the remaining two tests were completed in the vertically trimmed orientation to allow for the evaluation of horizontal drainage parameters. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot for the horizontally trimmed tests. The bulk unit weight measured from the specimens ranged from 17.0 kN/m³ to 17.7 kN/m³, with a measured specific gravity between 2.74 and 2.77. The detailed results of the oedometer test are shown on Figures B-5 to B-8 in Appendix B, and the test results are summarized below.

Borehole / Sample No.	Sample Elevation (m)	w _n (%)	γ (kN/m ³)	σ _{vo} ' (kPa)	σ _p ' (kPa)	OCR	e _o	C _c	C _r	C _v (cm ² /s)
21-08 / 8A	232.3	53.5	17.2	70	150	2.1	1.4	0.66	0.046	0.006
21-08 / 8B	232.0	54.4	17.1	70	N/A	N/A	N/A	N/A	N/A	N/A
21-04 / 14A	228.2	49.5	17.0	154	238	1.5	1.4	0.74	0.041	0.0017
21-04 / 14B	228.1	49.5	17.7	154	N/A	N/A	N/A	N/A	N/A	N/A

Notes: Parameters presented calculated within the operative stress range for this project.

Where: w_n Natural Moisture content (%)
 γ Unit weight (kN/m³)
 σ_{vo}' Effective overburden pressure (kPa)
 σ_p' Preconsolidation pressure (kPa)
 OCR Overconsolidation Ratio
 e_o Initial void ratio
 C_c Compression index
 C_r Recompression index
 C_v Coefficient of consolidation in the normally consolidated range (cm²/s)

During the previous investigations a 16.8 m and 18.3 m thick clayey silt to silty clay to clay deposit was encountered in Boreholes H3-25 and 2 at Elevations 239.6 m and 240.0 m, respectively.

The SPT 'N'-values measured within this deposit are between 0 blows (weight of hammer) and 7 blows per 0.3 m of penetration. In-situ field vane test values measured within this deposit generally range from about 30 kPa to 80 kPa indicating a soft to stiff consistency.

Seven Atterberg limits tests and four grain size distributions were carried out on samples of the silty clay to clay deposit and the results are shown on the Record of Borehole sheets in Appendix A. In addition, three consolidation tests were completed on samples of this deposit. The natural water content measured on samples of this deposit were between about 32% and 67%.

4.2.8 Silt to Sand

A 0.6 m to 8.7 m thick silt to sand deposit was encountered below the clayey silt to silty clay deposit in Boreholes 21-04, 21-06, 21-07, 21-09, and 21-10 between Elevations 237.8 and 225.8 m. In Boreholes 21-04, 21-06 and 21-07, the deposit was not fully penetrated with the boreholes being terminated in the deposit after exploring for between 0.6 m and 7.2 m.

The SPT 'N'-values measured within the deposit range from 0 blows (i.e., weight of rod) to 28 blows per 0.3 m of penetration, indicating a very loose to compact state of compaction.

Grain size distribution testing was carried out on eight samples of this deposit and the results are presented on Figure B-9 in Appendix B. The natural moisture content measured on samples of the deposit were between 11% and 33%.

Atterberg limits tests were carried out on six samples of the deposit, which yielded non-plastic results.

During the previous investigations, a 11.3 m thick layer of silt to silty sand was encountered in Borehole 2 at Elevation 221.7 m underlying the cohesive deposit. In Borehole 23-5, the top of the silt deposit was encountered at Elevation 222.8 m with the borehole being terminated after exploring the deposit for 4.1 m.

The SPT 'N'-values measured within this deposit are between 1 blow and 19 blows per 0.3 m of penetration, indicating a very loose to compact state of compactness.

Grain size distribution tests were carried out on two samples of the deposit and the results are presented on the Record of Borehole sheets in Appendix A. The natural moisture content measured on two samples of the deposit were about 10% and 22%.

4.2.9 Sand to Gravelly Sand

A deposit of gravelly sand to sand was encountered below the silt to sand in Boreholes 21-09 and 21-10 at Elevations 230.0 m and 234.6 m, respectively. Boreholes 21-09 and 21-10 were terminated within the deposit after exploring for 2.6 m and 2.2 m, respectively. Refusal to practical casing advancement was encountered in Borehole 21-10 at a depth of 8.2 m; however, a dynamic cone penetration test (DCPT) was advanced at the bottom of the borehole from 8.8 m to 10.7 m depth where refusal to further cone penetration (hammer bouncing) was observed.

The SPT 'N'-values measured within this deposit are between 6 blows and 72 blows per 0.3 m of penetration, indicating a loose to very dense state of compactness.

The natural moisture content measured on two samples of the deposit was 11% and 16%.

During the previous investigation, a deposit of sand and gravel was encountered below the silt to silty sand deposit in Borehole 2. The surface of the deposit was encountered at Elevation 210.4 m and the borehole was terminated within this deposit exploring it for a thickness of 6.7 m. A DCPT was advanced from the bottom of Borehole 2 until effective refusal (100 blows / 0.3 m of penetration) was achieved at a depth of 43.4 m below ground surface (Elevation 197.5 m).

The SPT 'N'-values measured in Borehole 2 within this deposit range between 28 blows and 47 blows per 0.3 m of penetration, indicating a compact to dense state of compactness.

A grain size distribution test was carried out on one sample of the deposit and the result is shown on the previous Record of Borehole sheet in Appendix A.

The natural moisture content measured on one sample of the deposit was about 5%.

4.3 Groundwater Conditions

The unstabilized groundwater levels relative to ground surface measured inside the augers upon completion of drilling are shown on the borehole records in Appendix A. A stabilized groundwater level was measured in the standpipe piezometer installed in Borehole 21-09 that was subsequently decommissioned on the last day of the field investigation. A summary of the measured groundwater levels at the borehole locations is provided below.

Borehole No.	Depth ¹ to Groundwater Level (m)	Approximate Groundwater Elevation (m)	Notes
21-03	3.2	240.6	Inside augers (unstabilized)
21-04	9.2	234.4	Inside augers (unstabilized)
21-05	9.6	232.6	Inside augers (unstabilized)

Borehole No.	Depth ¹ to Groundwater Level (m)	Approximate Groundwater Elevation (m)	Notes
21-06	12.4	231.0	Inside augers (unstabilized)
21-07	6.8	236.5	Inside augers (unstabilized)
21-08	2.8	238.2	Inside casing (unstabilized)
21-09	3.3 0.4	240.0 242.9	Inside augers (unstabilized) Piezometer, February 9, 2021 (stabilized)
21-10	-(1.9)	242.6	Inside casing (unstabilized) Potential artesian conditions in granular layer below cohesive deposit.

¹ Depth relative to ground surface

The groundwater level measured upon completion of drilling in previous Borehole H3-25 was at Elevation 240.6 m (1.6 m below ground surface). The groundwater was encountered at about ground surface in the previous Borehole 2 corresponding to Elevation 240.9 m.

The water level near the outlet of the Fairbank Creek culvert was measured by others to be at Elevation 240.9 m on January 7, 2021. Groundwater and creek water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Tibor Berecz, EIT, under the overall direction of Mr. Matthew Thibeault, P.Eng. This report was prepared by Mr. Tibor Berecz, EIT, and the technical aspects were reviewed by Mr. Matthew Thibeault, P.Eng., a geotechnical engineer. Mr. Kevin Bentley, P.Eng., an MTO Foundations Designated Contact and Associate with Golder, conducted an independent quality control review of this report.

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PART B

FOUNDATION DESIGN REPORT
EMBANKMENT WIDENING – STA 14+300 to 14+725
TOWNSHIP OF DENISON
HIGHWAY 17 AND MUNICIPAL ROAD 55 WEST JUNCTION INTERSECTION
IMPROVEMENTS
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5032-19-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides Foundation design recommendations for the Highway 17 embankment widening between STA 14+300 and 14+725. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface exploration. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible foundation alternatives and carry out the design of the structure foundations, as required. This Foundation Investigation and Design Report, including the discussion and recommendations are intended for the use of the MTO and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A) of the 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 the 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.

6.1 General

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Limited (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO), to provide recommendations on foundation aspects for the detail design of the proposed embankment widening along Highway 17, in the Township of Denison, to accommodate an eastbound acceleration lane ramp from Municipal Road 55. The proposed embankment widening is anticipated to be about 3.5 m wide, up to 4 m high to match the existing Highway 17 grade and will extend from the intersection of Highway 17 and Municipal Road 55. We understand that a 5 m widening may also be considered as the design progresses.

This report presents the results of embankment stability/settlement analyses and provides recommendations for stable embankment geometry, embankment fill materials and implementation of mitigation alternatives that may be required as a means to improve stability and reduce post construction settlements, as necessary. The report also addresses potential construction concerns and geotechnical challenges associated with embankment construction, sub excavating soft/organic materials, and placement of fill materials.

6.1.1 Consequence and Site Understanding Classification

Highway 17 carries a relatively large volume of traffic and has the potential to impact alternative transportation corridors; therefore, a “typical consequence level” is considered appropriate for the foundation design at this site, as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2019) and its Commentary. Further, given the scope of work of the foundation field investigation and laboratory testing program, as presented in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding” has been utilized. 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.

6.2 Embankment Widening Over Soft Ground

Based on the proposed final geometry configuration provided by AECOM, widening of the east side of the embankment to about 3.5 m will be required between about Station 14+300 to 14+725.

Section 6.2.1 of this report discusses potential fill options for the widening. Sections 6.2.2 and 6.2.3 summarize the methods used to analyze the stability and settlement for critical sections along the proposed widening and the results of the analysis for an unmitigated construction approach. Section 6.3 presents potential alternatives for mitigating post-construction embankment settlement, with the results of analysis for select mitigation

approaches presented in Section 6.4. Section 6.5 presents a discussion on monitoring instrumentation and Section 6.6 outlines general aspects of subgrade preparation and embankment construction.

The stability and settlement analyses assume that the peat and near surface organic soils (i.e., peat, muskeg and/or topsoil) will be removed prior to constructing the new embankments. For details on the thickness of organic deposits, refer to Section 4. The piezometric / groundwater conditions assumed in the analyses are based on the groundwater levels noted in the open boreholes, depths where wet samples were first encountered in the boreholes, and from the piezometer readings. In general, the stabilized groundwater level was measured at about the level of the natural ground surface adjacent to the existing embankment.

6.2.1 Embankment Fill Types

Different fill materials (i.e., rock fill and granular fill) used for embankment construction provide relative advantages and disadvantages in terms of availability, weight (i.e., driving force and applied load to the founding deposit), construction cost and time, ease of construction and post construction performance. Further, the availability of select fill materials in the vicinity of the project may impact the economical considerations for fill selection. For this project, it is understood that granular material will be the preferred embankment fill material as no rock blasting for road cuts are required elsewhere on the project that would produce easily available rock fill for placement. In this regard, the stability and settlement analyses discussed in Section 6.2 have been carried out on the basis that the highway embankment widening will be constructed of granular fill.

Granular Fill

The main advantages of using granular fill for embankment construction are the ease of construction; and negligible post construction settlement within the embankment fill itself. However, this fill option requires a larger volume of material and potentially wider right of way because the side slopes of granular fill embankments (2H:1V) are flatter than those of rock fill. For this project, acceptable granular fill is considered to be well graded, locally available and/or imported granular material meeting OPSS.PROV 1010 Select Subgrade Material or Granular A or B.

For granular fill embankments, 2 m wide berms should be incorporated into the side slope profiles for uninterrupted slopes greater than 8 m high. Given that the embankment under consideration is less than 8 m high, 2 m wide mid slope berms are not required.

If wick drains are being considered as a potential settlement mitigation option (see later sections in this report), granular fill should also be used as backfill below the ground surface after removal of the organic deposits and above the cohesive deposit to allow for ease of installation through the backfill (i.e., the wick drains would be installed from the top of the backfill drainage blanket).

Rock Fill

The main advantages of constructing embankments using rock fill are the ability to achieve steeper side slopes of 1.25 Horizontal to 1 Vertical (1.25H:1V), which is required in areas with limited right of way; reducing the overall quantity of fill material required for the project; and for placement of material in sub excavated areas under water. The disadvantage of using rock fill for the construction of embankments is that some post construction settlement of the embankment fill itself will occur and future excavations within the embankment may be more difficult. Settlement of the rock fill is discussed further in the following sections of this report.

In accordance with MTO Northern Region Pavement Practices and Guidelines (1997) as amended by MTO Memorandum "Use of Mid Slope Berms for Rockfill Embankments" dated February 8, 2005, 2 m wide berms should be incorporated into the rock fill embankment side slope profile for uninterrupted slopes greater than

10 m high. Given that the embankment under consideration is less than 10 m high, 2 m wide mid slope berms are not required.

6.2.2 Embankment Stability

The following sections outline the methodology used to interpret geotechnical parameters for the foundation soils and evaluate embankment stability for the critical section along the embankment widening over soft ground.

6.2.2.1 Methodology

Stability analyses were carried out at the southern slope of the critical section (i.e., at about Station 14+380), which approximately corresponds to the greatest embankment height and maximum thickness of soft founding soils. The stability analysis assumes that the organic deposits have been removed and replaced in accordance with OPSD 203.020 (Embankments Over Swamp) prior to construction of the new embankment.

The limit equilibrium analyses were performed using the commercially available program GeoStudio 2021 (Version 11.0.1.21429), produced by GEOSLOPE International Ltd., by employing the Morgenstern-Price method to assess the short-term (undrained) conditions and long-term (drained) conditions. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure.

For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, Φ_{gu} (i.e., $\text{FoS} = 1 / [\Psi * \Phi_{gu}]$). A minimum factored FoS of 1.33 in the short-term condition was required, based on a typical consequence level and a typical degree of site understanding, as per the CHBDC (2019). Similarly, a minimum factored FoS of 1.54 in the long-term condition was required.

For the analysis, it is assumed that the new fill is free-draining and that the ground water level is located at the bottom of the fill/top of the native subgrade adjacent to the existing embankment. The stability analysis was carried out to check if the proposed embankment widening design meets the required minimum FoS at the critical section in both short-term and long-term conditions.

6.2.2.2 Parameter Selection

A summary of the foundation engineering parameters employed in the stability models for the cohesive deposit encountered (i.e., clayey silt to silty clay) is presented on Figure 1. The granular fill was assumed to have an effective friction angle of 35° with a compacted unit weight of 21 kN/m^3 .

The founding soils at the location of the critical section include a combination of organic soils, cohesive deposits (clayey silt to silty clay) and granular soils. For granular soils, effective stress parameters were employed in the analyses assuming drained conditions for both short-term and long-term analyses. For cohesive deposits, total stress or effective stress parameters were employed in the analyses, as appropriate.

The effective stress parameters (effective friction angle and effective cohesion) for the organic and granular soils were estimated with engineering judgement based on experience in similar soil conditions.

The total stress parameters (i.e., mobilized undrained shear strength) for the cohesive soils were assessed based on the results of in-situ field vane shear tests, inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. For the consolidation tests performed in the clayey soils, the following correlation proposed by Mesri (1975) was employed to estimate the mobilized undrained shear strength:

$$S_{u(FV-uncorrected)} = 0.22\sigma_p'$$

where: $S_{u(FV-uncorrected)}$ = average mobilized undrained shear strength (kPa)
 σ_p' = preconsolidation pressure (kPa)

With respect to the overconsolidated cohesive crust encountered near ground surface, the design line for the mobilized undrained shear strength presented on Figure 1 was adjusted to account for potential fissuring after Tavenas and Leroueil (1980).

The Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils (ASTM D2573) states that the peak undrained shear strength from the field vane test needs to be multiplied by a vane correction factor (μ) to give a mobilized field value of undrained strength for geotechnical analysis. It also includes the following expression:

$$\tau_{mobilized} = \mu_v(S_u)_{FV}$$

where: $\tau_{mobilized}$ = the mobilized shear strength ($S_{u(mob)}$) for geotechnical analysis

μ_v = an empirical correction factor that has been related to plasticity index (PI) and/or liquid limit (w_L) and/or other parameters based on back calculation from failure case history records of full-scale projects.

For a horizontally layered varved clay stratum, a maximum correction factor (μ_{max}) can be applied over a range of failure surface angles relatively close to the horizontal (e.g., approximately $i = 0^\circ \pm 5^\circ$ to $\pm 15^\circ$), while the minimum correction factor (i.e., $\mu=1$ or no correction) is applied over a range of failure surface angles oblique to the horizontal (e.g., $-45^\circ < i < +45^\circ$). Ladd and Foott (1977) suggest that the near horizontal failure surface mobilizing the minimum shear strength should (i.e., along-shear) be defined by $i = 0^\circ \pm 10^\circ$, while the portions of the slip surface oblique to the horizontal mobilizing the maximum shear strength (i.e., cross-shear) be defined by $i = 30^\circ$ to 60° .

For the stability analyses presented herein, a simplified μ_{avg} correction factor was applied to the undrained shear strength design line, where applicable, to account for the affect of varves on the lower shear strength mobilized 'along-shear' in the field. Figure 2 presents data available from literature for both non-varved and varved clay sites and a proposed correlation based on plasticity index to select a μ_{avg} . For this site, a correction was obtained using an upper bound $PI_{(max)}$ of 25 with the correlation proposed by Golder on Figure 2. A $\mu_{(avg)}$ correction factor of 0.85 was used for both the stability and settlement analysis.

The effective parameters for the cohesive soils were assessed based on a combination of engineering judgement and empirical correlations. In particular, the effective friction angle was based on correlations to Atterberg limit testing (i.e. [Mitchell, 1993], [Ladd, 1977] and [Kulhawy and Mayne, 1990]). The effective cohesion was conservatively assumed to be negligible and the same $\mu_{(avg)}$ was applied in deposits that were observed to be varved.

6.2.2.3 Results of Unmitigated Stability Analysis

The results of the stability analyses carried out at STA 14+380 are presented on Figures 3 to 5 for the short-term total stress analysis, short-term effective stress analysis, and long-term effective stress analysis. We understand that the proposed design includes a 3.5 m embankment widening; however, based on discussions with AECOM, we further understand that a 5 m embankment widening may be considered as the design progresses.

Therefore, the stability analyses were carried out for a 5 m widening, which is considered to provide conservative results for the currently proposed 3.5 m widening. Based on the results of the stability analysis, a Factor of Safety greater than 1.54 is calculated assuming a widening of 5 m with a side slope of 2H:1V; thus,

satisfying the global stability requirements outlined in the CHBDC. We further understand that 3H:1V embankment slopes might be considered for the widening, which would further increase the global stability of the proposed embankment.

6.2.3 Embankment Settlement

The following sections outline the methods used to carry out the analyses, interpretation of the geotechnical parameters and results of analysis associated with settlement. Section 6.3 provides recommendations regarding possible design and construction alternatives to mitigate post construction settlement.

6.2.3.1 Methodology

The settlement performance criteria for embankment widening are outlined in Section 1.3 of MTO Foundation Guideline, "Embankment Settlement Criteria for Design", dated July 2010. Total settlements and differential settlements are to be less than 50 mm and 200:1, respectively, over a 20-year period following completion of construction for a "freeway".

Where widened embankments approach structural elements, more stringent settlement criterion will apply in accordance with Section 1.2 of the MTO Foundation Guideline. The discussions herein are focused on the proposed embankment widening along the entirety of the embankment widening, specific details related to the anticipated settlement performance at the CSP and Fairbanks Creek Culvert extensions are provided in separate Foundation Investigation and Design Reports for this project.

Settlement analyses were carried out along the shoulder of the proposed embankment widening. To estimate the magnitude of the expected settlements, the anticipated increase in loading due to the embankment widening was modeled using the commercially available program Settle3 (Version 5.010) produced by Rocscience Inc. The settlement analysis assumes that the organic deposits and near surface cohesive soils containing excessive organics have been removed and replaced in accordance with OPSD 203.020 (Embankments Over Swamp) prior to construction of the new embankments.

The sources of settlement are considered to include:

- immediate settlement of the native granular soils;
- primary time dependent consolidation of the cohesive deposits (using Terzaghi's one dimensional consolidation theory); and,
- secondary time dependent (creep) compression of the cohesive deposits (long term)

The thickness of the compressible foundation soils and the height/width of the embankment widening will vary along the proposed ramp length and as such the settlements along the length of a given alignment will similarly vary.

As noted above, in addition to primary consolidation being evaluated using Terzaghi's one dimensional consolidation theory within the cohesive deposits (i.e., clayey silt to clay), secondary compression is also assessed. Secondary compression (i.e. creep settlement) occurs over a long period of time, after dissipation of excess pore pressure under a constant stress. The following relationship has been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location:

$$S_c = HC_{\alpha\epsilon} \log \left(\frac{t}{t_{EOP}} \right)$$

where: S_c = secondary compression (creep) settlement (mm)
 $C_{\alpha\epsilon}$ = modified secondary compression index
 H = initial thickness of compressible clay deposit (mm)
 t = post construction period of interest (20 years)
 t_{EOP} = time to reach end of primary consolidation (years)

Based on experience from other sites in Northeastern Region, the secondary compression was applied to the settlement models after a degree of primary consolidation (U) of 90% was achieved.

6.2.3.2 Parameter Selection

The simplified stratigraphy together with the associated deformation and time rate consolidation parameters employed for the different native soil types were evaluated based on in-situ field testing, laboratory testing and engineering judgement. A summary of the foundation engineering parameters employed in the settlement models for the cohesive deposits is presented on Figure 1.

The immediate compression of the cohesionless deposits (i.e., silt, sand or gravel) were modelled by estimating an elastic modulus of deformation from engineering judgement based on similar soils in Northeastern Ontario.

The primary consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation tests and, where appropriate, in-situ field vane tests to estimate the deformation parameters. In addition, for the clayey soils the results of the laboratory index tests were also employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Koppula (1981) and Azzouz et al. (1976), as well as an empirical correlation developed from research performed by Golder for the MTO in Northeastern Region (Geocres No. 32D-35). The literature correlations were compared to the results of the consolidation testing for this site (by elevation and void ratio) to select an appropriate site-specific correlation for use in the selection of design lines.

For clayey soils, the following correlation relating in-situ mobilized undrained shear strength to preconsolidation pressure (Mesri, 1975) was employed:

$$\sigma_p' = \tau_{mobilized} / 0.22$$

where: σ_p' = preconsolidation pressure (kPa)

$$\tau_{mobilized} = \mu_v (s_u)_{FV}$$

where: $\tau_{mobilized}$ = the mobilized shear strength ($s_{u(mob)}$) for geotechnical analysis

μ_v = an empirical correction factor that has been related to plasticity index (PI) and/or liquid limit (w_L) and/or other parameters based on back calculation from failure case history records of full-scale projects.

The coefficient of consolidation, c_v (cm²/s), required in the time rate settlement analysis, was established for the site using the combined results of the laboratory consolidation tests and the estimated c_v values based on the Unified Facilities Criteria (U.S. Navy, NAVFAC 1986) correlation with liquid limit for normally consolidated and overconsolidated soils.

A value of c_h was also assessed from the results of a laboratory consolidation test performed on two specimens trimmed in the vertically trimmed orientation (VTO) from samples taken of the cohesive deposit. The results of

the conventional horizontally trimmed orientation (HTO) and the vertically trimmed orientation (VTO) laboratory consolidation tests at similar elevations/locations were used to assess the ratio between the horizontal and vertical coefficient of consolidation (c_h/c_v) employed in the preliminary analyses for wick drains if considered as an appropriate settlement mitigation option as discussed in Section 6.3. Specifically, the results of the consolidation test comparison indicated a $c_h/c_v = 2$, which is within the typical range provided in literature of 2 to 5.

The secondary compression index was evaluated using a method proposed by Mesri et al. (1994), which indicates that for every soil, a constant value of the ratio of C_{α}/C_c holds at all combinations of consolidation pressure and time. Specifically, for this site a ratio of $C_{\alpha(e)}/C_c = 0.04 \pm 0.01$ for inorganic soils (Mesri et al., 1994).

6.2.3.3 Settlement of Embankment Fill

We understand that granular fill is considered the preferred embankment fill material for the proposed embankment widening at this site; therefore, assuming the granular fill consists of OPSS.PROV SSM, Granular A or B and is properly compacted in accordance with OPSS 501 (Construction Specification for Compacting), significant settlement is not anticipated to occur within the embankment fill itself after construction.

6.2.4 Results of Unmitigated Settlement Analysis

For the settlement analyses, the proposed embankment widening of 3.5 m (to match existing Hwy 17 grade), and approximate ramp alignment / interchange grading was modelled as external loads based on the design drawings provided by AECOM. Settlements were estimated at the following four delineated areas / zones that were considered relevant for highway / ramp design:

- Existing Highway 17 EBL Shoulder from STA 14+300 to 14+350;
- Proposed Channelized Ramp Shoulder from Municipal Road 55 to Highway 17 STA 14+350;
- Proposed Highway 17 EBL Shoulder from 14+350 to 14+400; and,
- Proposed Highway 17 EBL Shoulder from 14+400 to 14+725.

The results of the settlement analyses are presented in the table below.

Area / Zone	Proposed Widening Width / Ramp Width (m)	20-year Post Construction Settlement (mm)	
		Total Magnitude	Differential (Longitudinal)
¹ Hwy 17 EBL STA 14+300 to 14+350 (Existing Shoulder)	3.5	50 to 175	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)	3.5	50 to 250	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)	3.5	25 to 225	<200H:1V
Hwy 17 EBL STA 14+400 to 14+725 (Proposed Shoulder)	3.5	~25	<200H:1V

¹ Backfill assumed to be placed at southeast quadrant of existing Hwy 17 / RR55 intersection.

Based on the results, the total settlements in three of the four areas / zones assessed exceed the MTO Foundation Embankment Settlement Guideline (MTO, 2010) that states settlements are to be less than 50 mm over a 20-year post construction period for a freeway.

We understand based on discussions with AECOM that a 5 m widening might be considered as the design progresses. Therefore, preliminary settlement estimates for a 5 m widening were also assessed and provided results similar to those presented in the table above. However, if a 5 m widening is deemed necessary, Golder should be provided the opportunity to review and revise the analyses to confirm estimated settlement magnitudes and the associated settlement mitigation measures, which are presented in Section 6.3 for a widening of 3.5 m only, as appropriate.

6.3 Settlement Mitigation Options

The unmitigated results of analysis based on the in-situ soil conditions and proposed 3.5 m embankment widening indicate that settlement mitigation will be required. Potential settlement mitigation options are generally discussed in the following sections, with the preferred mitigation options (from a foundations perspective) and details of each option presented in Section 6.4.

6.3.1 Full Sub Excavation

Sub-excavation of the compressible (i.e., clayey) deposits underlying the footprint of the proposed embankment widening in advance of the fill placement would improve the global stability of the embankment and reduce long-term settlement. We understand that based on MTO field experience on similar embankment construction projects, the practical maximum depths that can be reached with conventional and long stick excavator equipment is about 6 m and 12 m, respectively. Below a depth of 12 m, specialized drag line equipment would be required. As such, in the absence of unforeseen conditions which would prohibit its application, sub-excavation of organic and soft compressible soils is practical up to a depth of about 12 m. However, given the relatively thick clay deposits at this site (up to 20 m) and the existing adjacent embankment, full sub-excavation is not considered feasible at this site and is not discussed further herein.

6.3.2 Preloading

Preloading may be considered for reducing post construction settlements of the proposed embankments. Preloading refers to the placement of fill either up to the proposed profile grade and final widened condition of the highway/ramp in advance of embankment completion and final pavement construction. Preloading reduces the magnitude of long term, post construction settlements by allowing time for such settlements to occur during construction.

In general, preloading requires advanced placement of embankment fill and implementation of a wait period, during which time settlements and possibly pore water pressures are monitored. It should be noted that with preloading, it is still a requirement that all existing organic soils be removed prior to placement of any fill, because these soils are highly compressible and experience significant secondary (long-term) compression settlement.

In summary, the main advantages of this option are:

- reduced generation of excess excavation spoil compared with full sub-excavation;
- will not require a larger right of way corridor; and
- less embankment fill and a more stable embankment during construction as compared to surcharge.

The disadvantages of this option are:

- construction schedule is extended / delayed to allow for all or a portion of primary consolidation to be completed;
- may require lightweight fill for a portion of the construction of the final embankment widening to meet long-term post construction settlement criteria;
- requires a monitoring program to assess when the target degree of consolidation has been reached; and
- requires regrading to account for settlement prior to construction of the final pavement structure.

6.3.3 Surcharging

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements. The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the fill is placed to the full embankment height, followed by an additional lift(s) of fill (i.e., the surcharge) above that required to construct the final embankment geometry. This additional lift of fill applies greater stress to the underlying cohesive deposits and reduces the time to achieve the required primary consolidation settlement as compared to preloading only.

Depending on the additional fill height required for a surcharge placement, it could be necessary to construct toe berms or stage the placement of embankment fill and surcharge to limit the potential for instability. Depending on the stability conditions, toe berms required during the surcharge period may be temporary and could be fully or partially removed upon completion of the surcharge period. Upon the completion of the design surcharge period, the removed surcharge fill may be re used on other parts of the site.

Surcharging is well suited for locations where the stability of the higher surcharged embankment can be practically maintained by reasonably sized excavations, toe berms or staged construction, but where sufficient time for primary consolidation settlements to occur under preload fill loads alone is not available.

In summary, the main advantages of this option are:

- reduced generation of excess excavation spoil over full sub-excavation;
- potentially reduced secondary compression settlement;
- may not require a larger right of way corridor (depending on surcharge configuration and if toe berms are required); and,
- decreased wait time for construction over preloading alone.

The disadvantages of this option are:

- construction schedule is extended / delayed, albeit less than for preloading, to allow for primary consolidation to occur;
- longer construction time if staged construction is required;
- additional fill material required as compared to preloading;
- requires an instrumentation and monitoring program to assess when the target surcharge settlement is reached; and
- increased handling of the surcharge fill.

6.3.4 Wick Drains

Where sub-excavation is not practical (i.e., due to the thickness of or depth to the compressible soil deposits), and where the time required to reach the settlement performance criterion is considered too long, even with surcharging, consideration may be given to installing wick drains in conjunction with surcharging to accelerate the rate of primary consolidation. Wick drains are prefabricated geotextile drains installed vertically from ground surface into or through the soft, compressible foundation soils in order to increase the rate of excess pore pressure dissipation. Typically, wick drains are installed on a 1 m to 3 m triangular grid spacing over the embankment footprint.

The use of wick drains is most suited to areas with thick (i.e., greater than about 5 m) deposits of soft, compressible foundation soils and proposed embankment fills where primary consolidation times are considered too long for the proposed construction schedule.

Sub-excavation to remove the organic deposits and placement of a granular drainage blanket at ground surface would be required prior to the installation of the wick drains.

In summary, the main advantages of this option are:

- decreased primary consolidation time under surcharging; and
- decreased rate of staged construction, if required to maintain stability during construction.

The disadvantages of this option are:

- additional time and expense to install wick drains prior to embankment construction;
- additional long-term settlements due to creep settlement of the cohesive deposit (if not compensated for by surcharging or lightweight fill);
- requires an instrumentation and monitoring program to assess when the settlement performance criterion has been achieved; and
- requires re-grading to account for settlement prior to construction of the final pavement structure.

6.3.5 Lightweight Fill

Another alternative for reducing the magnitude of long-term settlement and improving stability in areas of weak/soft, compressible foundation soils is to use lightweight fill, for the embankment widening. The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill material. This in turn reduces the magnitude of post construction settlement and reduces the potential for instability.

Lightweight fill is not considered a practical option for general use due to the expense and/or shipping costs for the supply of these types of fills. Rather, lightweight fill is most suited for areas underlain by deep compressible subsurface deposits, where sub excavation is not practical or feasible, where long term post construction settlements affect the performance of the highway and where there is no available time in the construction schedule for a sufficient preload or surcharge period. In addition, lightweight fill can be used in conjunction with preloading, surcharging, and wick drain designs in order to optimize the design.

Expanded polystyrene could be considered; however, given the geometric constraints at this site, cellular concrete is likely more practical and constructible to counteract the potential buoyant effects of the high groundwater level. Cellular concrete is a product of cement, water, a foaming agent, and air placed by injecting air and foaming agent into a cement-water slurry to produce a cured concrete-like material with unit weights typically between 4 kN/m³ and 8 kN/m³. The cellular concrete should be placed in 0.5 m maximum lifts and the

next lift is not to be placed until the previous lift cures. As such, temporary protection systems adjacent to the cellular concrete, if required, will be subject to the fluid hydrostatic pressure during placement and curing. Cellular concrete has a further advantage that if future excavation is required in a specific area, the cellular concrete can be cut and excavated.

If considered suitable, it is recommended that the upper surface of the cellular concrete be sloped at least 2.5% toward the outside embankment slope. This slope during cellular concrete placement and finishing stage may not be reliably successful since cellular concrete is self-levelling. The slope should be provided by normal cast-in-place concrete, cutting and trimming once the cellular concrete achieves initial set or suitable alternative. Rock fill should not be used as the levelling pad due to the size of rock fill particles. As such, for this embankment, granular fill is to be used for levelling pad construction and for the side slope protective cover.

In summary, the main advantages of this option are:

- improved stability;
- reduced long-term post construction settlements; and
- shortened construction schedule.

The disadvantages of this option are:

- significant additional expense of lightweight fill (depending on the volume required); and
- not feasible to install below the groundwater table (due to buoyancy forces) and in low height embankments (due to minimum conventional soil cover requirements on top of the EPS).

6.4 Results of Mitigation Analysis

From the unmitigated settlement results, it should be noted that settlement mitigation measures are not anticipated to be required in the section / zone from STA 14+400 to 14+725, as the estimated settlement is within tolerance (less than 50 mm).

For the remaining three sections, given the maximum thickness of the cohesive deposits and the results of the consolidation testing, the time period to reach 90% of primary consolidation (t_{90}) is estimated to be greater than 10 years. The actual consolidation may occur more quickly in areas with lesser clay thickness or where the clay does not become normally consolidated; however, it is not practical to accommodate this length of wait period into the construction schedule. We understand that a 6-month wait period is reasonable for construction and as such, we limited assessments to this timeframe. Based on the project specific constraints and post construction settlement tolerances for freeways, the following settlement mitigation alternatives were evaluated to compare to the baseline unmitigated scenario discussed in Section 6.2:

- 6-month (i.e., 180 day) Preload Period
- 6-month (i.e., 180 day) Surcharge Period, assumed 1 m high surcharge with 1.5H:1V temporary side slopes given the limited platform width (i.e., 3.5 m wide)
- 6-month (i.e., 180 day) Surcharge Period, assumed 1 m high surcharge with 1.5H:1V temporary side slopes given the limited platform width (i.e., 3.5 m wide) and 1.5 m triangular spaced wick drains
- Lightweight fill with no wait period
- Lightweight fill after 6-month (i.e., 180 day) Preload Period

It should be noted that with the exception of the lightweight fill option, the mitigation will not reduce the total settlement experienced by the existing culverts along the Highway 17 embankment during construction. Therefore, consideration may need to be given to the installation of temporary culvert extensions as discussed in detail in the FIDR for the Fairbanks Creek Culvert and CSP Culverts.

The advantages, disadvantages, relative costs, and risks/consequences of the various mitigation alternatives are summarized in Table 1.

6.4.1 6-month (i.e., 180 day) Preload Period

A summary of the estimated settlements after a 180-day preload period are provided in the table below and indicate that post construction settlements exceed the target tolerance of less than 50 mm.

Area / Zone	Proposed Widening Width (m)	20-Year Post Construction Settlement (mm)	
		Total Magnitude	Differential (Longitudinal)
Hwy 17 EBL STA 14+300 to 14+350 (Existing Shoulder)	3.5	50 to 150	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)		25 to 175	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)		25 to 125	<200H:1V

6.4.2 6-month (i.e., 180 day) Surcharge Period

As the proposed embankment widening is limited to 3.5 m, a limited surcharge height would be practical to maintain traffic on the adjacent existing roadway. The results of the short-term stability analysis for a 1 m high surcharge with temporary surcharge side slopes of 1.5H:1V is presented in Figure 6 and indicates a satisfactory factor of safety against global instability for temporary conditions (as previously discussed, the stability analysis was carried out for a 5 m widening to allow for flexibility as the design progresses). The side slopes may require maintenance during the surcharge period if sloughing is noted within the surcharge fill.

A summary of the estimated settlements after a 180-day surcharge period are provided in the table below and indicate that post construction settlements exceed the target tolerance of less than 50 mm.

Area / Zone	Proposed Widening Width / Surcharge Height (m)	20-Year Post Construction Settlement (mm)	
		Total Magnitude	Differential (Longitudinal)
Hwy 17 EBL STA 14+300 to 14+350 (Existing Shoulder)	3.5 / 1	50 to 150	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)		25 to 175	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)		25 to 125	<200H:1V

6.4.3 6-month (i.e., 180 day) Surcharge Period with Wick Drains

As the proposed preload/surcharge wait period of six months had a limited impact on reducing post-construction settlements to acceptable levels, consideration could be given to wick drains with 1.5 m triangular spacing to further expediate the settlement to occur during construction, thereby reducing post-construction settlements. Based on a preliminary wick drain analysis using a 1.5 m triangular wick drain spacing installed through the clay deposit the degree of consolidation within the cohesive deposits would be about 70% in the thickest clay deposits.

A summary of the estimated settlements after a 180-day surcharge period with wick drains are provided in the table below and the results indicate that post construction settlements exceed the target tolerance of less than 50 mm.

Area / Zone	Proposed Widening Width / Surcharge Height (m)	20-Year Post Construction Settlement (mm)	
		Total Magnitude	Differential (Longitudinal)
Hwy 17 EBL STA 14+300 to 14+350 (Existing Shoulder)	3.5 / 1	25 to 75	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)		25 to 100	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)		25 to 75	<200H:1V

6.4.4 Lightweight Fill

An alternative to reduce post-construction settlement of the embankment is to incorporate lightweight fill into the embankment fill mass. The lightweight fill treatment could be limited to areas of particular concern (e.g., Fairbanks Creek Culvert) where larger settlements could cause large maintenance costs or lightweight fill could be applied in all areas where post-construction settlements are anticipated to exceed the target performance criteria.

For the purposes of the analysis, cellular concrete with a unit weight of 5 kN/m³ was assumed. A summary of the estimated settlements using cellular concrete (2 m thick) are provided in the table below.

Area / Zone	Proposed Widening Width / Cellular Concrete Fill Thickness (m)	20-Year Post Construction Settlement (mm)	
		Total Magnitude	Differential (Longitudinal)
Hwy 17 EBL STA 14+300 to 14+350 (Existing Shoulder)	3.5 / 2	25 to 75	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)	3.5 / 2	25 to 150	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)	3.5 / 2	<25 to 75	<200H:1V

Should lightweight fill be selected as the preferred settlement mitigation alternative, Golder could prepare an example specification to supply and install the cellular concrete for incorporation into the Contract documents.

6.4.5 6-month (i.e., 180 day) Preload Period with Earth Fill followed by Lightweight Fill Replacement

Given the relatively strict post-construction settlement criteria (i.e., <50 mm in 20 years), a combination of a 6-month preload period with earth fill followed by a partial replacement with lightweight fill may be considered.

For the purpose of the analysis, cellular concrete with a unit weight of 5 kN/m³ was assumed. A summary of the estimated settlements after the preload followed by replacement with a 3 m thick cellular concrete are provided in the table below. The results indicate that the target settlement tolerance of less than 50 mm can be achieved.

Section	Proposed Widening Width / Cellular Concrete Fill Thickness (m)	20-Year Post Construction Settlement (mm)	
		Total Magnitude	Differential (Longitudinal)
Hwy 17 EBL STA 14+300 to 14+350 (Existing Shoulder)	3.5 / 3*	<50	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)	3.5 / 3*	50	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)	3.5 / 3*	<50	<200H:1V

*Installed after preload period.

Should the preload and partial replacement with lightweight fill option be selected as the preferred settlement mitigation alternative, Golder could prepare an example specification to supply and install the cellular concrete for incorporation in the Contract documents.

6.5 Foundation Instrumentation and Monitoring

If the selected settlement mitigation measure incorporates a wait period, it is recommended that a monitoring program be implemented during construction to confirm that the measured field conditions are corresponding well with the design model estimates (i.e., validate design assumptions). If monitoring observations indicate that the field conditions are deviating from the design, the information would allow relevant stakeholders to make informed decisions on possible additional mitigation measures and/or schedule adjustments.

For the proposed embankment widening, the magnitude and time rate of settlement during and after construction of embankments should be assessed with monitoring instrumentation. Such monitoring would consist of installing settlement pins/stakes, and settlement plates (SPs) below the embankments and taking regular measurements/readings at given intervals of time during and after construction of the embankments for the duration of the wait period.

Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring frequency should be included as a Non-Standard Special Provision (NSSP) in the Contract Documents. A settlement monitoring NSSP for this site will be provided after the preferred alternative is selected, as appropriate.

6.6 Subgrade Preparation and Embankment Construction

The following sections discuss general aspects of subgrade preparation and embankment construction for this site, including removal of surficial and near surface organic materials; excavation and replacement of soft, cohesive deposits; groundwater control; and placement of embankment fills. Items related specifically to the Fairbanks Creek Culvert or the CSP Culverts are addressed in their respective reports and are not discussed further herein.

6.6.1 Temporary Excavations / Sub-excavation of Organic Deposits

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

It is anticipated that temporary excavations will consist of the existing embankment fill (Type 3 in OHSA), organic silts / peat (classified as Type 4 in OHSA) and may penetrate into the native firm to stiff silty clay to clayey silts (classified as Type 3 in OHSA). As such temporary excavations through Type 3 soils should maintain a minimum 1 Horizontal:1Vertical (1H:1V) temporary cut slope if workers are anticipated to enter the excavation. Temporary excavations through the organic deposits, or localized softer or saturated cohesionless soils, would be classified as Type 4 soils and temporary slopes should be no steeper than 3H:1V.

Based on the information gathered from the boreholes advanced during the field investigation, the thickness of organic deposits (i.e., peat, topsoil and organic silt) along the proposed embankment widening are generally up to about 1.3 m thick. At some locations (Borehole 21-04 and 21-05), the organic layer is encountered below the existing embankment fill and extends to depths up to 5 m below ground surface. Based on the ground surface contours in the area, it appears as though a previous fill pad / laydown area may have existed south of Highway 17 from about Station 14+400 to Station 14+500. Based on Boreholes 21-04 and 21-05, it is assumed the fill outside of the highway embankment was placed “uncontrolled” and the organic deposits were not removed prior to fill placement. As such, it is recommended that these materials be removed within the embankment widening footprint.

After clearing and grubbing the embankment footprint area and prior to the placement of any fill for new construction, all organic deposits within the new embankment widening footprint should be stripped and the existing embankment side-slopes temporarily cut in accordance with OPSS.PROV 209 (Embankments Over Swamp and Compressible Soils).

Temporary cuts / sub-excavation of organics soils adjacent the existing highway embankment should be carried out in strips of limited width to maintain stability and to protect the existing roadway during sub-excavation and replacement operations as follows:

- Removal of the organic deposits within the proposed embankment widening footprint should be carried out in accordance with OPSS.PROV 209 (Embankments Over Swamp and Compressible Soils) in sections perpendicular to the proposed roadway alignment.
- Temporary excavation side slopes or back slopes through the organic deposits should be no steeper than 1.25H:1V (if no persons need to enter the excavation) or 3H:1V if personnel will be entering the excavation (as per OHSA) and be in accordance with OPSD 203.020 (Embankments Over Swamp).
- If localized areas are encountered where significant organic deposits existing below the existing embankment, some distress to the existing roadway / shoulder could occur during the staged excavation and, as such, consideration for a provision to temporarily close shoulders / divert traffic should be included in the Contract to maintain the safe operation of Highway 17 during the excavation and backfilling operations.

The recommendations provided above should be incorporated into a Non-Standard Special Provision (NSSP) in the Contract (an example is included in Appendix C).

6.6.2 Groundwater and Surface Water Control

Temporary excavation within the plan limits of the proposed works will be required to remove existing fills and organic deposits prior to embankment fill placement, which will extend below the groundwater table that is anticipated to be at about ground surface adjacent to the toe of the existing embankment (about Elevation 241 m to 243 m) at the site. It is noted that artesian groundwater conditions may be present within the underlying granular soils (below the silty clay deposit) near the north limit of the site, where groundwater levels were measured to be above ground surface and/or within the existing embankment fill. Given the relatively shallow depth of sub-excavation required on site, the potential artesian conditions are not anticipated to impact construction operations.

Groundwater flow into the excavations will occur due to the presence of relatively permeable deposits and relatively high groundwater levels observed during the field investigation. Unwatering / dewatering is not considered to be required for the excavation and backfilling provided that OPSS.PROV 1010 Granular A or B Type II soils are used as backfill below the groundwater level, however, surface water should be directed away from the excavations at all times.

6.6.3 Backfilling

Sub-excavation of organic soils within the proposed embankment widening (adjacent the existing embankment) and along the proposed new ramp are anticipated as discussed in the previous sections.

From a foundations perspective, it is recommended that granular fill meeting OPSS PROV. 1010 (Aggregates) Granular 'B' Type II with 100% passing the 26.5 mm sieve size and not more than 5 per cent passing the 0.075 mm sieve size be used for the replacement of the sub-excavated material below the groundwater level.

Where sub-excavation of organic deposits are being carried out in wet / saturated conditions, it will not likely be possible to place the fill in accordance with OPSS PROV. 206 (Construction Specification for Grading); therefore, granular fill placed below the groundwater table should be in accordance with OPSS PROV. 209, followed by placement of embankment fill above the groundwater level as per the next section.

6.6.4 Embankment Fill Placement

Placement of granular fill above the groundwater table for construction of the embankment widening should be carried out in accordance with the requirements as outlined in OPSS.PROV 206. The embankment widening should be benched into the existing embankment in accordance with OPSD 208.010, with a minimum bench width of 1 m.

Compaction of the embankment fill above the water table should be in accordance with OPSS.PROV 501 (Construction Specification for Compacting). Side slopes for the granular fill should be no steeper than 2H:1V for construction of the proposed embankment.

If applicable, any surcharge fill should be constructed with temporary side slopes no steeper than 1.5H:1V side.

6.6.5 Slope Flattening

It is understood that consideration is being given to flattening the proposed embankment side-slopes (see OPSD 203.020) using surplus materials (from excavations or pavement rehabilitation operations). However, depending on the type of material used, and the timing of placement of the surplus material, slope flattening may adversely affect the long-term performance of the highway by inducing further post-construction settlement.

It is assumed that the embankment fill side slopes will be constructed at an inclination of 2H:1V before slope flattening. It is also understood that the material used for the slope flattening could consist of granular fill or excess earth material, excavated elsewhere or locally.

Considerations with respect to impact to the highway embankment settlement and stability if slope flattening is being considered are discussed below.

6.6.5.1 Stability

In general, global stability is enhanced when side slopes are flattened, hence instability of the final embankment slopes is not an issue.

6.6.5.2 Settlement

Post-construction settlement of the embankment will occur as a result of placement of the excess material in the slope flattening areas of the embankment. Therefore, the timing of placement of the additional/excess material load should be considered in determining whether slope flattening should be implemented. Three scenarios are presented below for different stages of placement of the additional slope flattening material as well as the corresponding settlement implications.

- Concurrently with construction of the embankment. This construction method would produce the least amount of post-construction settlement of the roadway embankment; although depending on the settlement mitigation option selected, this could extend preload / surcharge wait times and will increase settlement magnitudes.
- After construction of the embankment to preload/surcharge height, but prior to the wait period. Any settlement induced prior to construction of the final roadway can be accommodated by grading operations. However, depending on the settlement mitigation option selected, this could extend preload / surcharge wait times and will increase settlement magnitudes.
- After the preload / surcharge period is complete. This construction method imposes additional loads from the slope flattening material, which will cause immediate and long-term settlement beneath the embankment side slopes and potentially below the roadway; therefore, from a Foundations perspective, this is the least preferred construction method for slope flattening.

Given the construction scenarios described above it is recommended that, if desired, slope flattening in the area of the embankment widening be carried out concurrently with embankment (i.e., prior to any preload/surcharge wait period). The effects on preload/surcharge wait times and settlement magnitudes would need to be assessed when the geometry and details of the slope flattening are provided.

6.6.5.3 Embankment Platform Widening

In accordance with the requirements of MTO Northern Region Engineering Directive NRE 98 200, "Northern Region Embankment Design Guidelines", the construction of the embankments should include an allowance for platform widening (in 0.5 m increments) to accommodate settlements during construction as well as post construction settlements, so that the minimum standard shoulder widths are maintained if future grade raises on the embankments are required. According to NRE 98 200, the need for future raises in road grade could occur due to settlement / compression of the embankment fill, settlement of the foundation soils and to accommodate future pavement overlays up to 200 mm thick. It is understood that this directive applies to all rock fill embankments, as well as for granular fill embankments where widening restrictions are present (such as the presence of a sensitive body of water or due to space/property issues). It is further understood that the minimum required platform widening on major highways (i.e., including Highway 17) over soft ground is 2 m per side,

unless the preferred mitigation option eliminates uncertainty regarding embankment settlement/performance (i.e., full sub excavation to bedrock and backfilling with granular material).

The minimum required embankment platform widening is calculated based on the estimated consolidation settlement of the foundation soils (including secondary compression) plus an additional 200 mm for the future pavement overlay, multiplied by the horizontal component of the side slope of the pavement structure (3H:1V), but cannot be less than the minimum platform widening requirements as described above.

Therefore, the platform widening required to account for post-construction settlement and future overlay at this site is 2 m; thus, the final embankment widening should take this into account.

7.0 CLOSURE

This report was prepared by Mr. Matthew Thibeault, P.Eng., a geotechnical engineer with Golder. Mr. Kevin Bentley, P.Eng., Golder's Designated MTO Foundations Contact for this project and an Associate of Golder, conducted an independent quality review of the report.

Signature Page

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[https://golderassociates.sharepoint.com/sites/128666/Project Files/6 Deliverables/Foundations/Predraft/R1-Highway 17 Embankment Widening/20253807-R01-RevA-Hwy 17 Emb.Widening FIDR Draft 16Jul_2021.docx](https://golderassociates.sharepoint.com/sites/128666/Project%20Files/6%20Deliverables/Foundations/Predraft/R1-Highway%2017%20Embankment%20Widening/20253807-R01-RevA-Hwy%2017%20Emb.Widening%20FIDR%20Draft%2016Jul_2021.docx)

REFERENCES

- Azzouz, A.S., Krizek, R.J., and Corotis, R.B. 1976. Regression Analysis of Soil Compressibility, Soils and Foundations. 16(2): 19-29.
- Bjerrum, L., 1973. Problems of Soil Mechanics and Construction of Soft Clays and Structurally Unstable Soils. State of the art Report, Session 4. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Vol. 3, pp. 111-159.
- Canadian Standards Association (CSA), 2019. Canadian Highway Bridge Design Code and Commentary. CSA Special Publication.
- Koppula, S.D. 1981. Statistical Estimation of Compression Index, Geotechnical Testing Journal, GTJODJ. 4(2): 68-73.
- Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- Ladd, C.C. and Foott, R. 1977. Foundation Design of Embankments Constructed on Varved Clays. FHWA TS-77-214. US Department of Transportation, Federal Highway Administration, Contract No. DOT-FH-11-8973.
- Mitchell, J. 1993. Fundamentals of Soil Behavior. Wiley New York, Second Edition, pp. 335.
- Mesri, G. 1975. Discussion on New Design Procedure for Stability of Soft Clays. ASCE Journal of the Geotechnical Engineering Division, Vol. 101, GT4, pp. 409-412.
- Mesri, G., Kwan, L., and Feng, T.W. 1994. Settlement of Embankments on Soft Clays. In the Proceedings of the Conference on Vertical and Horizontal Deformations of Foundations and Embankments. Part 2 (of 2), College Station, TX, USA. 8-56.
- Ministry of Natural Resources. 1991. Bedrock Geology of Ontario – Map 2542 West Central Sheet.
- Ministry of Transportation, Ontario:
- Embankment Settlement Criteria for Design, July 2010.
 - Geocres No. 32D-025, Foundation Investigation and Assessment Report. Swamp Crossing H6/H7 Realignment of Highway 66 at Virginiatown from 14+020 to 14+650, Contract 2015-5121, Township of McGarry. Agreement No. 5015-E-0045 – Work Order 12.
 - Geocres No. 41I-092, Preliminary Investigation for Fairbanks Creek Structures, Highway 17, Line D, District 17 (Sudbury), Township of Denison, Reg. Mun. of Sudbury. W.P. 61-74-02&03, Site 46-298A&B.
 - Geocres No. 41I-323, Foundation Investigation and Design Report. High Fill Embankments over Swamps, Township of Denison, Highway 17 Four-laning Extension from 20.5 km to West of Highway 144, Easterly for 6.5 km. GWP 156-98-00.
 - Geocres No. 41I-325, Foundation Investigation and Design Report. Fairbanks Creek Culvert – Station 14+420, Township of Denison, Highway 17 Four-laning Extension from 20.5 km to West of Highway 144, Easterly for 6.5 km. GWP 156-98-00.

Northeastern Region Geotechnical Section Memorandum. "Use of Mid-Slope Berms for Rockfill Embankments, Northeastern Region" dated February 8, 2005.

Northern Region Engineering Directive NRE 98-200. Northern Region Embankment Design Guidelines. October 1998.

Naval Facilities Engineering Command. 1986. Foundations & Earth Structures. Design Manual 7.02.

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

Ontario Geological Society. Northern Ontario Engineering Geology Terrain Study. Digital Map 41ISW.

Tavenas, F. and Leroueil, S. 1980. The behaviour of embankments on clay foundations. Canadian Geotechnical Journal, 17, pp. 236-260.

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils

Commercial Software:

Settle3 (Version 5.010) by Rocscience Inc.

GeoStudio (Version 11.0.1.21429) by GEOSLOPE International Ltd.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Drawing:

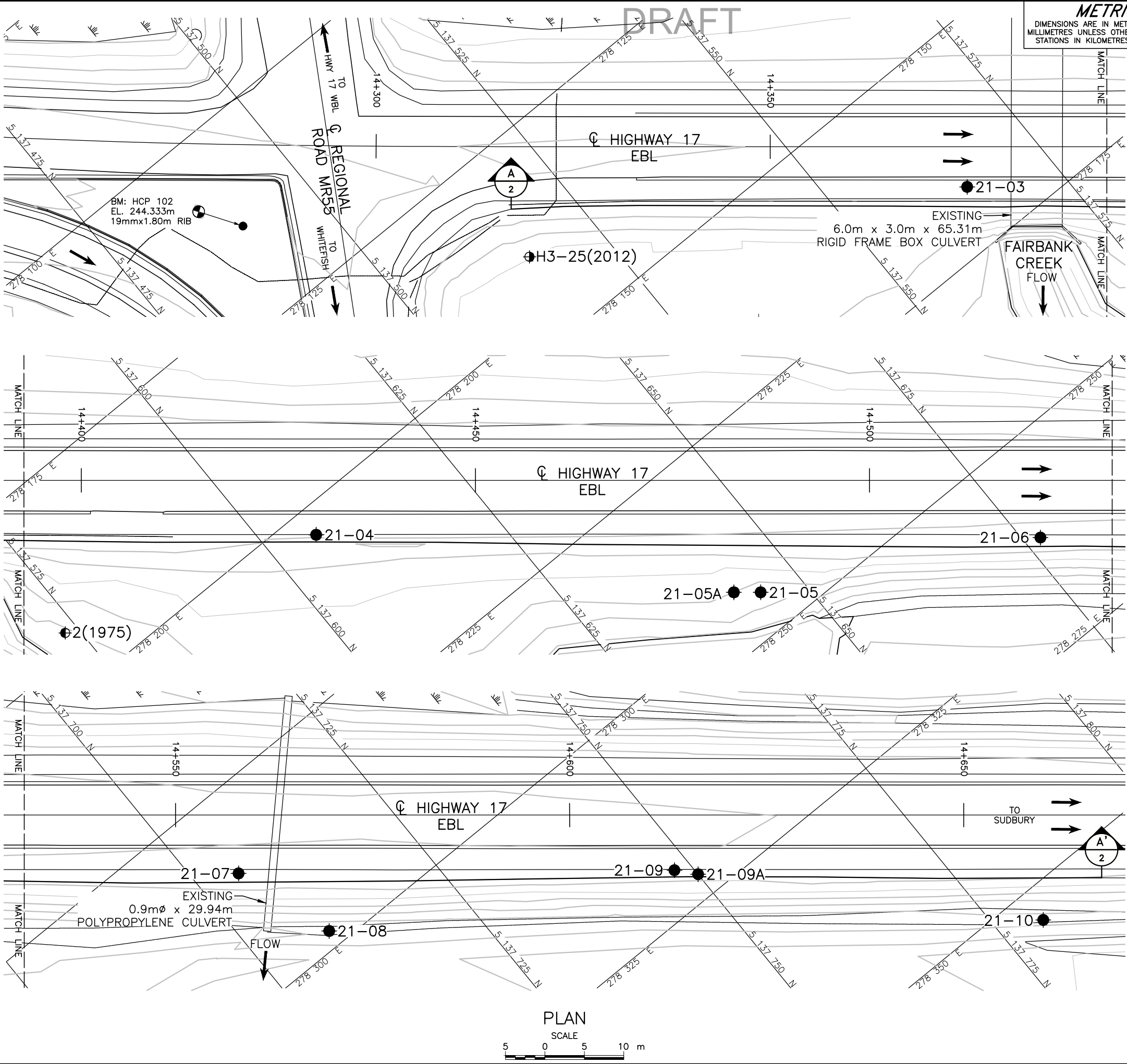
OPSD 203.020 Embankments Over Swamp, Existing Slope Excavated

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 1010	Material Specification for Aggregates (Base, Subbase, Select Subgrade and Backfill Material)


Ontario Water Resources Act:

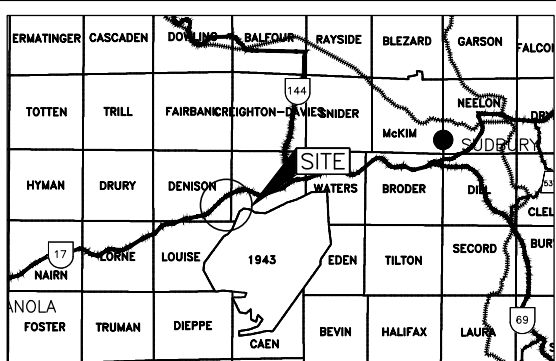
Ontario Regulation 903 Wells (as amended)



CONT No.
GWP No. 5032-19-00

HIGHWAY 17
EMBANKMENT WIDENING
EBL STA. 14+300 TO 14+725
BOREHOLE LOCATIONS

**GOLDER**
MEMBER OF WSP



KEY PLAN
SCALE
10 0 10 20 km

LEGEND

- Borehole - Current Investigation
- ⊕ Borehole - Previous investigation (MTO-1975)
- ⊕ Borehole - Previous investigation (Golder-2012)

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
21-03	243.8	5137564.6	278167.4
21-04	243.6	5137605.9	278203.5
21-05	242.2	5137645.0	278244.8
21-06	243.4	5137676.9	278261.9
21-08	241.0	5137709.2	278297.9
21-07	243.3	5137704.9	278285.0
21-09	243.3	5137748.0	278319.6
21-10	241.2	5137780.3	278354.1
21-05A	242.2	5137642.4	278242.7
21-09A	243.3	5137750.0	278321.9
2(1975)	240.9	5137573.4	278193.0
H3-25(2012)	242.2	5137516.0	278139.2

DRAFT

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 CANADA LTD., drawing file no. Hwy 17-MR55.dwg, received MARCH 8, 2021.

NO.	DATE	BY	REVISION

Geocres No.,

HWY. 17	PROJECT NO. 20253807	DIST. .
SUBM'D.	CHKD. TB	DATE: 7/16/2021
DRAWN: TR	CHKD. MT	APPD. .

DWG. 1



NO.	DATE	BY	REVISION
Geocres No.			
HWY. 17		PROJECT NO. 20253807	DIST. .
SUBM'D.	CHKD. TB	DATE: 7/16/2021	SITE:
DRAWN: TR	CHKD. MT	APPD.	DWG. 2



Photograph 1: Highway 17 (EBL) South Embankment Slope, Looking Northeast



Photograph 2: Highway 17 (EBL) South Embankment Slope, Looking Southwest



Photograph 3: Highway 17 (EBL) South Embankment Slope, Looking Southwest



Photograph 4: Highway 17 (EBL) South Embankment Slope, Looking Northeast

**Table 1: Evaluation of Settlement Mitigation Options
Highway 17 – STA 14+300 to 14+725**

Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<p>Preloading following by removal/replacement with Lightweight Fill (2 m Cellular Concrete)</p> <p>■ 6-month preload</p>	1	<p>■ Meets settlement criteria.</p> <p>■ Preload wait period allows some settlement to occur during construction, thereby reducing post-construction total settlement of foundations soils.</p> <p>■ Lightweight fill installed after preload wait period completed will reduce total load on foundation soils, thereby further reducing post-construction settlement.</p>	<p>■ Restricted thickness / density that can be used dependent on overall thickness of embankment / groundwater/water level.</p> <p>■ Wait period needs to be incorporated into construction schedule to allow for sufficient settlement to meet post-construction settlement criterion even with lightweight fill replacement.</p> <p>■ Double handling of fill for preload and removal / replacement with cellular concrete.</p> <p>■ Re-grading is required prior to final pavement structure construction.</p>	<p>■ Cellular concrete more expensive material compared to conventional embankment fill.</p> <p>■ Increased cost for double handling of preload fill to install cellular concrete at end of preload period compared to other options.</p>	<p>■ Risk of experiencing unexpected post-construction settlements unless instrumentation and monitoring plan incorporated.</p> <p>■ Risk that additional time may be required for extended preloading based on the monitoring data results.</p> <p>■ Availability of cellular concrete may increase costs.</p>
<p>Wick Drains with Surcharge (1 m high)</p> <p>■ > 6-month surcharge</p>	2	<p>■ Reduces post-construction settlement.</p>	<p>■ 6-month wait time does not meet settlement criteria; thus, addition time would be needed, or settlement criteria relaxed.</p> <p>■ Increasing surcharge height will impact Highway traffic unless retaining</p>	<p>■ Increased cost for surcharge fill material compared to preload, although excess material could be reused on site.</p> <p>■ Additional cost for wick drains installation</p>	<p>■ Risk that wait time to achieve settlement criteria (> 6 months) may not be practical for construction schedule.</p> <p>■ Risk of experiencing unexpected post-construction settlements unless instrumentation and monitoring plan incorporated.</p>

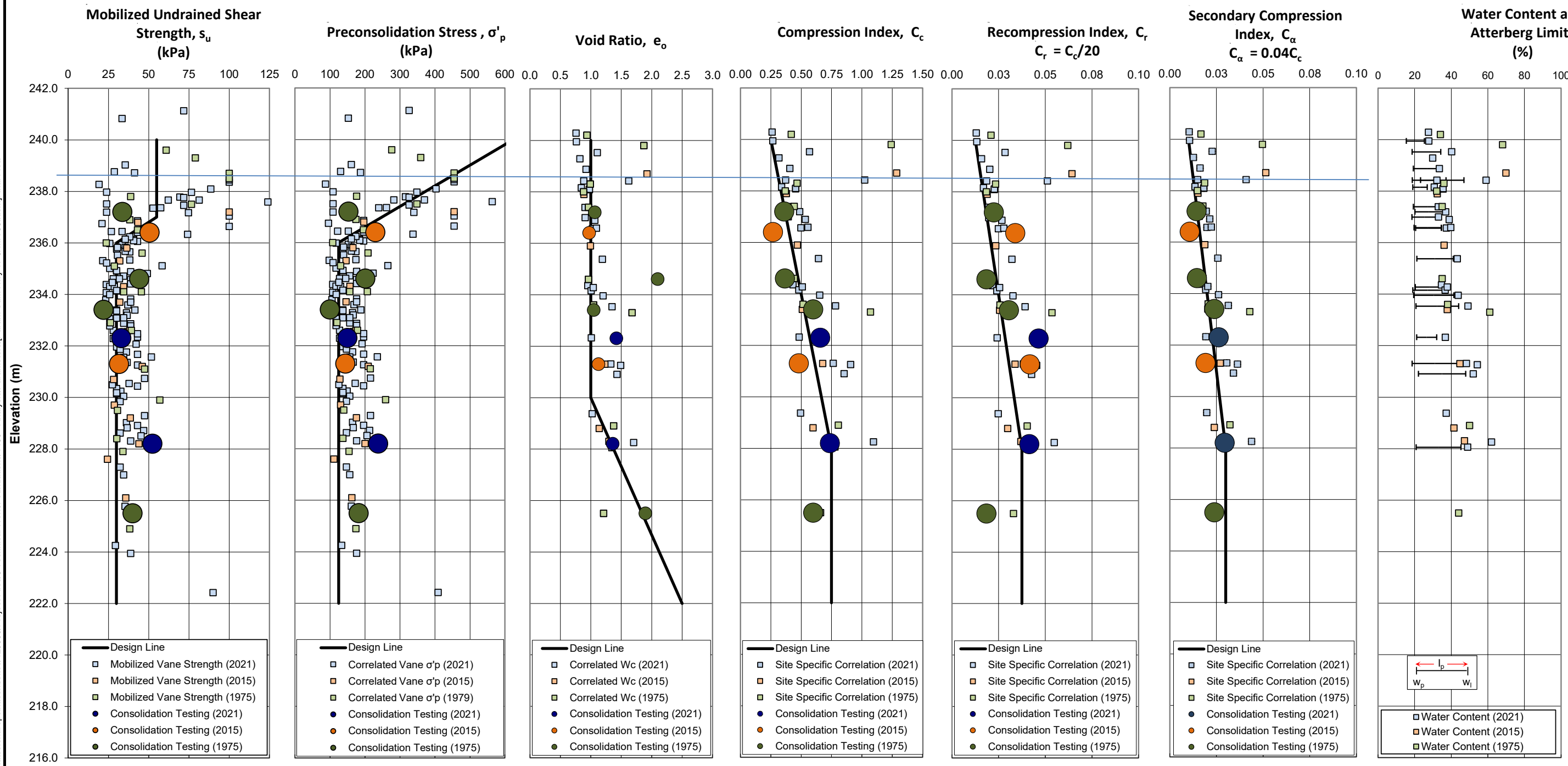
Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<p>walls / systems considered.</p> <ul style="list-style-type: none"> ■ Re-grading is required prior to final pavement structure construction. ■ Time and cost of wick drain and instrumentation installation. 	<p>and/or temporary protection systems to contain surcharge height so as not to impact Highway 17.</p>	<ul style="list-style-type: none"> ■ Low risk that additional time may be required beyond design wait period, but delay period is subject to the monitoring data. ■ Potential artesian conditions impacting performance of wick drains.
<p>Lightweight Fill (2 m Cellular Concrete)</p> <ul style="list-style-type: none"> ■ 6-month preload 	3	<ul style="list-style-type: none"> ■ Lightweight fill installed during construction will reduce total load on foundation soils thereby reducing post-construction total settlements. 	<ul style="list-style-type: none"> ■ 6-month wait time does not meet settlement criteria; thus, additional time would be needed, or settlement criteria relaxed. ■ Restricted thickness that can be used dependent on overall thickness of embankment and groundwater/water level. 	<ul style="list-style-type: none"> ■ Cellular concrete more expensive material compared to conventional embankment fill. 	<ul style="list-style-type: none"> ■ Risk that wait time to achieve settlement criteria (likely > 2 year) may not be practical for construction schedule. ■ Risk of experiencing unexpected post-construction settlements unless instrumentation and monitoring plan incorporated.
<p>Surcharge (1 m high)</p> <ul style="list-style-type: none"> ■ 6-month surcharge 	4	<ul style="list-style-type: none"> ■ Standard construction operation. ■ Reduces post-construction settlement. 	<ul style="list-style-type: none"> ■ Does not meet settlement criteria. ■ Wait time in construction schedule to allow for settlement to occur during construction. 	<ul style="list-style-type: none"> ■ Additional cost for surcharge fill material compared to preload, although excess material can be reused on site. ■ Lower initial cost but high frequent maintenance costs to address settlements 	<ul style="list-style-type: none"> ■ Risk that wait time to achieve settlement criteria (likely > 5 year) may not be practical for construction schedule. ■ Risk that additional time may be required for continued surcharge time based on the monitoring data results. ■ Low risk of experiencing unexpected post-construction settlement.

Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Preloading ■ 6-month preload	5	■ Standard construction operation. ■ Reduces post-construction settlement. ■ Makes use only of fill material that is required for embankment construction.	■ Does not meet settlement criteria. ■ Wait time in construction schedule to allow for settlement to occur during construction.	■ Lowest initial cost but settlement criteria not met. ■ High frequent maintenance costs to address settlements.	■ Risk that wait time to achieve settlement criteria (likely > 5 year) is not practical for construction schedule. ■ Risk that additional time may be required for continued preloading based on the monitoring data results. ■ Low risk of experiencing unexpected post-construction settlement.
Full Sub-Excavation (up to 19 m deep)	NF	Reduces magnitude of total settlement of foundations soils as soft compressible material has been removed.	■ Generation of very large volume of excess excavation spoil. ■ Very large quantity of back fill required. ■ Longer construction period required to sub-excavate to 19 m depth and backfill. ■ Specialized equipment / temporary support systems / dewatering and high effort required for deep sub-excavation and replacement. ■ Unable to compact large thickness of end dumped backfill below water table.	■ Highest cost for sub-excavation (long-stick or drag-line) equipment and/or temporary protection systems, disposal and replacement of weak/soft, compressible deposits. ■ Potential additional costs to acquire additional property.	■ High risk of not achieving/maintaining stability of excavation slopes and temporary protection system required. ■ Low risk of experiencing unexpected post-construction settlements associated with long term rock fill settlement as long as all soft compressible clay soils are removed. ■ High risk that not all compressible soils are removed during the sub-aqueous operations which could lead to unexpected settlement.

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SUMMARY OF ENGINEERING PARAMETERS
FOR COHESIVE DEPOSITS
Highway 17 and MR 55 Widening

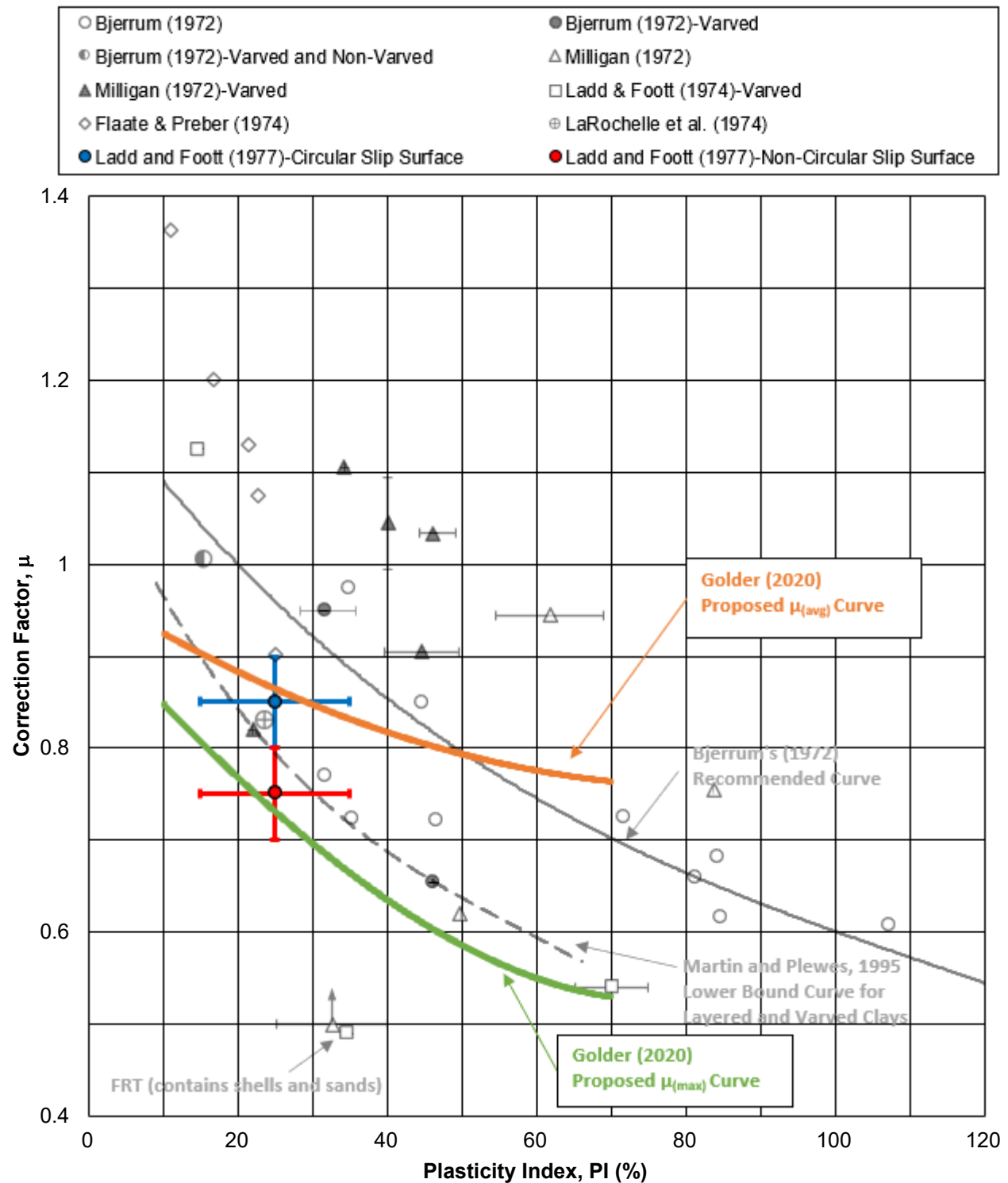
FIGURE 1



https://golderassociates.sharepoint.com/sites/128666/Project Files/5 Technical Work/Foundations/1400 - Analysis/Parameters/[Param Sum Hwy 17 & MR 55.xlsm]Final Plots









PROPOSED FIELD VANE CORRECTION FACTORS FOR VARVED CLAYS

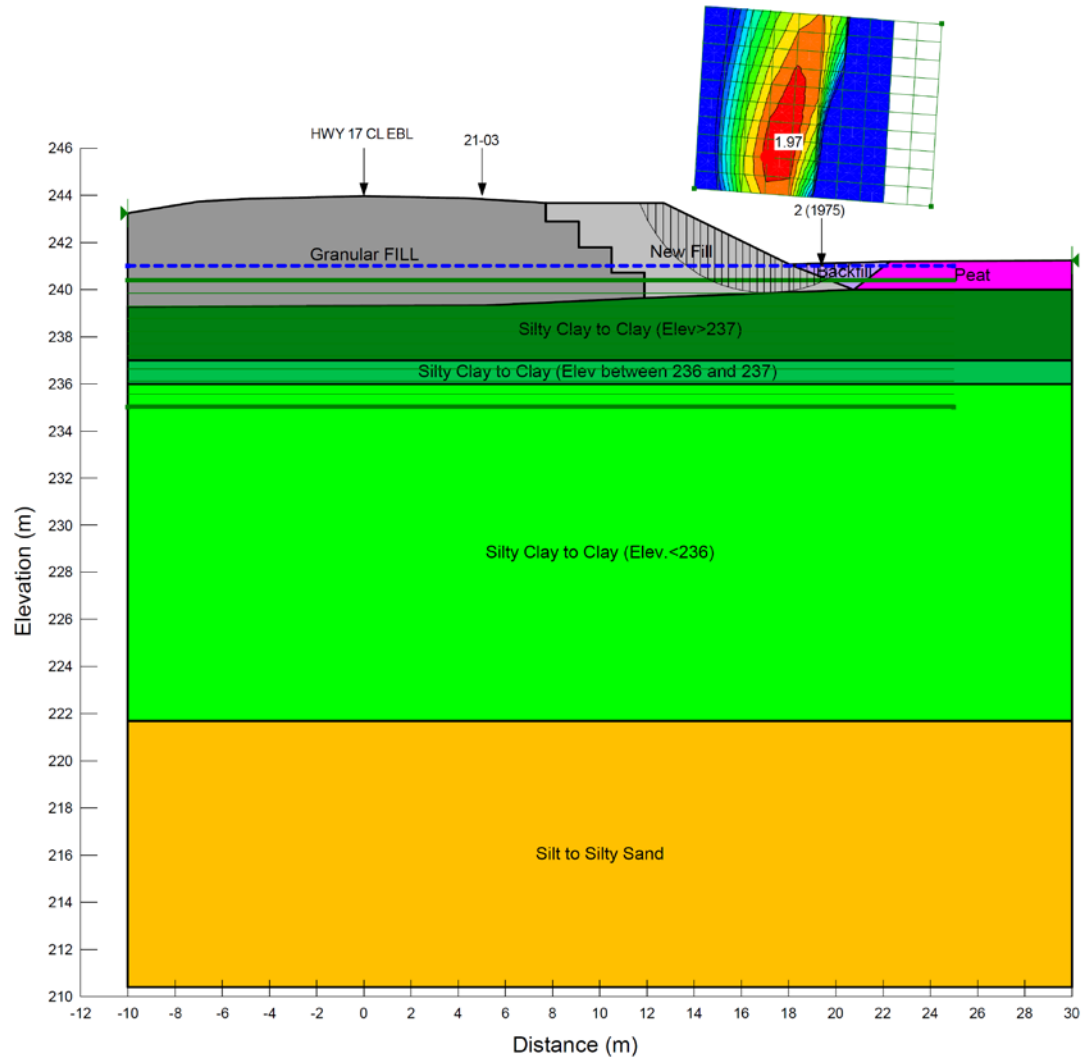
FIGURE 2



(after Ladd et al., 1977; from Ladd and DeGroot, 2003)

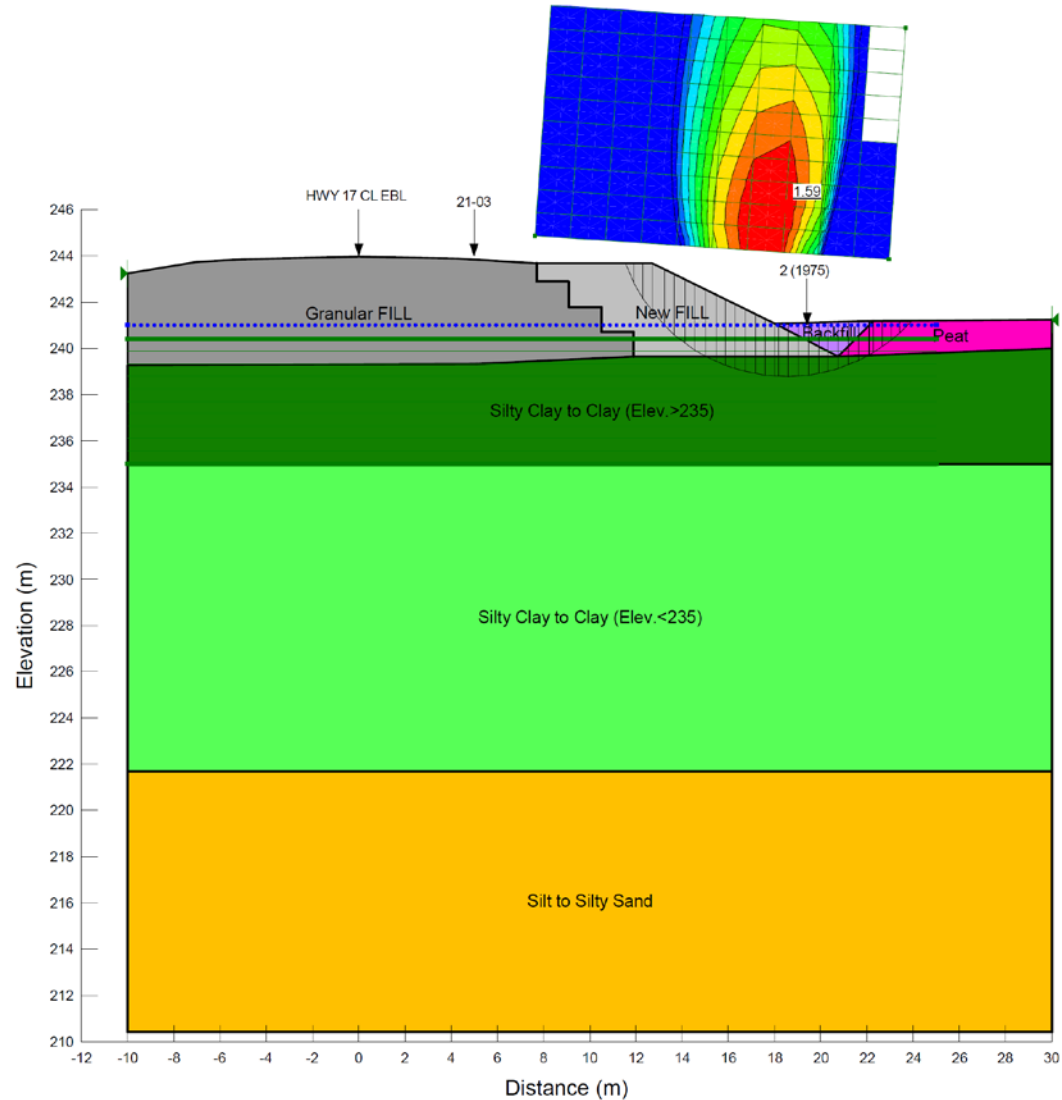
Station 14+380, Short-term Condition (Total Stress Parameters)

Color	Name	Unit Weight (kN/m ³)	Cohesion (kPa)	C-Datum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Backfill	12.5			1	27.5
	Granular FILL	20			0	35
	New Fill	20			0	35
	Peat	12.5			1	27.5
	Silt to Silty Sand	18			0	28
	Silty Clay to Clay (Elev between 236 and 237)	18		55		
	Silty Clay to Clay (Elev.<236)	18	30			
	Silty Clay to Clay (Elev>237)	18	55			



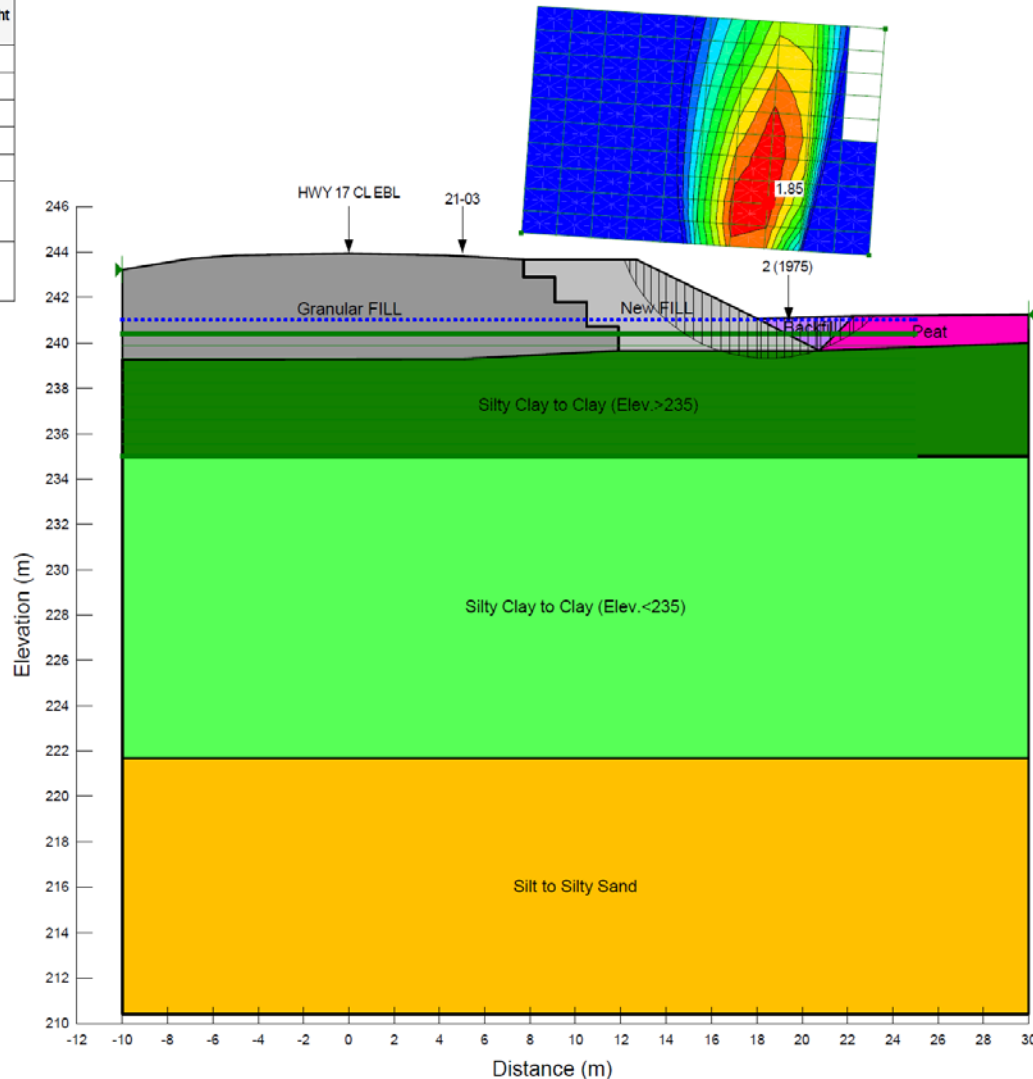
Station 14+380, Short-term Condition (Effective Stress Parameters)

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	B-bar	Add Weight
	Backfill	12.5	1	27.5	0	No
	Granular FILL	20	0	35	0	No
	New FILL	20	0	35	0	Yes
	Peat	12.5	1	27.5	0	No
	Silt to Silty Sand	18	0	28	0	No
	Silty Clay to Clay (Elev.<235)	18	0	29	1	No
	Silty Clay to Clay (Elev.>235)	18	0	31	0.3	No



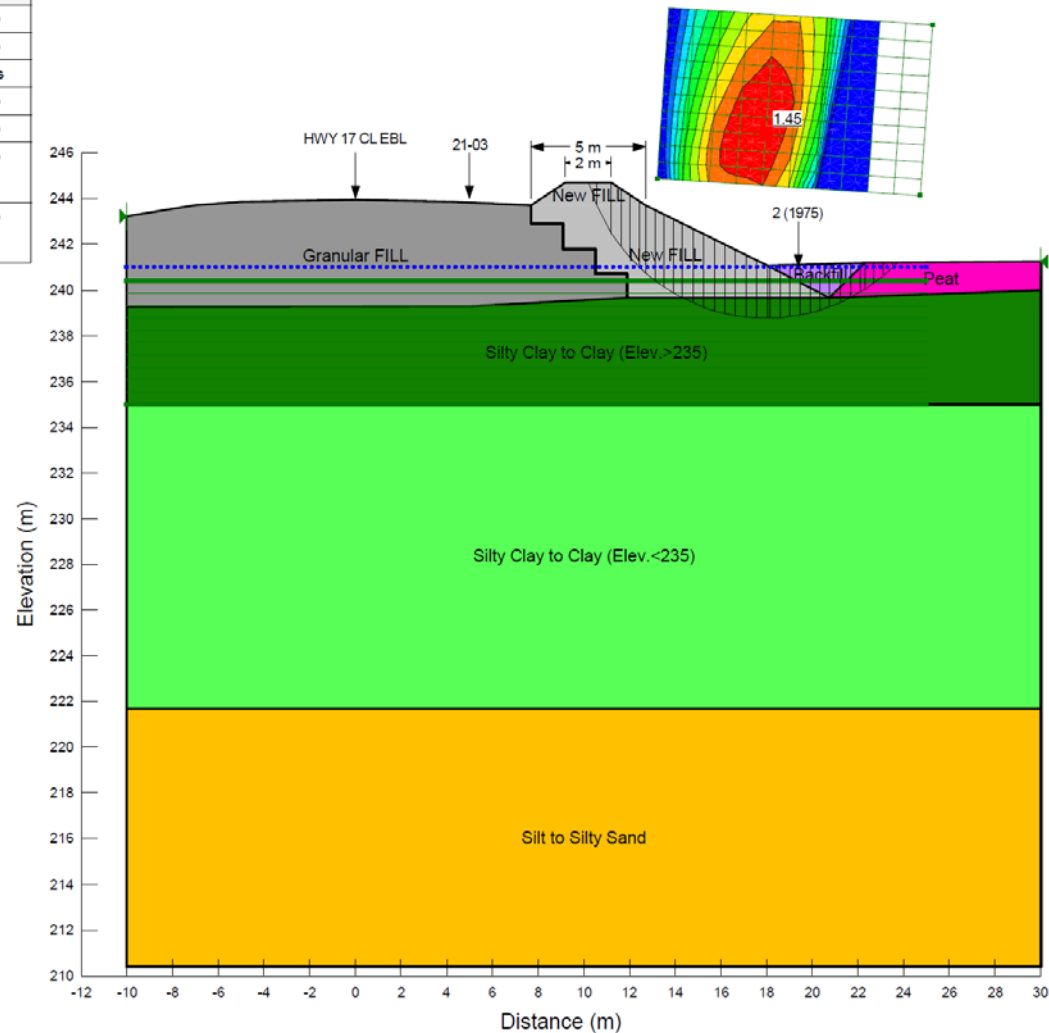
Station 14+380, Long-term Condition (Effective Stress Parameters)

Color	Name	Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	B-bar	Add Weight
	Backfill	12.5	1	27.5	0	No
	Granular FILL	20	0	35	0	No
	New FILL	20	0	35	0	Yes
	Peat	12.5	1	27.5	0	No
	Silt to Silty Sand	18	0	28	0	No
	Silty Clay to Clay (Elev.<235)	18	0	29	0	No
	Silty Clay to Clay (Elev.>235)	18	0	31	0	No



**Station 14+380, Surcharge, Short-term Condition
(Effective Stress)**

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	B-bar	Add Weight
	Backfill	12.5	1	27.5	0	No
	Granular FILL	20	0	35	0	No
	New FILL	20	0	35	0	Yes
	Peat	12.5	1	27.5	0	No
	Silt to Silty Sand	18	0	28	0	No
	Silty Clay to Clay (Elev.<235)	18	0	29	1	No
	Silty Clay to Clay (Elev.>235)	18	0	31	0.3	No



APPENDIX A

Record of Boreholes

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ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

- Only applicable to components not described by Primary Group Name.
- Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

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

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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, 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
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

- Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

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LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

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

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (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 = $(w_L - w_P)$
NP	non-plastic
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
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified 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
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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 or 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
2

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

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

PROJECT <u>20253807</u>			RECORD OF BOREHOLE No. 21-03			2 OF 2 METRIC		
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137564.6; E 278167.4 NAD83 MTM ZONE 12 (LAT. 46.377267; LONG. -81.346208)</u>			ORIGINATED BY <u>TB/NP</u>		
DIST <u> </u> HWY <u>17</u>			BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>TR</u>		
DATUM <u>GEODETIC</u>			DATE <u>February 8, 2021</u>			CHECKED BY <u>MT</u>		

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa		W _p	W		
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
227.9	SILTY CLAY (CI) Firm to very stiff Grey Wet - Laminations of clayey silt observed in split-spoon sample No. 12.		12	SS	WH		20 40 60 80 100 + 4 + 4 + 6						
			13	TO	PH								
			14	SS	WH								
15.9	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 3.2 m below ground surface (Elev. 240.6 m) inside augers upon completion of drilling.												

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

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

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PROJECT <u>20253807</u>			RECORD OF BOREHOLE No. 21-04			3 OF 3 METRIC		
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137605.9; E 278203.5 NAD83 MTM ZONE 12 (LAT. 46.377641; LONG. -81.345741)</u>			ORIGINATED BY <u>AD/NP</u>		
DIST <u> </u> HWY <u>17</u>			BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>TR</u>		
DATUM <u>GEODETIC</u>			DATE <u>February 4, 2021</u>			CHECKED BY <u>MT</u>		

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 20 40 60					
	--- CONTINUED FROM PREVIOUS PAGE ---															
218.6	SILT (ML), trace to some sand, trace clay Very loose to compact Grey Wet		20	SS	12	219										
25.0	END OF BOREHOLE NOTE: 1. Water level measured at a depth of 9.2 m below ground surface (Elev. 234.4 m) in augers upon completion of drilling.															

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PROJECT 20253807			RECORD OF BOREHOLE No. 21-05/21-05A					1 OF 1		METRIC								
G.W.P. 5032-19-00			LOCATION N 5137645.0; E 278244.8 NAD83 MTM ZONE 12 (LAT. 46.377996; LONG. -81.345206)					ORIGINATED BY AD										
DIST HWY 17			BOREHOLE TYPE CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers					COMPILED BY TR										
DATUM GEODETIC			DATE February 2, 2021					CHECKED BY MT										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED										
242.2	GROUND SURFACE						20 40 60 80 100											
0.0	TOPSOIL (50 mm)																	
0.1	SILTY CLAY (CI), trace organics (FILL) Firm to stiff Brown Moist		1	SS	8													
			2	SS	5													
240.5	ORGANIC SILT (OL) Very loose to loose Dark grey Moist		3A	SS	4													
			3B															
239.6	Gravelly SAND (SP), some silt, trace clay Very loose to compact Grey Wet		4A	SS	11													
			4B															
238.5	CLAYEY SILT (CL) Firm to stiff Grey Wet		5	SS	2													
			6	SS	4													
			7	TO	PH													
			8	SS	WH													
236.2	SILTY CLAY (CI) Firm Grey Wet		9	TO	PH													

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

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PROJECT <u>20253807</u>		RECORD OF BOREHOLE No. 21-06		1 OF 2 METRIC	
G.W.P. <u>5032-19-00</u>		LOCATION <u>N 5137676.9; E 278261.9 NAD83 MTM ZONE 12 (LAT. 46.378282; LONG. -81.344986)</u>		ORIGINATED BY <u>AD</u>	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 3, 2021</u>		CHECKED BY <u>MT</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	W _p			W	W _L	
243.4	GROUND SURFACE																
0.0	Gravelly SAND (SP) to SAND (SP), trace gravel (FILL) Loose to very dense Brown Moist to wet		1	AS	-												
			2	SS	23												
			3	SS	37												
			4	SS	52												
			5	SS	10												
239.5			6	SS	4												
3.9	CLAYEY SILT (CL), trace organics in upper zone Stiff Grey Wet		7	SS	6												
			8	SS	2												
236.5			9	SS	WH												
6.9	SILTY CLAY (CI) Firm Grey Wet		10	SS	WH												
			11	SS	WH												

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Continued Next Page

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

PROJECT <u>20253807</u>		RECORD OF BOREHOLE No. 21-06				2 OF 2 METRIC	
G.W.P. <u>5032-19-00</u>		LOCATION <u>N 5137676.9; E 278261.9 NAD83 MTM ZONE 12 (LAT. 46.378282; LONG. -81.344986)</u>				ORIGINATED BY <u>AD</u>	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>				COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 3, 2021</u>				CHECKED BY <u>MT</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20	40	60	80	100	20	40	60		
	--- CONTINUED FROM PREVIOUS PAGE ---																
	SILTY CLAY (CI) Firm Grey Wet		12	SS	WH	▽	231										
							230										
			13	SS	WH		229										
228.2																	
15.3	SILT (ML), trace clay, trace sand Very loose Grey Wet		14	SS	2		228										
227.5																	
15.9	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 12.4 m below ground surface (Elev. 231.0 m) in augers upon completion of drilling.																

SUD-MTO 001 R:\SUDBURY\SIM\CLIENTS\IMTO\HWY17_MR_5502_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

R:\SUSBURY\SIM\CLIENTS\IMTO\HWY17 MR 55102 DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

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

PROJECT <u>20253807</u>			RECORD OF BOREHOLE No. 21-07			2 OF 2 METRIC		
G.W.P. <u>5032-19-00</u>			LOCATION <u>N 5137704.9; E 278285.0 NAD83 MTM ZONE 12 (LAT. 46.378534; LONG. -81.344687)</u>			ORIGINATED BY <u>TB/NP</u>		
DIST <u> </u> HWY <u>17</u>			BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>			COMPILED BY <u>TR</u>		
DATUM <u>GEODETIC</u>			DATE <u>February 2, 2021</u>			CHECKED BY <u>MT</u>		

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
								20	40	60	80	100					
231.2 12.1	SILT (ML), trace clay, trace sand Very loose to loose Grey Wet	X	12	SS	4												
			13	SS	1												
228.5 14.8	SAND (SP), some silt Very loose Grey Wet	X	14	SS	3												
227.4 15.9	END OF BOREHOLE NOTE: 1. Water level measured at a depth of 6.8 m below ground surface (Elev. 236.5 m) in augers upon completion of drilling.																

SUD-MTO 001 R:\SUDBURY\SIM\CLIENTS\IMTO\HWY17_MR_5502_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

PROJECT <u>20253807</u>		RECORD OF BOREHOLE No. 21-08		1 OF 1 METRIC	
G.W.P. <u>5032-19-00</u>		LOCATION <u>N 5137709.2; E 278297.9 NAD83 MTM ZONE 12 (LAT. 46.378574; LONG. -81.344521)</u>		ORIGINATED BY <u>TB</u>	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>Portable Equipment, NW Casing with Wash Boring</u>		COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 8, 2021</u>		CHECKED BY <u>MT</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED				W _p	W	W _L							
241.0	TOP OF SNOW																				
0.0	SNOW / ICE																				
240.7																					
0.3	SAND (SP) and gravel, trace silt (FILL) Very loose		1	SS	2																
	- No recovery in Sample No. 1.																				
240.0																					
1.0	CLAYEY SILT (CL) Stiff Grey Moist to wet		2	SS	2																
			3	SS	5																
			4	SS	1																
	- Vane could not be advanced at 4.0 m depth.		5	SS	WH																
236.0																					
5.0	SILTY CLAY (CI) Firm to stiff Grey Wet																				
			6	TO	PH																
	- Varves of clayey silt encountered below 7.2 m depth.		7	SS	2																
			8A	TO	PH																
			8B																		
			9	SS	WH																
230.1																					
10.9	END OF BOREHOLE																				
	NOTES: 1. Water level measured at a depth of 2.8 m below ground surface (Elev. 238.2 m) in casing upon completion of drilling.																				

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

SUD-MTO 001 R:\SUDBURY\SIM\CLIENTS\MT\Hwy17_MR_5502_DATA\GINT\20253807\20253807.GPJ GAL-MISS GDT 7/16/21 TR

PROJECT <u>20253807</u>		RECORD OF BOREHOLE No. 21-09/21-09A		1 OF 2 METRIC	
G.W.P. <u>5032-19-00</u>		LOCATION <u>N 5137748.0; E 278319.6 NAD83 MTM ZONE 12 (LAT. 46.378923; LONG. -81.344241)</u>		ORIGINATED BY <u>TB/NP</u>	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>		COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 3 & 4, 2021</u>		CHECKED BY <u>MT</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	W _p			W	W _L	
243.3	GROUND SURFACE																
0.0	Gravelly SAND (SP/GP) to SAND (SP), some gravel, some silt (FILL) Dense to very dense Brown Moist - Auger grinding from ground surface to 2.1 m depth (potentially frozen).		1	AS	-												
			2	SS	46												
			3	SS	75												
			4	SS	30												
240.3																	
3.0	CLAYEY SILT (CL) Soft Grey Wet		5	SS	4												
239.6	SILTY CLAY (CI) Soft to firm Grey Wet		6	TO	PH												
3.7			7	SS	WH												
			8	TO	PH												
236.1																	
7.2	SILT (ML), trace sand, trace clay Loose Grey Wet		9	SS	5												
				10	SS	5											
233.1																	
10.2	SILTY SAND (SM) Loose to compact Grey to brown Wet - Heaving sand encountered from 10.2 m to 12.3 m depth.		11	SS	4												

Continued Next Page

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

SUD-MTO 001 R:\SUDBURY\CLIENTS\IMTO\HWY17_MR_5502_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

PROJECT <u>20253807</u>		RECORD OF BOREHOLE No. 21-09/21-09A				2 OF 2 METRIC	
G.W.P. <u>5032-19-00</u>		LOCATION <u>N 5137748.0; E 278319.6 NAD83 MTM ZONE 12 (LAT. 46.378923; LONG. -81.344241)</u>				ORIGINATED BY <u>TB/NP</u>	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers</u>				COMPILED BY <u>TR</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 3 & 4, 2021</u>				CHECKED BY <u>MT</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
								20	40	60	80	100					
	--- CONTINUED FROM PREVIOUS PAGE ---																
230.0	SILTY SAND (SM) Loose to compact Grey to brown Wet		12	SS	28		231										
13.3	Gravelly SAND (SP) Dense to very dense Grey to brown Wet		13	SS	47		230										
							229										
							228										
227.4	- Heaving sand encountered in Sample No. 14.		14	SS	72												
15.9	END OF BOREHOLE																
	NOTES: 1. Vanes obtained in a separate borehole (21-09A) advanced 3.0 m north of Borehole 21-09. 2. Water level measured at a depth of 3.3 m below ground surface (Elev. 240.0 m) upon completion of drilling. 3. Water level in piezometer measured below ground surface as follows: Date Depth (m) Feb 9, 2021 0.4 4. Water was frozen in piezometer at a depth of 0.5 m (Elev. 242.8 m) below ground surface on February 10, 2021.																

SUD-MTO 001 R:\SUDBURY\CLIENTS\IMTO\HWY17_MR_5502_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

PROJECT 20253807			RECORD OF BOREHOLE No. 21-10			1 OF 1 METRIC																
G.W.P. 5032-19-00			LOCATION N 5137780.3; E 278354.1 NAD83 MTM ZONE 12 (LAT. 46.379215; LONG. -81.343793)			ORIGINATED BY TB/NP																
DIST _____ HWY 17			BOREHOLE TYPE Portable Equipment, NW Casing with Wash Boring			COMPILED BY TR																
DATUM GEODETIC			DATE February 9, 2021			CHECKED BY MT																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p — W — W _L			γ			GR SA SI CL			
241.2	TOP OF SNOW							20 40 60 80 100														
0.0	SNOW / ICE						241															
240.7																						
0.6	PEAT (PT) (50 mm) Brown Wet		1	SS	2																	
	CLAYEY SILT (CL) Firm Grey Wet		2A	SS	1																	
	- Trace organics in Sample No. 1.		2B																			
	- Vane could not be advanced at 2.7 m depth.																					
			3	SS	4																	
237.8							238															
3.4	SILT (ML) to SILT (ML) and SAND (SP), trace clay Loose to compact Grey Wet		4	SS	8																	
			5	SS	6																	
			6	SS	4																	
			7	SS	11																	
	- No recovery in split-spoon No. 7.																					
234.6							236															
6.6	SAND (SP) to gravelly SAND (SP) Loose Grey Wet		8	SS	9																	
	- Refusal to practical casing advancement at 8.2 m depth.		9	SS	6																	
232.4							233															
8.8	END OF BOREHOLE																					
							232															
							231															
230.5	END OF DCPT																					
10.7	REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING) NOTE: 1. Water level measured at a depth of 1.9 m above ground surface (Elev. 242.6 m) inside casing upon completion of drilling. (i.e. 1.4 m above snow and ice).																					

SUD-MTO 001 R:\SUBBURY\SIM\CLIENTS\MT\HWY17_MR_5502_DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

PROJECT <u>11-1191-0007</u>		RECORD OF BOREHOLE No H3-25		2 OF 2 METRIC	
G.W.P. <u>156-98-00</u>		LOCATION <u>N 5137516.0; E 278139.2</u>		ORIGINATED BY <u>LK</u>	
DIST <u> </u> HWY <u>17</u>		BOREHOLE TYPE <u>108 mm I.D. Continuous Flight Hollow Stem Augers, NW Casing, Wash Boring</u>		COMPILED BY <u>EC</u>	
DATUM <u>Geodetic</u>		DATE <u>June 8 and 11, 2012</u>		CHECKED BY <u>SEMC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
--- CONTINUED FROM PREVIOUS PAGE ---								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%)								
								20	40	60	80	100	20	40	60						
224.4	SILTY CLAY to CLAY Firm to stiff Grey Wet		11	SS	WH																
224.4	CLAYEY SILT Firm Grey Wet		12	SS	WH																
17.8																					
222.8			13	SS	5																
19.4	SILT, trace to some sand, trace clay Loose to compact Grey Wet		14	SS	8																
			15	SS	12																
			16	SS	14																
218.7																					
23.5	END OF BOREHOLE																				
	Note: 1. Water level at a depth of 1.6 m below ground surface (Elev. 240.6 m) upon completion of drilling.																				

SUD_MTO 003 11-1191-0007.GPJ GAL-MISS.GDT 28/11/13 DATA INPUT:

DRAFT

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 2

W.P. 61-74-02/03

LOCATION CO-ORDS. 16,854,844N; 912,670E.

ORIGINATED BY MM

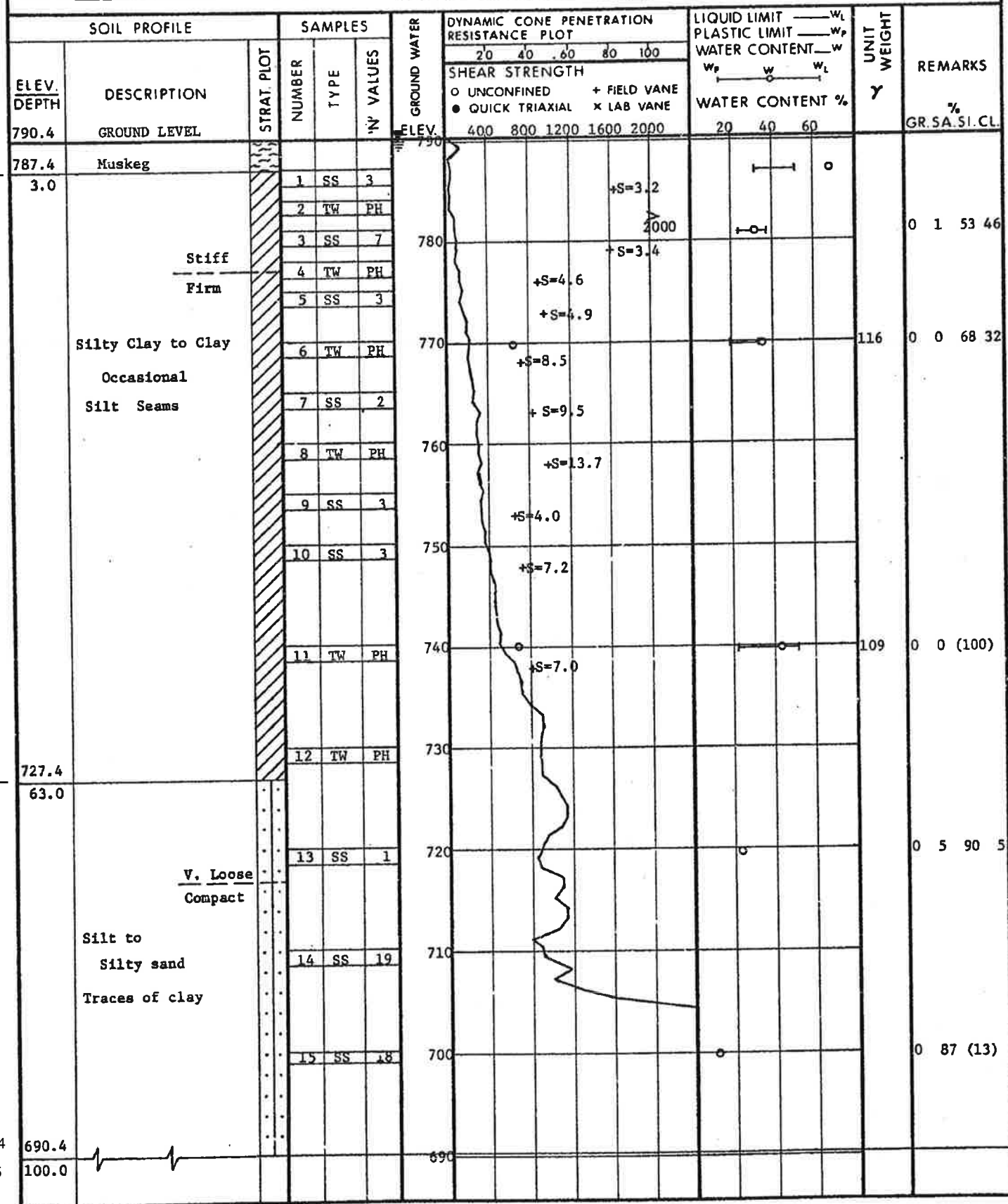
DIST. 17 HWY. 17, Line 'D' BORING DATE January 16, 1975

COMPILED BY MM

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGER AND CONE TEST

CHECKED BY



ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2 (Continued)

W.P. 61-74-02/03

LOCATION CO-ORDS. 16,854,844N; 912,670E.

ORIGINATED BY MM

DIST. 17 HWY. 17 Line 'D'

BORING DATE January 16th, 1975

COMPILED BY MM

DATUM GEODETIC

BOREHOLE TYPE HOLLOW STEM AUGER & CONE TEST

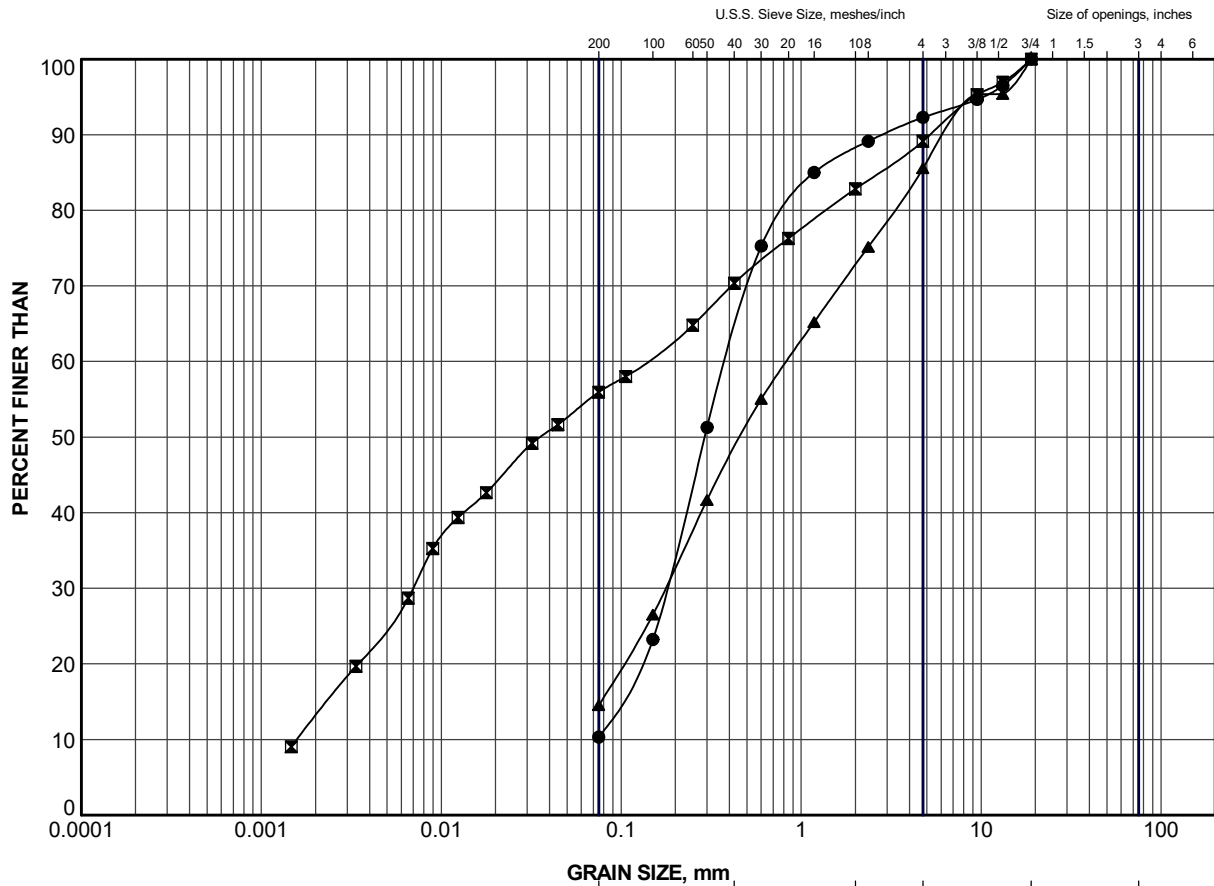
CHECKED BY _____

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT		LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w		UNIT WEIGHT γ	REMARKS			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100						w_p — w — w_L		
							SHEAR STRENGTH						WATER CONTENT %		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 400 800 1200 1600 2000						20 40 60		
690.4			16	SS	35	690									
100.0	Sand and gravel Traces of Silt Compact to dense		17	SS	28	680						51 42 (7)			
668.4			18	SS	47	670									
122.0	End of Borehole					660									
647.9						650									
142.5	End of Cone Penetration					640									

UPPIL KUPUKU UN DUL CAPUKAIK!

APPENDIX B


Laboratory Test Results

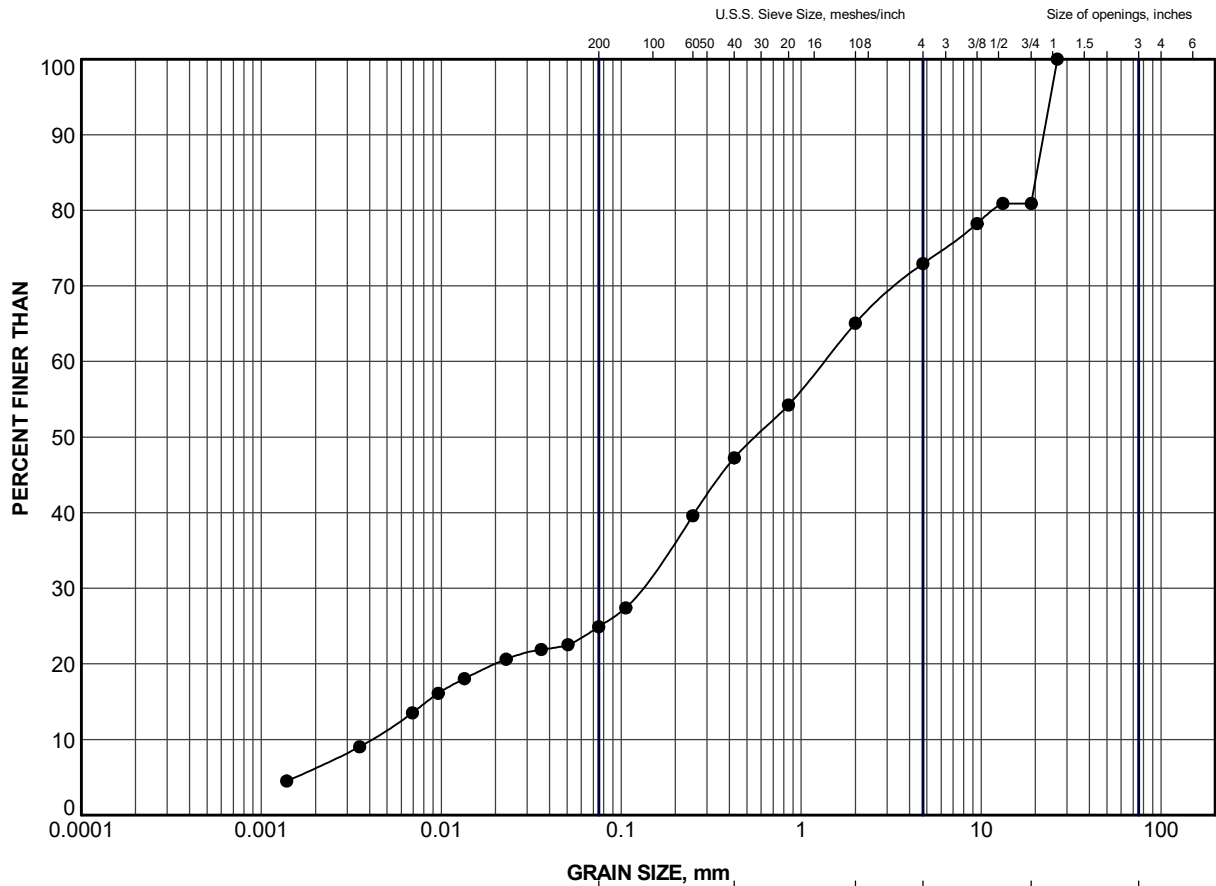


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-03	3	242.0
■	21-07	3	241.5
▲	21-09	4	240.7


PROJECT		HIGHWAY 17 EMBANKMENT WIDENING STA. 14+300 to STA. 14+725			
TITLE		GRAIN SIZE DISTRIBUTION Sandy SILT (ML) to SAND (SP) (FILL)			
 GOLDER MEMBER OF WSP SUDBURY, ONTARIO		PROJECT No. 20253807 DRAWN TR Jul 2021 CHECK TB Jul 2021 APPR MT Jul 2021		FILE No. 20253807.GPJ SCALE N/A REV.	
		FIGURE B-1			

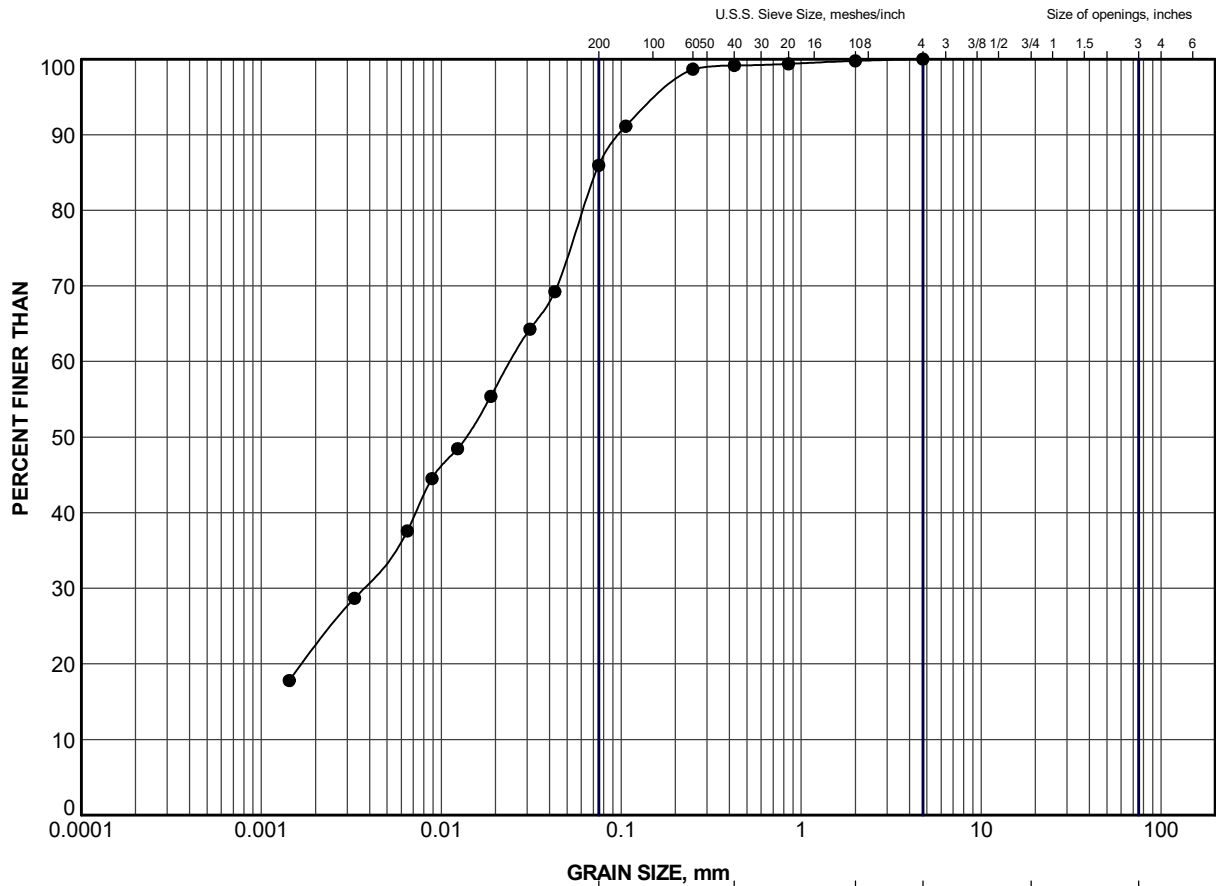


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-05	5	238.9


PROJECT						HIGHWAY 17 EMBANKMENT WIDENING STA. 14+300 to STA. 14+725					
TITLE						GRAIN SIZE DISTRIBUTION Gravelly SAND (SP)					
PROJECT No.			20253807			FILE No.			20253807.GPJ		
DRAWN	TR	Jul 2021	SCALE	N/A	REV.	FIGURE B-2					
CHECK	TB	Jul 2021									
APPR	MT	Jul 2021									
 GOLDER MEMBER OF WSP SUDBURY, ONTARIO											

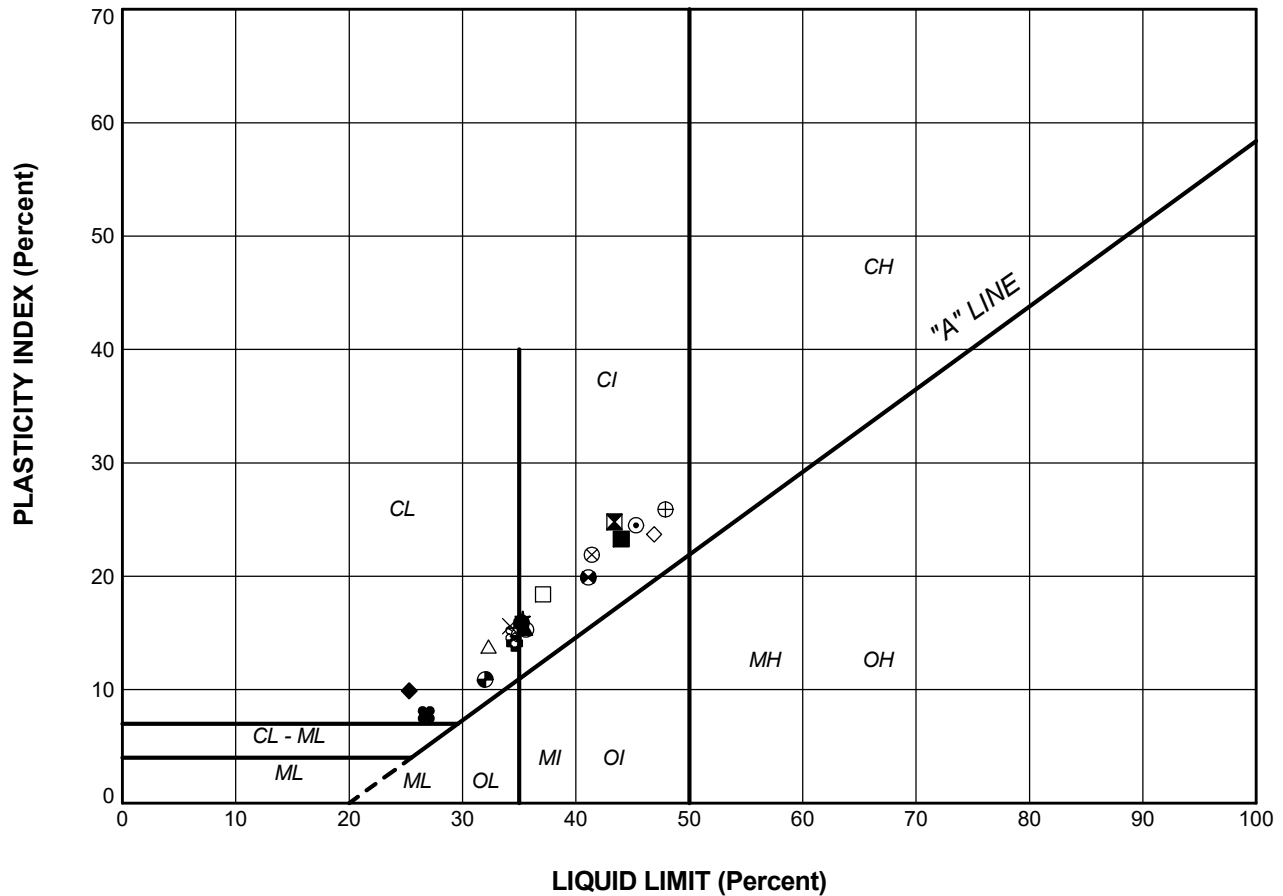


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-09	5	240.0

PROJECT						HIGHWAY 17 EMBANKMENT WIDENING STA. 14+300 to STA. 14+725					
TITLE						GRAIN SIZE DISTRIBUTION CLAYEY SILT (CL)					
PROJECT No.			20253807			FILE No.			20253807.GPJ		
DRAWN	TR	Jul 2021	SCALE	N/A	REV.	FIGURE B-3					
CHECK	TB	Jul 2021									
APPR	MT	Jul 2021									
 GOLDER MEMBER OF WSP SUDBURY, ONTARIO											



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	21-03	8	35.2	19.3	15.9
⊠	21-03	12	43.4	18.6	24.8
▲	21-04	8	35.5	20.1	15.4
★	21-04	10	35.3	19.0	16.3
⊙	21-04	14	45.3	20.8	24.5
⊕	21-05	8	34.6	20.5	14.1
○	21-05	10	35.6	20.3	15.3
△	21-06	8	32.3	18.5	13.8
⊗	21-06	10	41.4	19.5	21.9
⊕	21-06	12	47.9	22.0	25.9
□	21-07	7	37.1	18.7	18.4
⊗	21-07	9	41.1	21.2	19.9
⊕	21-07	11	32.0	21.1	10.9
★	21-08	3	34.6	19.3	15.3
⊗	21-08	5	34.5	19.6	14.9
■	21-08	7	44.0	20.7	23.3
◆	21-09	5	25.3	15.4	9.9
◇	21-09	7	46.9	23.2	23.7
×	21-10	2B	34.2	18.6	15.6
■	21-10	3	26.8	19.0	7.8

PROJECT			HIGHWAY 17 EMBANKMENT WIDENING STA. 14+300 to STA. 14+725		
TITLE			PLASTICITY CHART CLAYEY SILT (CL) to SILTY CLAY (CI)		
PROJECT No.		20253807		FILE No.	
DRAWN		TR		SCALE	
CHECK		TB		N/A	
APPR		MT		REV.	
				20253807.GPJ	
				JUL 2021	
				JUL 2021	
				JUL 2021	
				FIGURE B-4	



SUDBURY, ONTARIO

CONSOLIDATION TEST SUMMARY

FIGURE B-5

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	20253807	Sample Number	8A
Borehole Number	21-8	Sample Depth, m	8.7

TEST CONDITIONS

Test Method	B	Load Duration, hr	24
Oedometer Number	1	Load Increment Ratio	1
Date Started	March 3, 2021		
Date Completed	March 17, 2021		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	17.22
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	11.21
Area, cm ²	31.61	Specific Gravity, measured	2.770
Volume, cm ³	80.03	Solids Height, cm	1.045
Water Content, %	53.54	Volume of Solids, cm ³	33.04
Wet Mass, g	140.50	Volume of Voids, cm ³	47.00
Dry Mass, g	91.51	Degree of Saturation, %	104.2

TEST COMPUTATIONS

Stress	End of Primary Deformation ¹	Specimen Height ²	End of Primary Void Ratio ³	Average Height	Time ¹	Coefficient of Consolidation	Modulus of Volume Compressibility	Hydraulic Conductivity ⁴	Total Work
σ_v'	ΔH_{EOP}	H_{EOI}	e_{EOP}	$(H_p + H_{EOI})/2$	t_{90}	c_v	m_v	k_v	w
kPa	mm	cm		cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m ³
0	0.00	2.532	1.423	2.532					0
9	0.02	2.528	1.421	2.530			6.82E-05		0
17	0.07	2.522	1.412	2.525			4.58E-04		0
34	0.06	2.511	1.408	2.517	240	5.59E-03	8.98E-05	4.92E-08	0
69	0.07	2.494	1.396	2.502	437	3.03E-03	1.48E-04	4.41E-08	0
137	0.18	2.454	1.369	2.474	86	1.50E-02	1.61E-04	2.37E-07	1
273	1.65	2.225	1.190	2.339	3840	3.02E-04	5.42E-04	1.60E-08	17
547	1.43	2.033	0.992	2.129	960	1.00E-03	2.99E-04	2.93E-08	54
1095	0.99	1.899	0.851	1.966	540	1.52E-03	1.06E-04	1.58E-08	112
547		1.906	0.823	1.902					
137		1.931	0.847	1.918					
34		1.964	0.879	1.947					
9		1.988	0.902	1.976					

Note:

¹ Root Time Method (Taylor, 1942).² Specimen height corrected for apparatus deformation and presented for end of increment.³ Void ratio for unloading (i.e. rebound) calculated for the end of increment.⁴ Hydraulic conductivity calculated using coefficient of consolidation based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

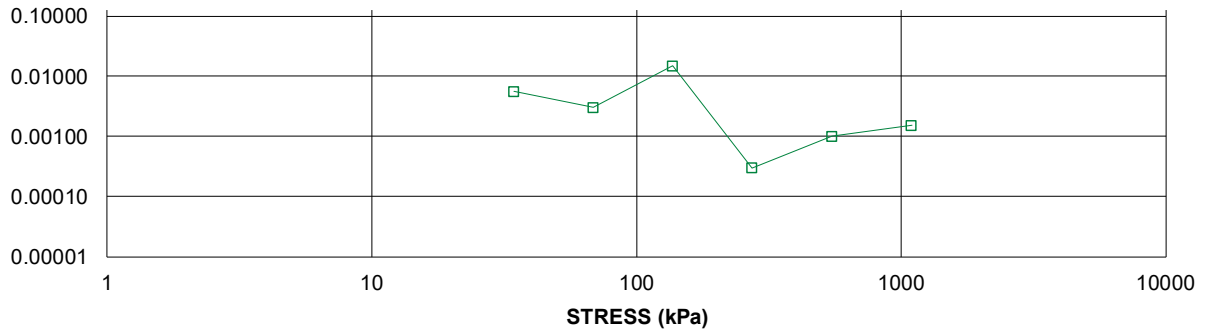
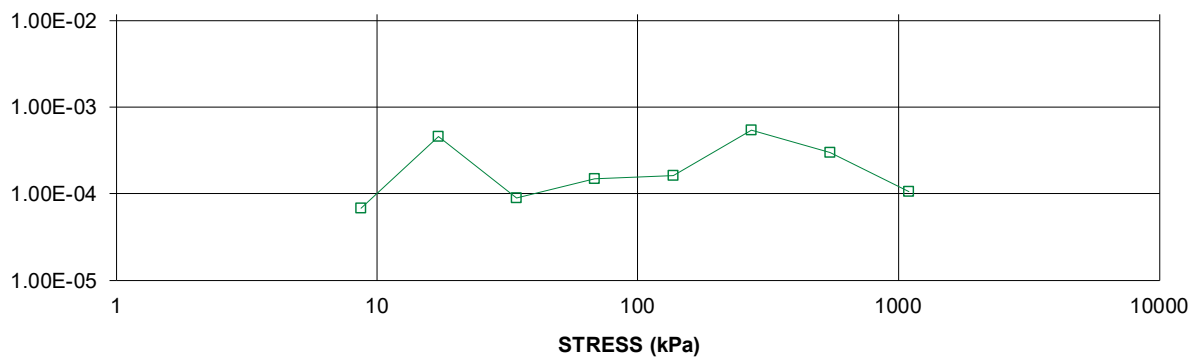
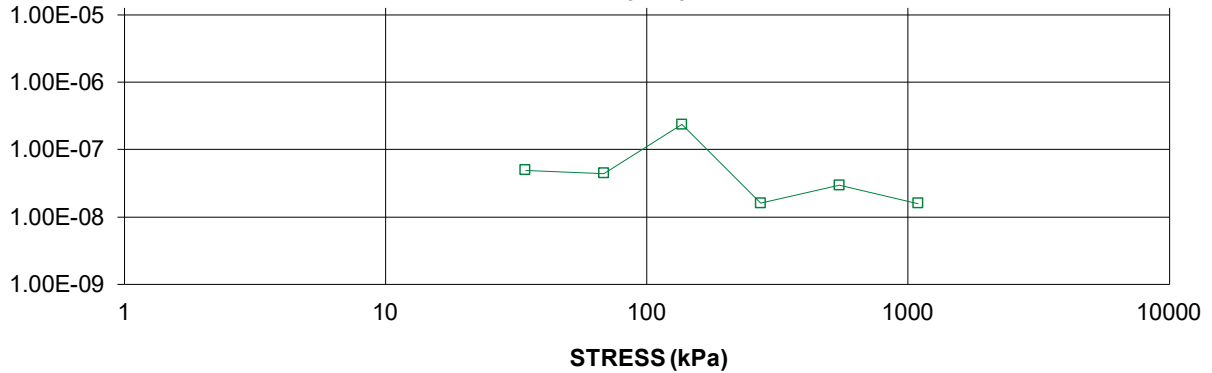
Sample Height, cm	1.99	Unit Weight, kN/m ³	19.46
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	14.28
Area, cm ²	31.61	Specific Gravity, measured	2.770
Volume, cm ³	62.83	Solids Height, cm	1.045
Water Content, %	36.24	Volume of Solids, cm ³	33.04
Wet Mass, g	124.67	Volume of Voids, cm ³	29.80
Dry Mass, g	91.51		

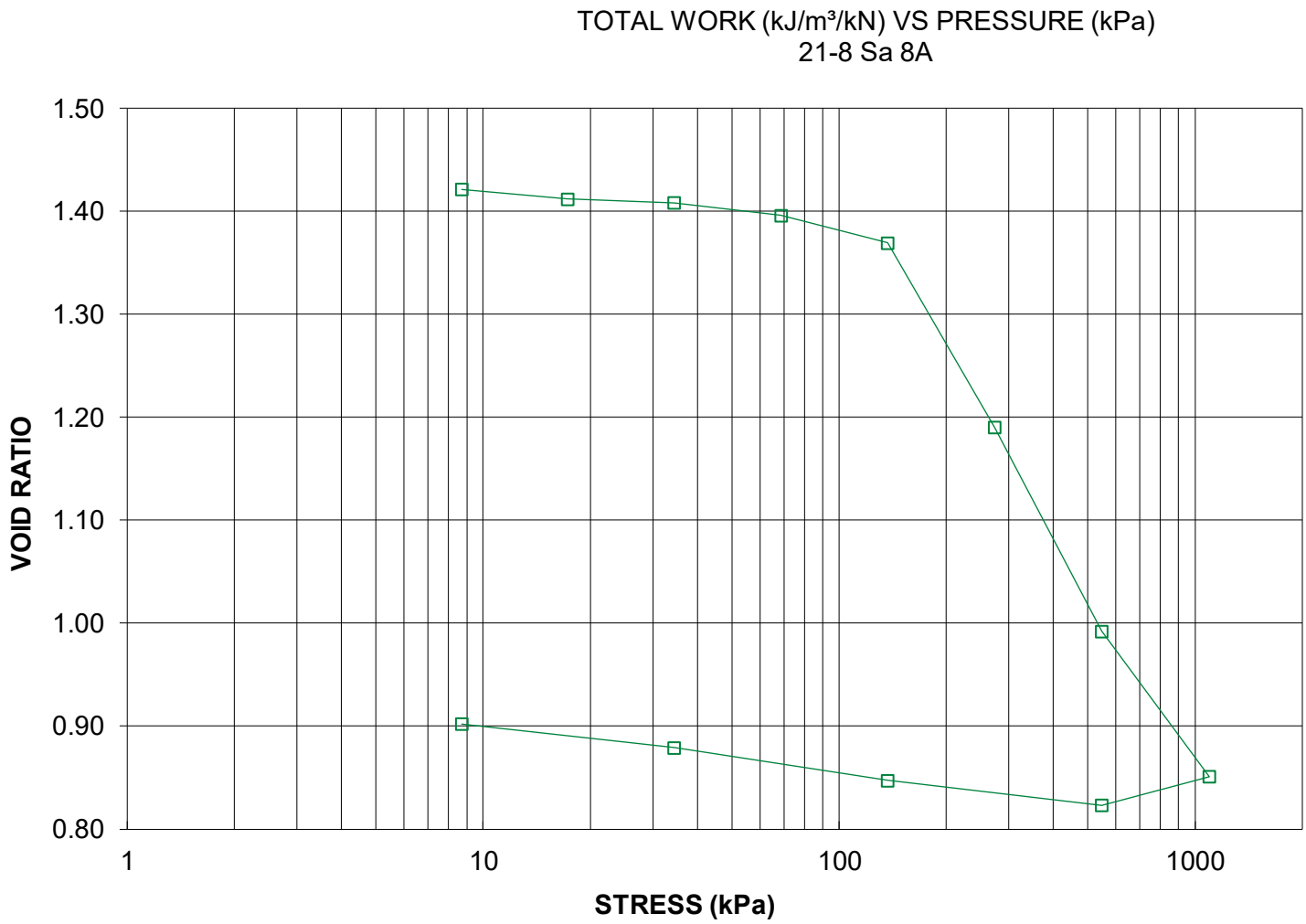


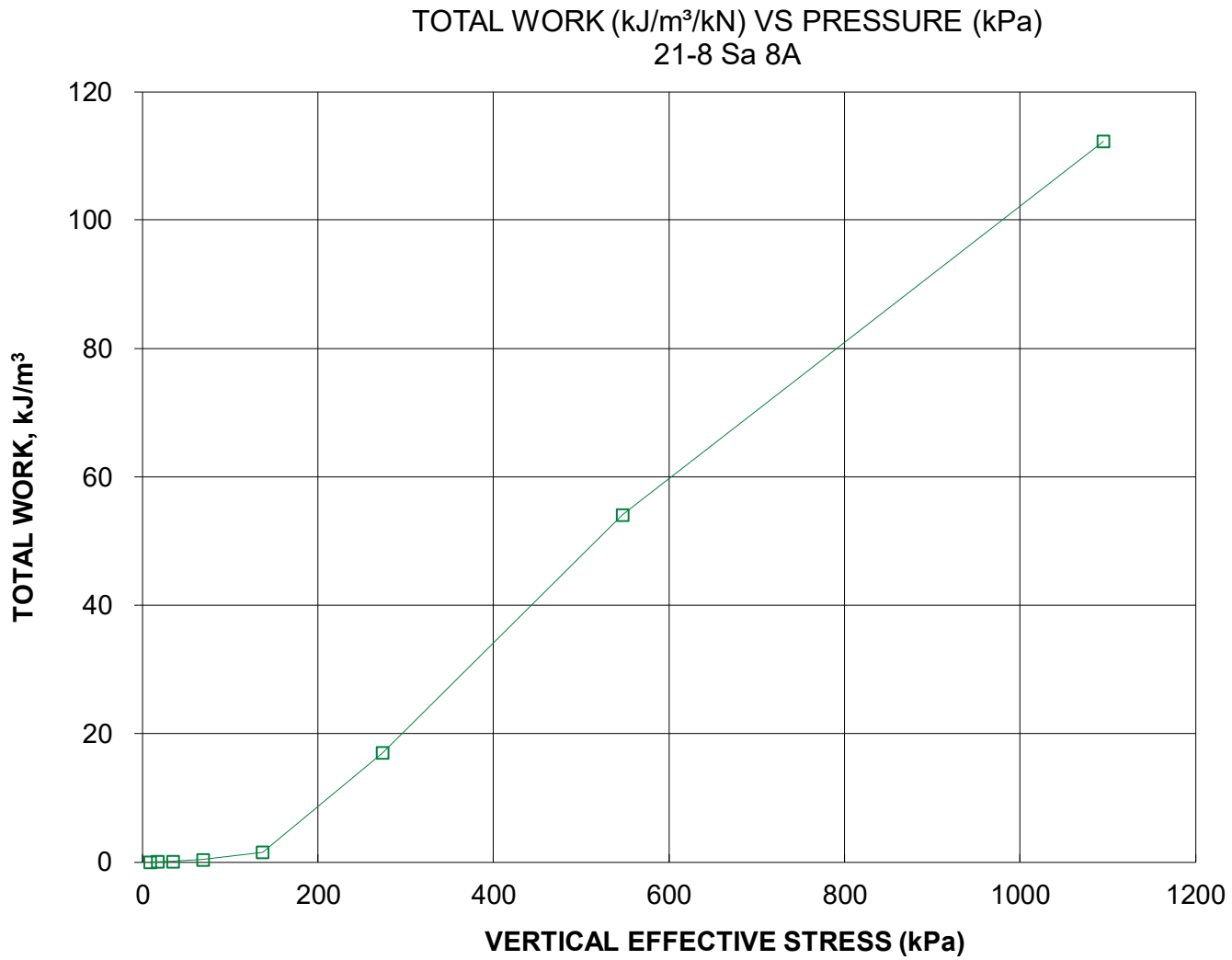
CONSOLIDATION TEST SUMMARY

FIGURE B-5

Pg. 2 of 4

COEFFICIENT OF CONSOLIDATION
(cm²/s)COEFFICIENT OF CONSOLIDATION (cm²/s) VS PRESSURE (kPa)
21-8 Sa 8AVOLUME COMPRESSIBILITY
(m²/kN)MODULUS OF VOLUME COMPRESSIBILITY (m²/kN) VS PRESSURE (kPa)
21-8 Sa 8AHYDRAULIC CONDUCTIVITY
(cm/s)HYDRAULIC CONDUCTIVITY (cm/s) vs PRESSURE (kPa)
21-8 Sa 8A





CONSOLIDATION TEST SUMMARY

FIGURE B-6

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	20253807	Sample Number	8B
Borehole Number	21-8	Sample Depth, m	9.0

TEST CONDITIONS

Test Method	B	Load Duration, hr	24
Oedometer Number	2	Load Increment Ratio	1
Date Started	March 3, 2021		
Date Completed	March 17, 2021		

Vertical Trimmed

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	17.12
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	11.09
Area, cm ²	31.66	Specific Gravity, measured	2.770
Volume, cm ³	80.10	Solids Height, cm	1.033
Water Content, %	54.42	Volume of Solids, cm ³	32.70
Wet Mass, g	139.86	Volume of Voids, cm ³	47.40
Dry Mass, g	90.57	Degree of Saturation, %	104.0

TEST COMPUTATIONS

Stress	End of Primary Deformation ¹	Specimen Height ²	End of Primary Void Ratio ³	Average Height	Time ¹	Coefficient of Consolidation	Modulus of Volume Compressibility	Hydraulic Conductivity ⁴	Total Work
σ_v'	ΔH_{EOP}	H_{EOI}	e_{EOP}	$(H_p + H_{EOI})/2$	t_{90}	c_v	m_v	k_v	w
kPa	mm	cm		cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m ³
0	0.00	2.530	1.450	2.530					
9	0.01	2.527	1.448	2.528			6.00E-05		0
17	0.14	2.514	1.433	2.520			7.61E-04		0
34	0.04	2.505	1.431	2.509	240	5.56E-03	4.91E-05	2.68E-08	0
69	0.11	2.483	1.415	2.494	135	9.77E-03	1.84E-04	1.76E-07	0
137	0.45	2.395	1.360	2.439	540	2.33E-03	3.29E-04	7.52E-08	3
273	1.10	2.210	1.212	2.302	577	1.95E-03	4.45E-04	8.48E-08	16
546	0.94	2.052	1.049	2.131	375	2.57E-03	2.44E-04	6.13E-08	46
1092	0.88	1.925	0.902	1.989	346	2.42E-03	1.10E-04	2.61E-08	105
546		1.928	0.867	1.927					
137		1.945	0.883	1.937					
34		1.966	0.904	1.956					
9		1.980	0.917	1.973					

Note:

¹ Root Time Method (Taylor, 1942).² Specimen height corrected for apparatus deformation and presented for end of increment.³ Void ratio for unloading (i.e. rebound) calculated for the end of increment.⁴ Hydraulic conductivity calculated using coefficient of consolidation based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

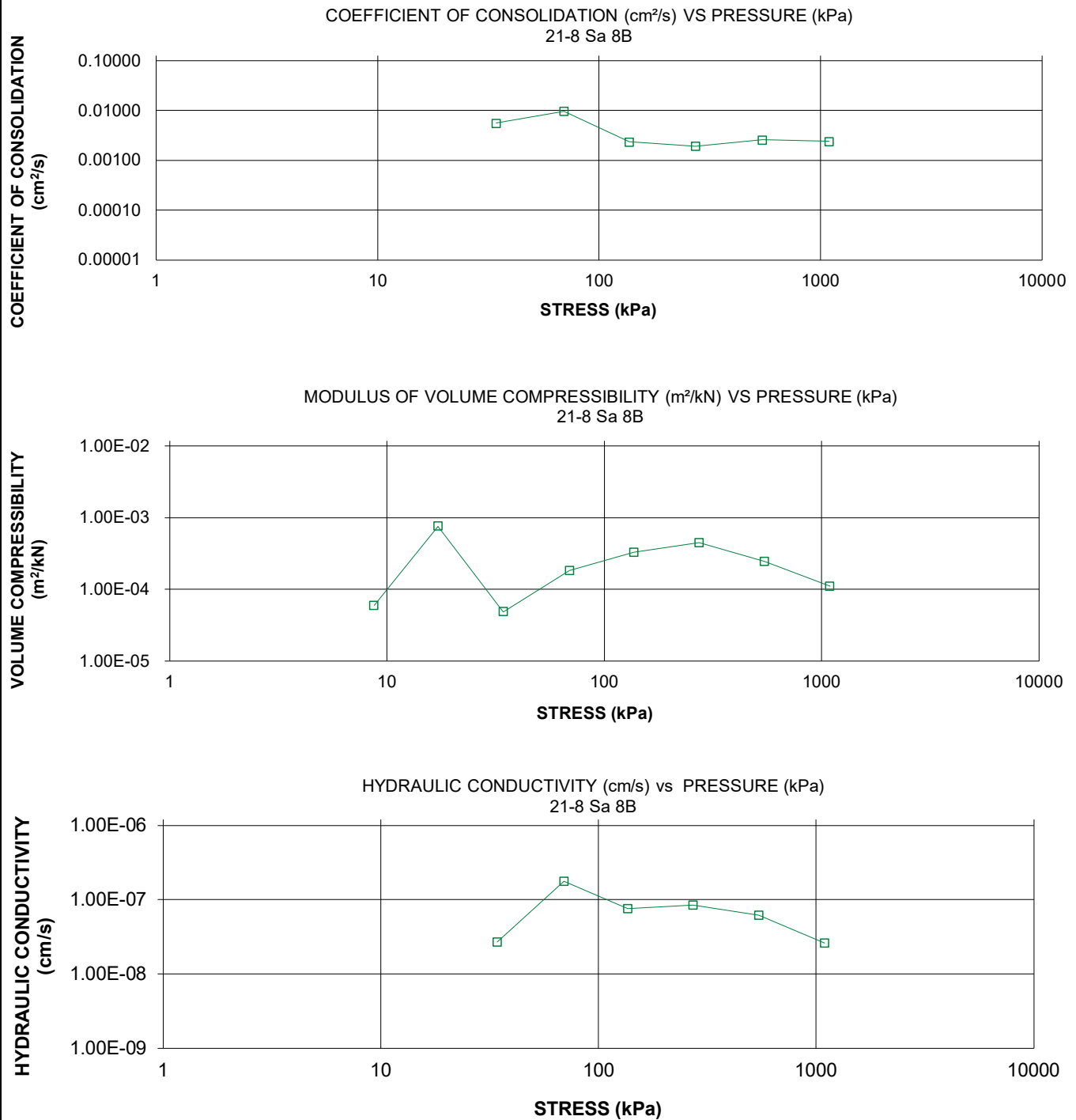
Sample Height, cm	1.98	Unit Weight, kN/m ³	19.38
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.17
Area, cm ²	31.66	Specific Gravity, measured	2.770
Volume, cm ³	62.67	Solids Height, cm	1.033
Water Content, %	36.77	Volume of Solids, cm ³	32.70
Wet Mass, g	123.87	Volume of Voids, cm ³	29.97
Dry Mass, g	90.57		

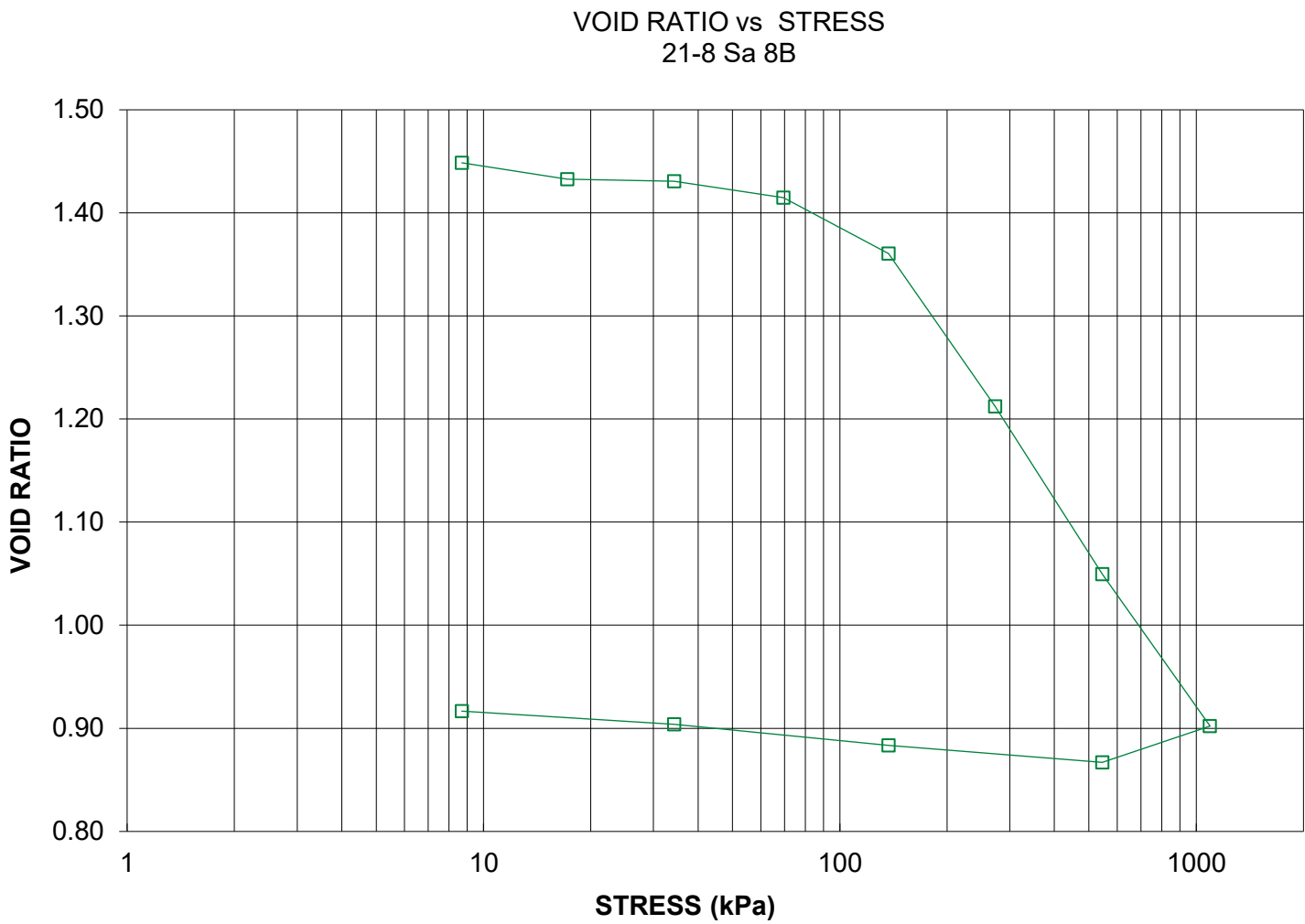


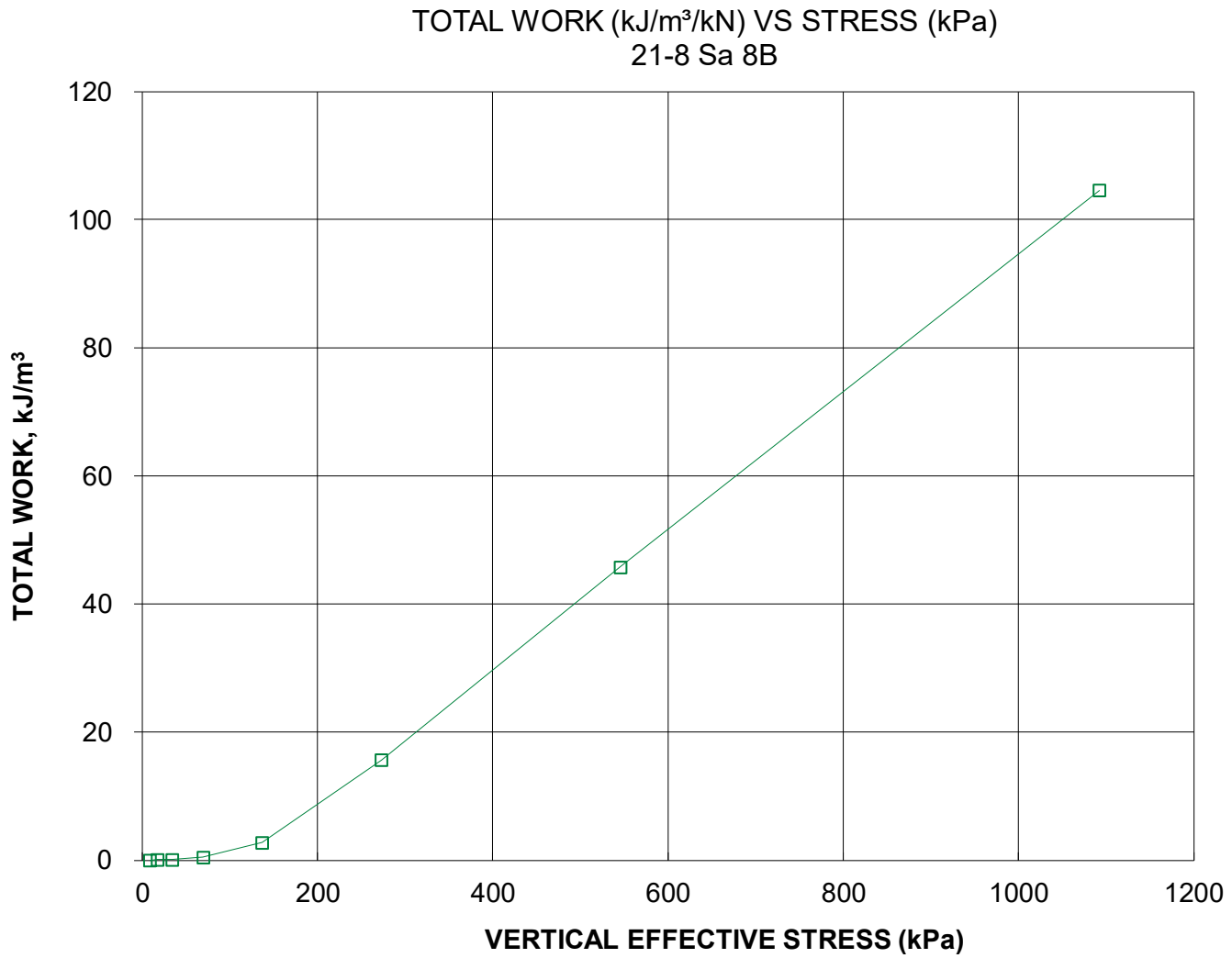
CONSOLIDATION TEST SUMMARY

FIGURE B-6

Pg. 2 of 4







CONSOLIDATION TEST SUMMARY

FIGURE B-7

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	20253807	Sample Number	14A
Borehole Number	21-4	Sample Depth, m	15.4

TEST CONDITIONS

Test Method	B	Load Duration, hr	24
Oedometer Number	1	Load Increment Ratio	1
Date Started	February 11, 2021		
Date Completed	February 25, 2021		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	17.03
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	11.39
Area, cm ²	31.61	Specific Gravity, measured	2.744
Volume, cm ³	80.03	Solids Height, cm	1.071
Water Content, %	49.53	Volume of Solids, cm ³	33.87
Wet Mass, g	138.96	Volume of Voids, cm ³	46.17
Dry Mass, g	92.93	Degree of Saturation, %	99.7

TEST COMPUTATIONS

Stress	End of Primary Deformation ¹	Specimen Height ²	End of Primary Void Ratio ³	Average Height	Time ¹	Coefficient of Consolidation	Modulus of Volume Compressibility	Hydraulic Conductivity ⁴	Total Work
σ_v'	ΔH_{EOP}	H_{EOI}	e_{EOP}	$(H_p + H_{EOI})/2$	t_{90}	c_v	m_v	k_v	w
kPa	mm	cm		cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m ³
0	0.00	2.532	1.363	2.532					
9	0.05	2.526	1.359	2.529	778	1.74E-03	2.15E-04	3.67E-08	0
17	0.04	2.519	1.354	2.523	577	2.34E-03	2.29E-04	5.25E-08	0
34	0.06	2.506	1.346	2.513	1500	8.92E-04	2.03E-04	1.78E-08	0
69	0.11	2.485	1.329	2.496	240	5.50E-03	2.15E-04	1.16E-07	1
137	0.21	2.447	1.300	2.466	406	3.18E-03	1.77E-04	5.51E-08	2
273	0.55	2.326	1.233	2.386	960	1.26E-03	2.08E-04	2.57E-08	8
547	1.71	2.129	1.011	2.227	1561	6.74E-04	3.42E-04	2.26E-08	48
1095	1.05	1.998	0.890	2.063	1382	6.53E-04	9.41E-05	6.02E-09	98
547		2.004	0.871	2.001					
137		2.028	0.892	2.016					
34		2.058	0.921	2.043					
9		2.084	0.945	2.071					

Note:

¹ Root Time Method (Taylor, 1942).² Specimen height corrected for apparatus deformation and presented for end of increment.³ Void ratio for unloading (i.e. rebound) calculated for the end of increment.⁴ Hydraulic conductivity calculated using coefficient of consolidation based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

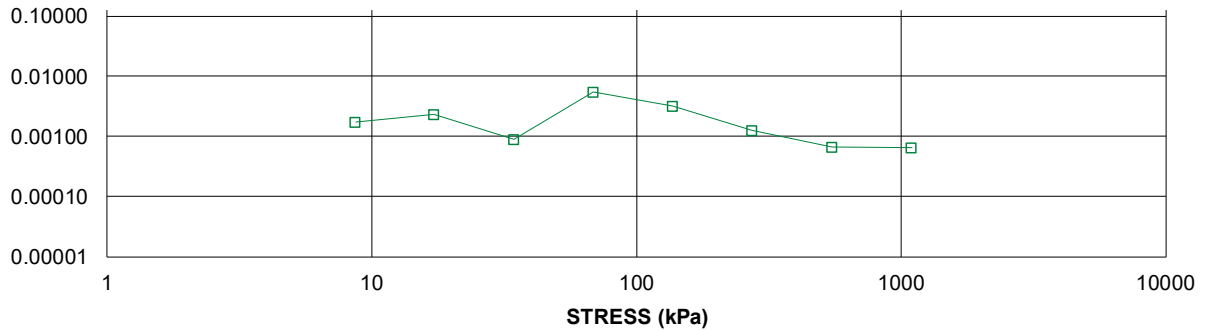
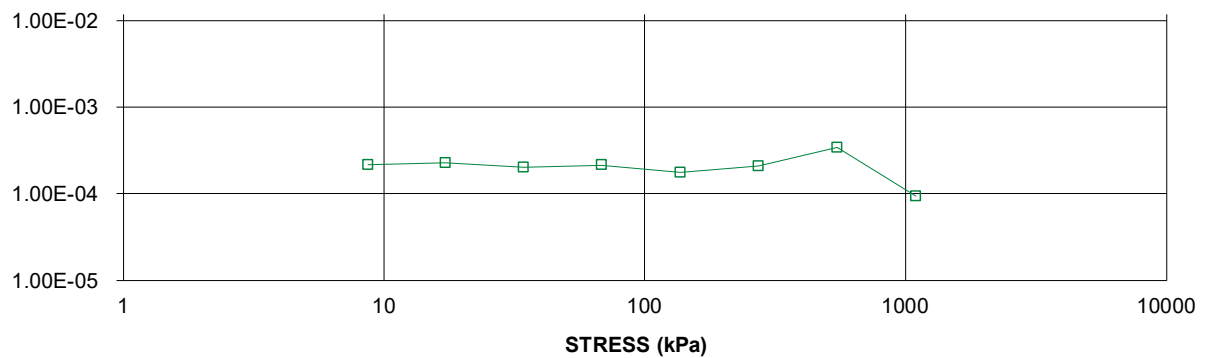
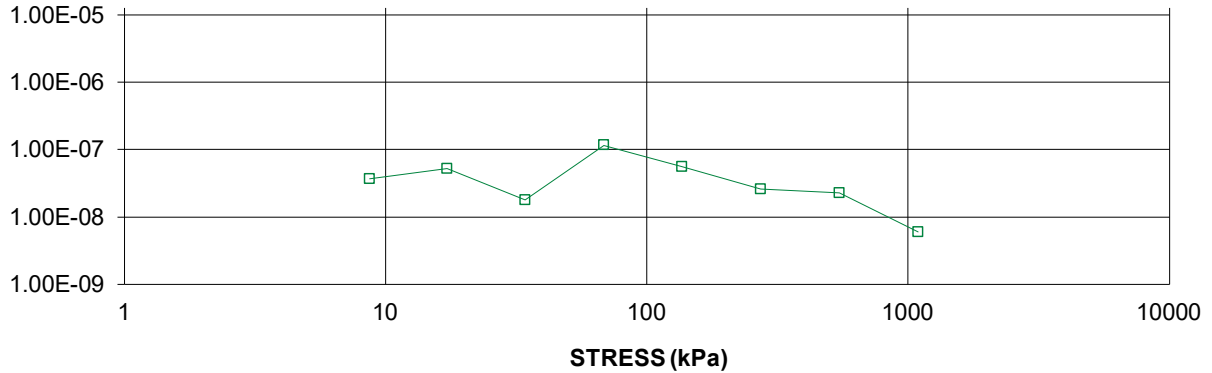
Sample Height, cm	2.08	Unit Weight, kN/m ³	18.70
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	13.84
Area, cm ²	31.61	Specific Gravity, measured	2.744
Volume, cm ³	65.87	Solids Height, cm	1.071
Water Content, %	35.17	Volume of Solids, cm ³	33.87
Wet Mass, g	125.61	Volume of Voids, cm ³	32.00
Dry Mass, g	92.93		

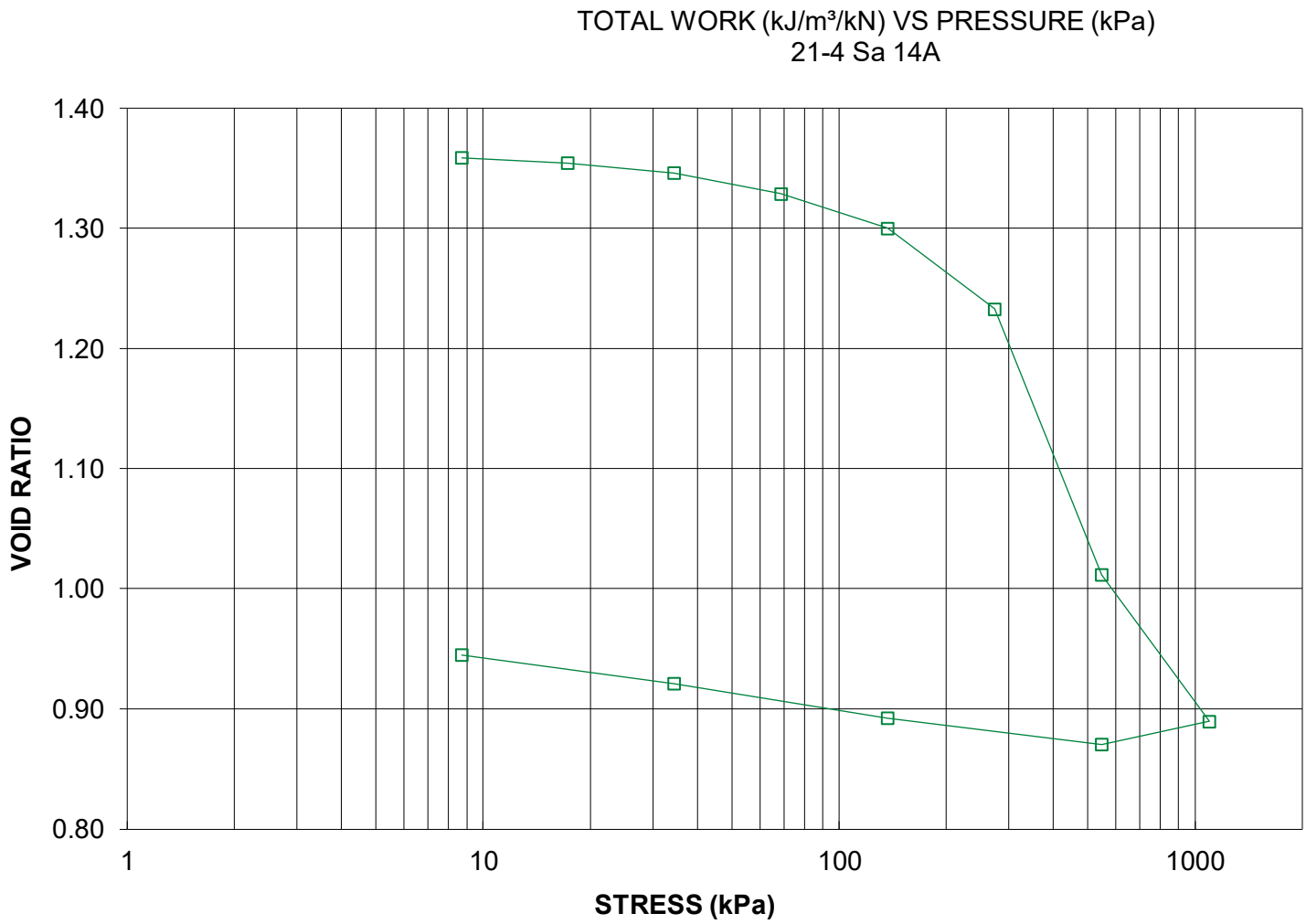


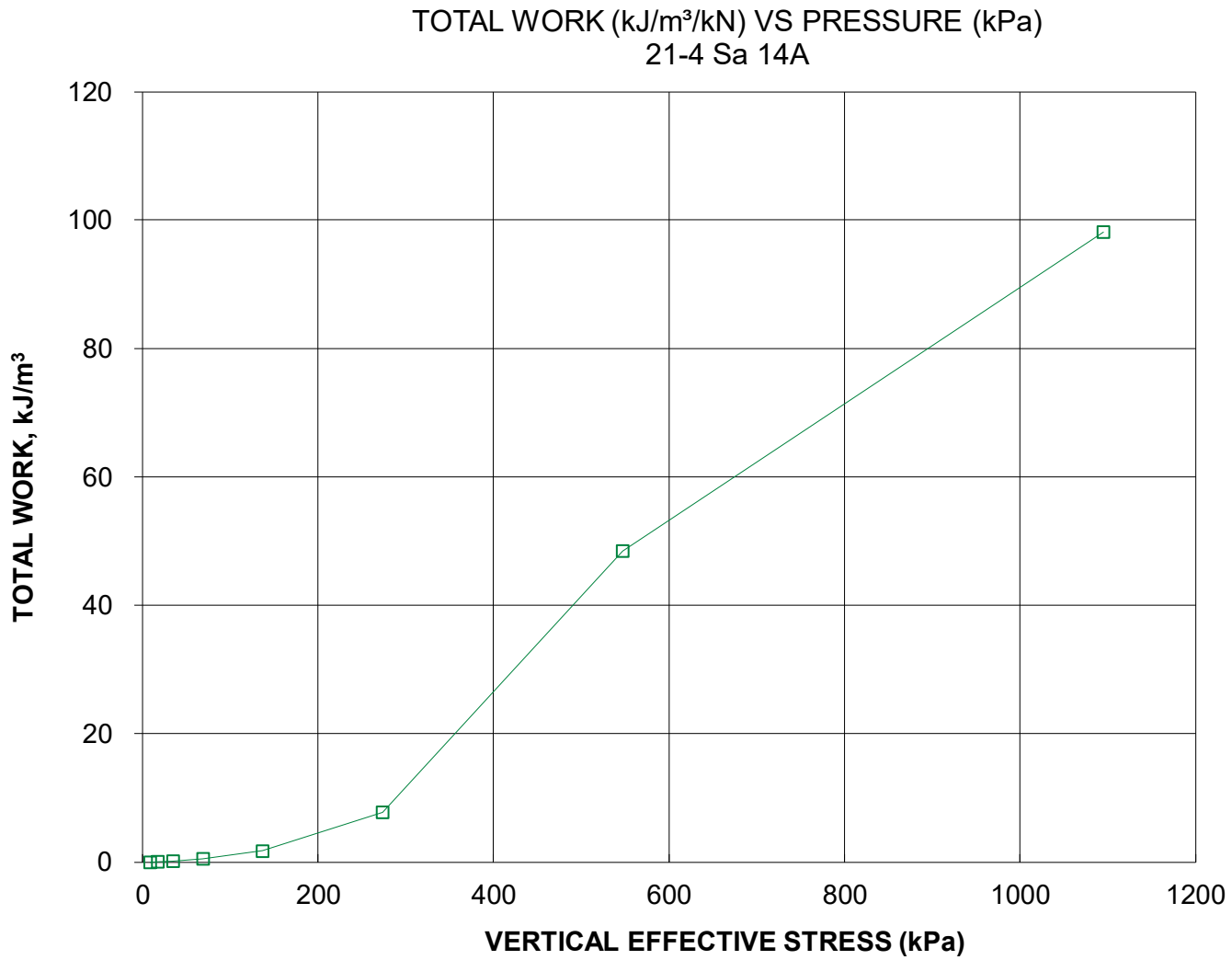
CONSOLIDATION TEST SUMMARY

FIGURE B-7

Pg. 2 of 4

COEFFICIENT OF CONSOLIDATION
(cm²/s)COEFFICIENT OF CONSOLIDATION (cm²/s) VS PRESSURE (kPa)
21-4 Sa 14AVOLUME COMPRESSIBILITY
(m²/kN)MODULUS OF VOLUME COMPRESSIBILITY (m²/kN) VS PRESSURE (kPa)
21-4 Sa 14AHYDRAULIC CONDUCTIVITY
(cm/s)HYDRAULIC CONDUCTIVITY (cm/s) vs PRESSURE (kPa)
21-4 Sa 14A





CONSOLIDATION TEST SUMMARY

FIGURE B-8

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	20253807	Sample Number	14B
Borehole Number	21-4	Sample Depth, m	15.5

TEST CONDITIONS

Test Method	B	Load Duration, hr	24
Oedometer Number	2	Load Increment Ratio	1
Date Started	February 11, 2021		
Date Completed	February 25, 2021		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.53	Unit Weight, kN/m ³	17.66
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	11.81
Area, cm ²	31.66	Specific Gravity, measured	2.744
Volume, cm ³	80.10	Solids Height, cm	1.111
Water Content, %	49.52	Volume of Solids, cm ³	35.16
Wet Mass, g	144.26	Volume of Voids, cm ³	44.94
Dry Mass, g	96.48	Degree of Saturation, %	106.3

TEST COMPUTATIONS

Stress	End of Primary Deformation ¹	Specimen Height ²	End of Primary Void Ratio ³	Average Height	Time ¹	Coefficient of Consolidation	Modulus of Volume Compressibility	Hydraulic Conductivity ⁴	Total Work
σ_v'	ΔH_{EOP}	H_{EOI}	e_{EOP}	$(H_p + H_{EOI})/2$	t_{90}	c_v	m_v	k_v	w
kPa	mm	cm		cm	sec	cm ² /s	m ² /kN	cm/s	kJ/m ³
0	0.00	2.530	1.278	2.530					
9	0.02	2.524	1.277	2.527	194	6.96E-03	7.50E-05	5.12E-08	0
17	0.03	2.514	1.269	2.519	346	3.89E-03	3.69E-04	1.41E-07	0
34	0.12	2.497	1.253	2.506	240	5.55E-03	4.19E-04	2.28E-07	0
69	0.12	2.469	1.238	2.483	217	6.04E-03	1.94E-04	1.15E-07	1
137	0.29	2.412	1.198	2.441	265	4.77E-03	2.60E-04	1.22E-07	2
273	0.53	2.275	1.124	2.344	2774	4.20E-04	2.36E-04	9.73E-09	9
546	0.86	2.121	0.971	2.198	913	1.12E-03	2.47E-04	2.71E-08	39
1092	1.05	1.988	0.816	2.055	540	1.66E-03	1.25E-04	2.03E-08	103
546		1.992	0.793	1.990					
137		2.009	0.809	2.000					
34		2.031	0.829	2.020					
9		2.048	0.844	2.040					

Note:

¹ Root Time Method (Taylor, 1942).² Specimen height corrected for apparatus deformation and presented for end of increment.³ Void ratio for unloading (i.e. rebound) calculated for the end of increment.⁴ Hydraulic conductivity calculated using coefficient of consolidation based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

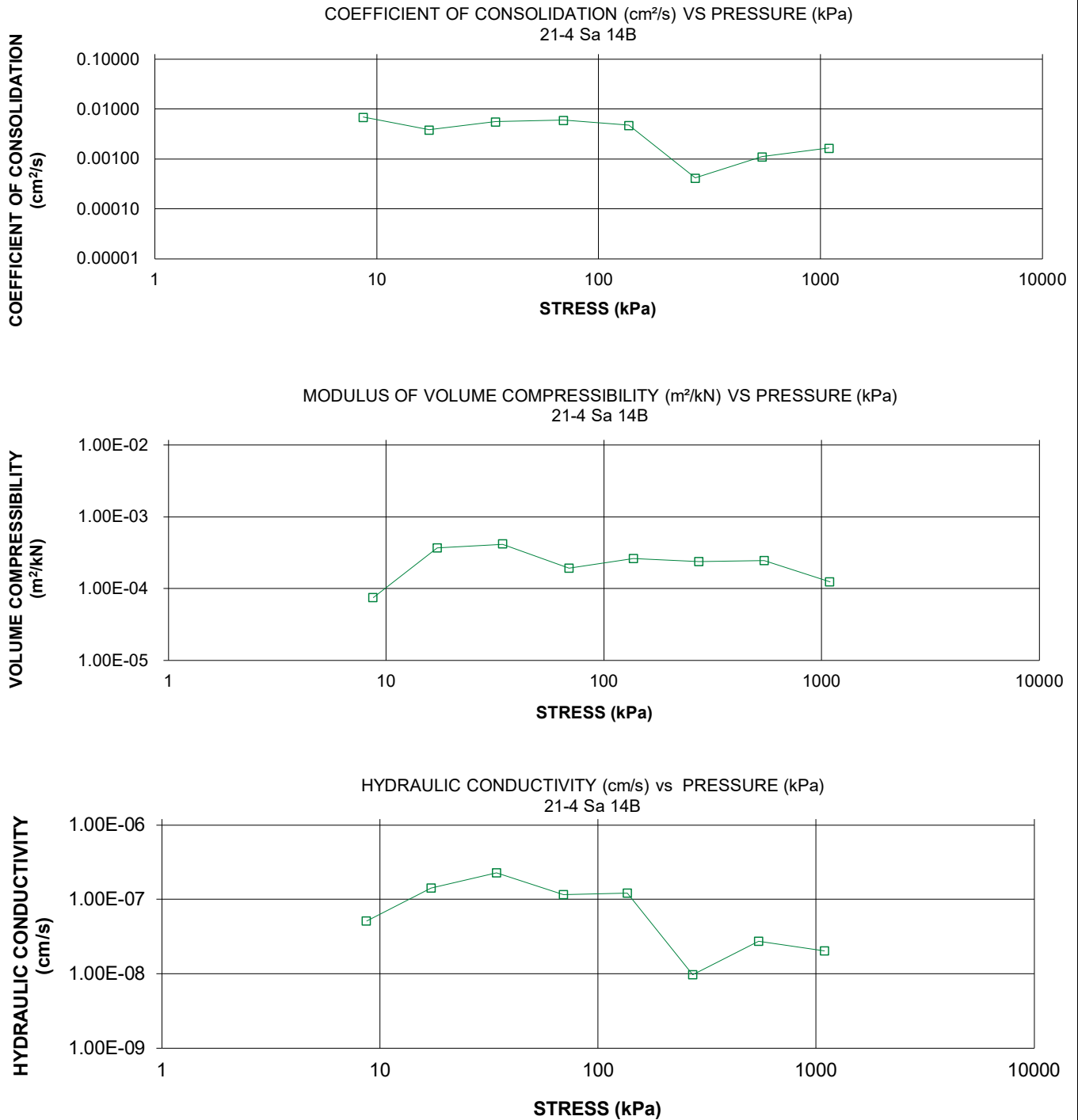
Sample Height, cm	2.05	Unit Weight, kN/m ³	19.74
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.59
Area, cm ²	31.66	Specific Gravity, measured	2.744
Volume, cm ³	64.85	Solids Height, cm	1.111
Water Content, %	35.29	Volume of Solids, cm ³	35.16
Wet Mass, g	130.53	Volume of Voids, cm ³	29.69
Dry Mass, g	96.48		

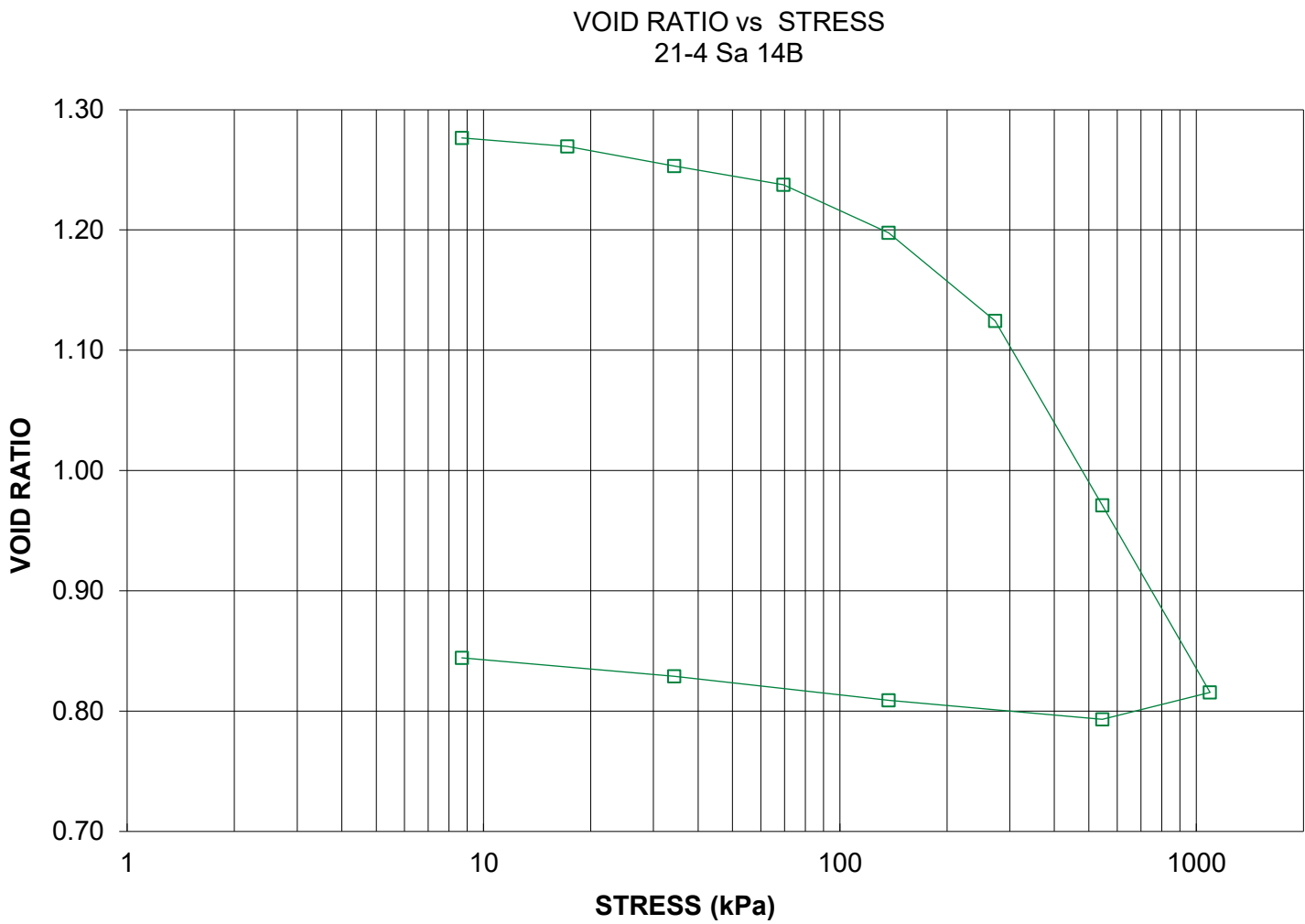


CONSOLIDATION TEST SUMMARY

FIGURE B-8

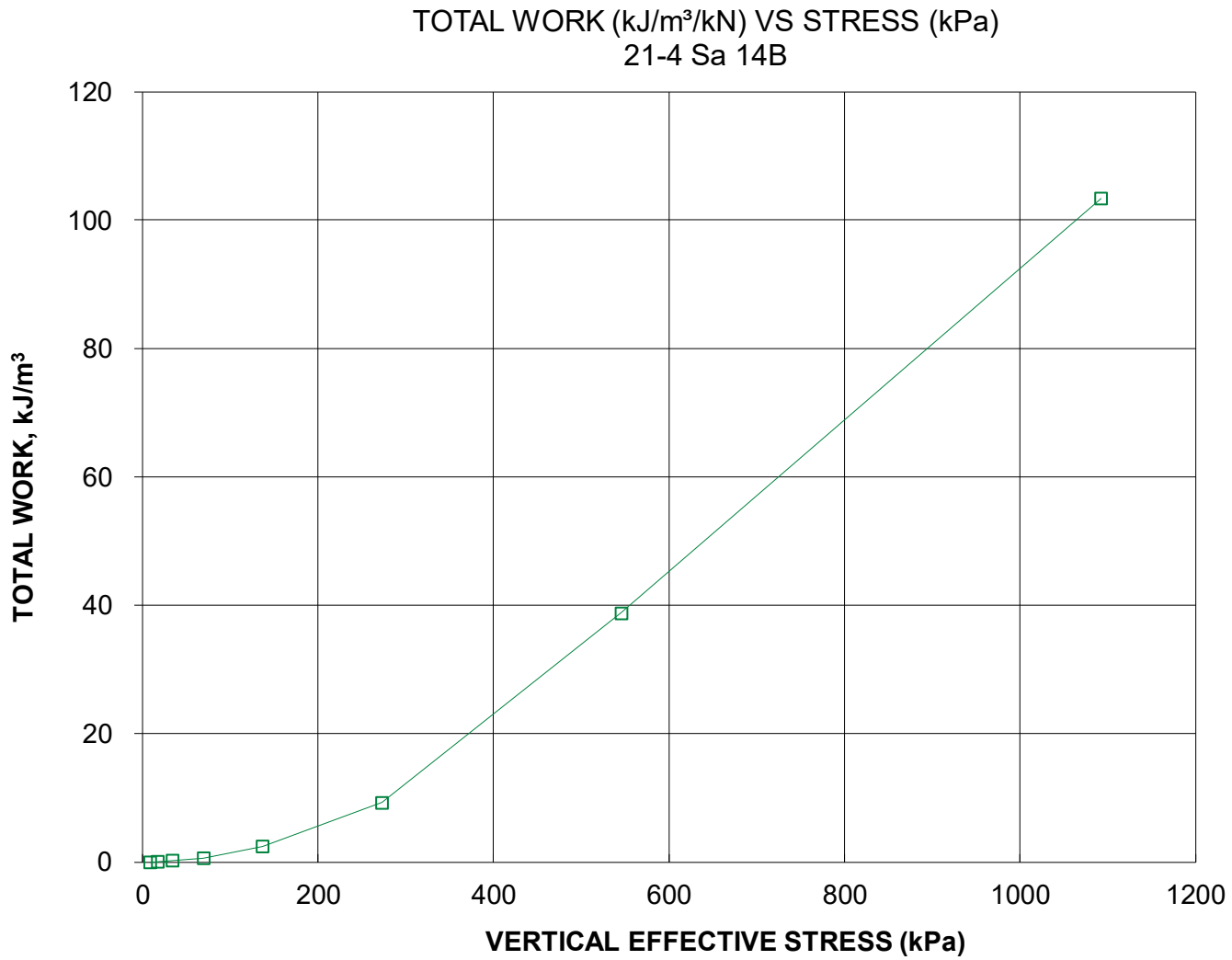
Pg. 2 of 4





CONSOLIDATION TEST
TOTAL WORK VS STRESS

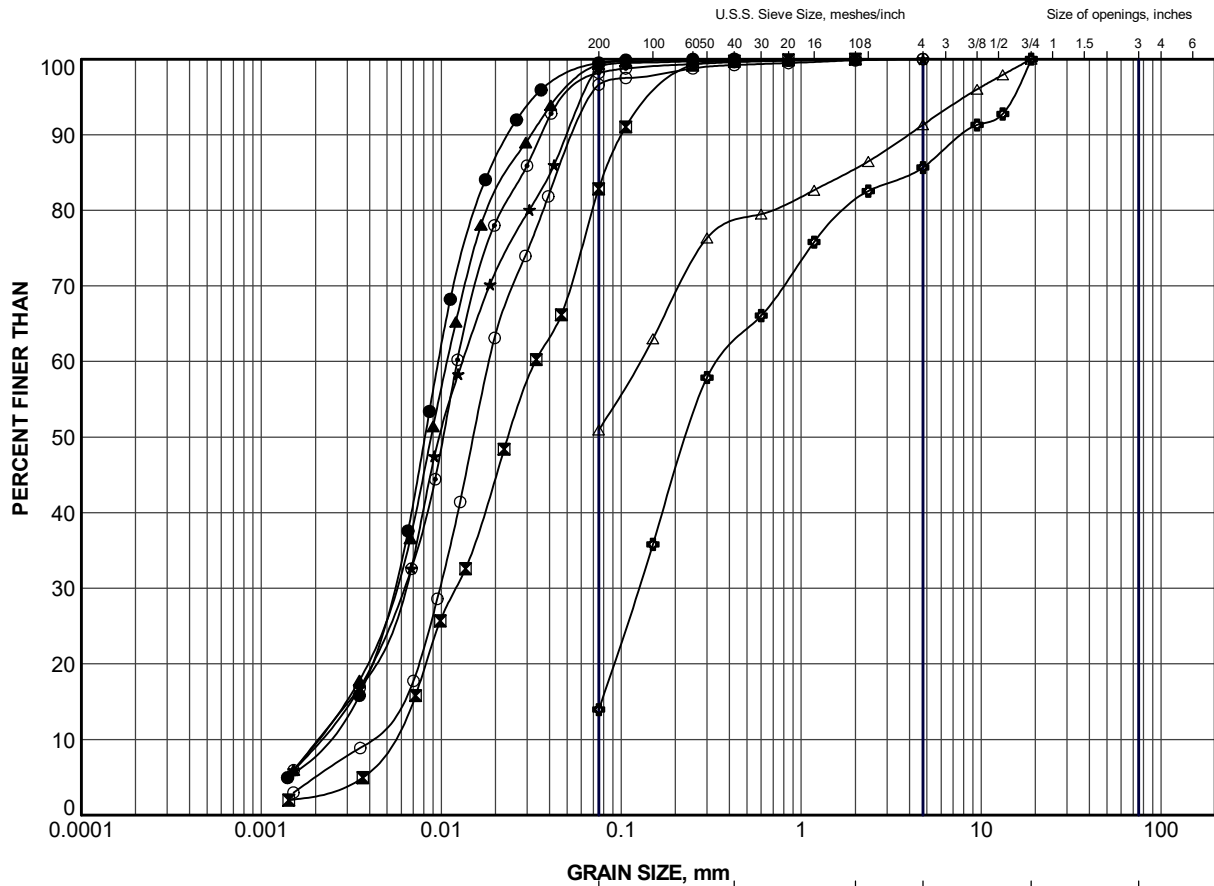
FIGURE B-8
Pg. 4 of 4



Project No. 20253807
Prepared By: TG



Checked By: MT



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	21-04	17	223.5
⊠	21-04	18	222.0
▲	21-06	14	227.9
★	21-07	12	230.8
⊙	21-09	9	235.4
⊕	21-09	12	230.8
○	21-10	5	236.6
△	21-10	6	236.1

PROJECT					HIGHWAY 17 EMBANKMENT WIDENING STA. 14+300 to STA. 14+725				
TITLE					GRAIN SIZE DISTRIBUTION SILT (ML) to SILTY SAND (SM)				
PROJECT No.		20253807		FILE No.		20253807.GPJ			
DRAWN	TR	Jul 2021		SCALE	N/A		REV.		
CHECK	TB	Jul 2021		FIGURE B-9					
APPR	MT	Jul 2021							
GOLDER MEMBER OF WSP SUDBURY, ONTARIO									

APPENDIX C

Non-standard Special Provision

Sub-Excavation of Organic Deposits – Item No.

Non-Standard Special Provision

This Non-Standard Special Provision outlines the procedure for sub-excavating organic deposits (i.e., peat, topsoil and organic silt) for the Highway 17 embankment widening from approximately Station 14+300 to 14+725 in Denison Township.

Staged excavations of limited extent shall be employed so as to maintain stability and protection of the existing Highway 17 embankment during sub-excavation and backfilling operations. The staged excavation procedures to be followed are:

- Removal of organic deposits within the proposed embankment widening footprint and backfilling of the excavation shall be carried out simultaneously in accordance with OPSS 209.
- Excavation shall be carried out in sections of no greater than 5 m wide parallel to the embankment toe.
- Provisions for traffic control measures shall be available on site to maintain the safe operation and traffic flow of Highway 17.

DRAFT



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