

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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20253807-01-Rev1

January 21, 2022

GEOCRES NO: **41I-374**

LAT: 46.377966

LONG: -81.345206



Distribution List

- 1 PDF Copy: Ministry of Transportation, Ontario (NE Region)
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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. There are two existing culverts located within the proposed widening at about Station 14+384 (structural box culvert referred to as Fairbanks Creek Culvert) and at Station 14+563 (non-structural circular culvert). 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 H3-25 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.

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.

4.2.2.1 Previous Investigations

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.

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%.

4.2.3.1.1 Previous Investigations

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.

4.2.4.1 Previous Investigations

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 (HTO)	232.3	53.5	17.2	70	150	2.1	1.4	0.66	0.046	0.006
21-08 / 8B (VTO)	232.0	54.4	17.1	70	N/A	N/A	N/A	N/A	N/A	N/A
21-04 / 14A (HTO)	228.2	49.5	17.0	154	238	1.5	1.4	0.74	0.041	0.0017
21-04 / 14B (VTO)	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)
HTO Horizontally trimmed orientation
VTO Vertically trimmed orientation

4.2.7.1 Previous Investigations

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.

4.2.8.1 Previous Investigations

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%.

4.2.9.1 Previous Investigations

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)
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 ²	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

² Piezometer screen installed between Elevations 231.2 m and 228.1 m

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.

Signature Page

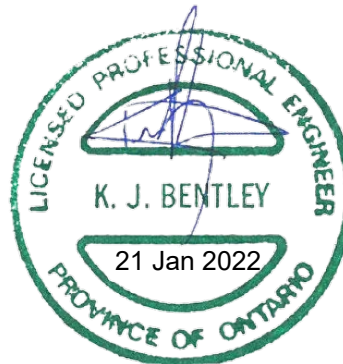
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PART B

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 (with the exceptions noted in the following sections). 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, with the exceptions noted in this report.

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/m³.

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:

$$S_{u(FV-uncorrected)} = \text{average mobilized undrained shear strength (kPa)}$$

$$\sigma_p' = \text{preconsolidation pressure (kPa)}$$

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.

A summary of the soil parameters selected for the stability analyses are provided in the tables on Figures 3 to 6.

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 as the design lines 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.
- As the wick drains are designed to rapidly dissipate excess pore water pressure by creating a preferential pathway for groundwater to transmit to surface, sites with artesian conditions pose an increased risk of the wick drain system modifying the groundwater regime in the area should the aquifer be connected to surface which could lead to uncontrolled seepage (artificial “springs”), additional time to achieve consolidation and/or additional ground settlements within the aquifer.

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 expedite 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 (up to 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

* Lightweight fill installed to a maximum of 2 m thick along the length of the Area/Zone. The actual lightweight fill thickness will vary, dependent on the total thickness of fill placement required above the design water level for the embankment widening.

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 of a portion of the granular fill with an up to 2 m thick cellular concrete layer 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 / 2*	<50	<200H:1V
MR 55 to Hwy 17 STA 14+350 (Proposed Ramp Shoulder)	3.5 / 2*	50	<200H:1V
Hwy 17 EBL STA 14+350 to 14+400 (Proposed Shoulder)	3.5 / 2*	<50	<200H:1V

*Lightweight fill installed after preload period. Lightweight fill installed to a maximum of 2 m thick along the length of the Area/Zone. The actual lightweight fill thickness will vary, dependent on the total thickness of fill placement required above the design water level for the embankment widening.

From a foundations perspective, this mitigation alternative is recommended to achieve the desired post-construction settlement performance. Golder has prepared an example specification, which is provided in Appendix C, to supply and install the cellular concrete for incorporation in the Contract documents.

6.5 Foundation Instrumentation and Monitoring

As the preferred mitigation measure incorporates a wait (preload) 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/nail pins (NPs) and settlement plates (SPs) on the existing culvert and below the widened embankments and taking regular measurements/readings at given intervals of time during and after construction of the earth fill preload embankments for the duration of the wait period. A settlement monitoring instrument location plan and details are provided on Drawings 3 and 4.

Specifications for the type, number, and layout of the settlement monitoring instruments, together with the supply, installation, protection, and estimated preload (wait) time should be included as Special Provisions in the Contract Documents. A NSSP titled “Supply and Installation of Embankment Monitoring Instruments” for the supply and installation of the embankment monitoring equipment for this site and an Operational Constraint titled “Embankment Preload” for the wait time is provided in Appendix C.

The details of the post instrumentation installation monitoring program, including the monitoring frequency of the instruments and interpretation of the data to confirm the preload (wait) time prior to replacement with cellular concrete is included in a separate work plan titled “Foundation Monitoring Program”, provided in Appendix D for incorporation into the Contract Administration contract for the Foundation Engineering Specialist.

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, organic silts / peat and anticipated native soils below the water table and may penetrate into the native firm to stiff silty clay to clayey silts. The granular fill and native soils within the anticipated excavation depths can be classified as Type 3 soil above the groundwater table and Type 4 soil below the groundwater table as per the OHSA. Temporary open-cut excavations in Type 3 and Type 4 soils can be sloped no steeper than 1H:1V and 3H:1V, respectively.

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 in this area. Based on discussions with the designer, specific design elevations for the excavation depths were developed for the contract documents between Station 14+400 to 14+500 with the understanding that some organic silt will be left in place in this area. The rationale for leaving some organic material in this area was due to the constructability concerns with the depth of excavation required to remove the material immediately adjacent to the existing embankment. The settlement of the organic silt to be left in place is anticipated to be minor, given that the organic silt layer is relatively thin, and has undergone sustained loading from the existing overlying fill that is understood to have been in place for at least 5 years.

After clearing and grubbing the embankment footprint area and prior to the placement of any fill for new construction, all organic deposits (with the exception of those identified above between Station 14+400 and 14+500) 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 (parallel to) the existing highway embankment should be carried out in strips of limited width equal to 3 m and immediately backfilled 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 of 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).
- Excavations and backfilling should occur simultaneously.
- 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.

In the settlement mitigation areas where the surcharge/preload fill will be replaced with lightweight fill, the lightweight fill (i.e., cellular concrete) should be placed in lifts and allowed sufficient time to cure prior to pouring subsequent lifts. Golder has prepared an example specification, which is provided in Appendix C, to supply and install the cellular concrete for incorporation in the Contract documents.

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.

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[https://golderassociates.sharepoint.com/sites/128666/Project Files/6 Deliverables/Foundations/Final/R1-Highway 17 Embankment Widening/Rev 1/2. Final Report/20253807-R01-Rev1-Hwy 17 Emb.Widening FIDR 21Jan_22.docx](https://golderassociates.sharepoint.com/sites/128666/Project%20Files/6%20Deliverables/Foundations/Final/R1-Highway%2017%20Embankment%20Widening/Rev%201/2.%20Final%20Report/20253807-R01-Rev1-Hwy%2017%20Emb.Widening%20FIDR%2021Jan_22.docx)

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- 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.

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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:

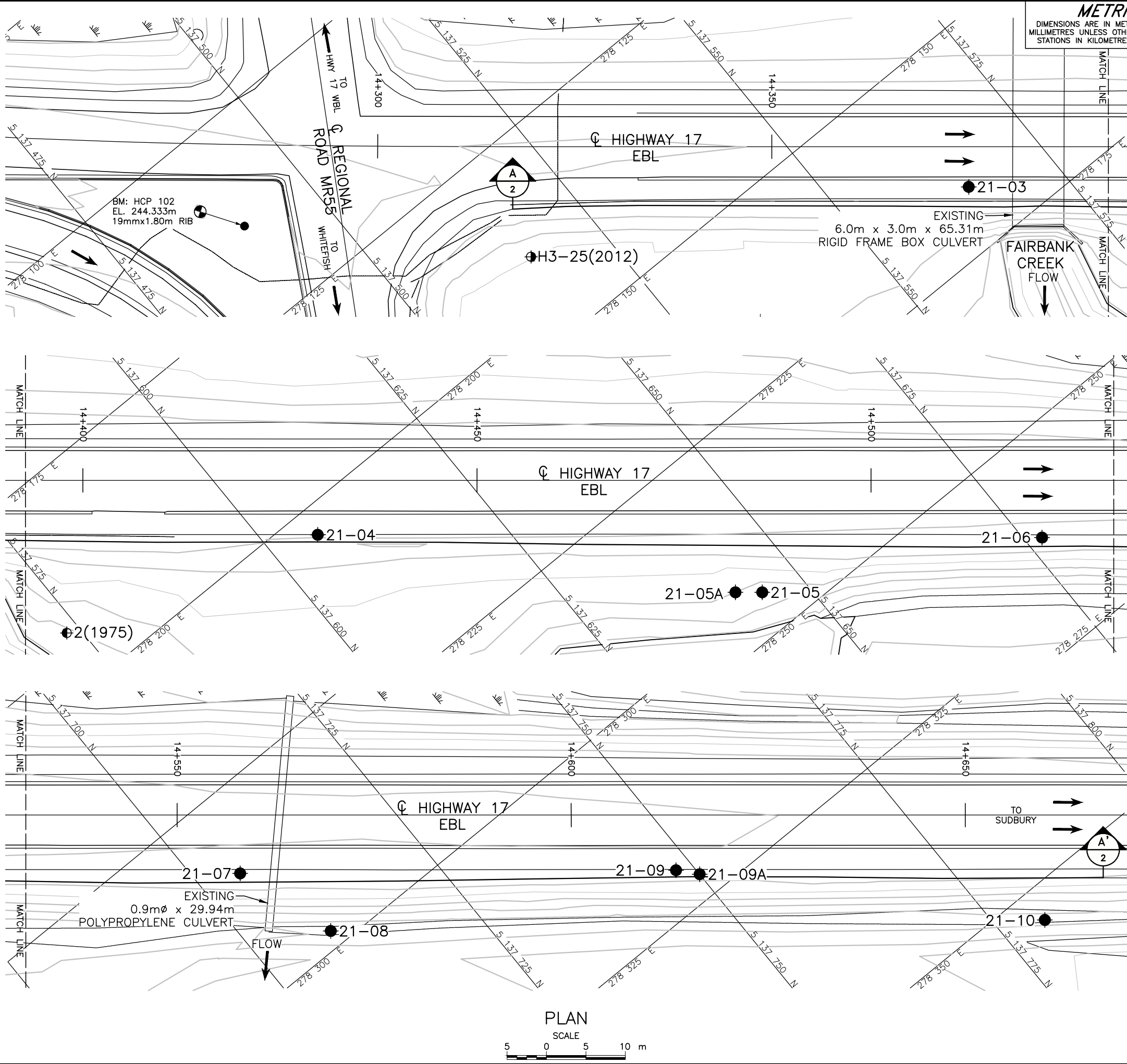
OPSD 203.020	Embankments Over Swamp, Existing Slope Excavated
OPSD 208.010	Benching of Earth Slopes

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 209	Embankments Over Swamp and Compressible Soils
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:

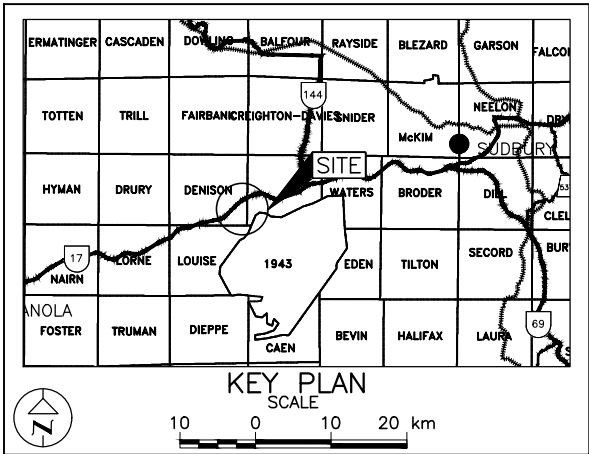
Ontario Regulation 903 Wells (as amended)



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

CONT No.
GWP No. 5032-19-00

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



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
2(1975)	240.9	5137573.4	278193.0
H3-25(2012)	242.2	5137516.0	278139.2
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



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.

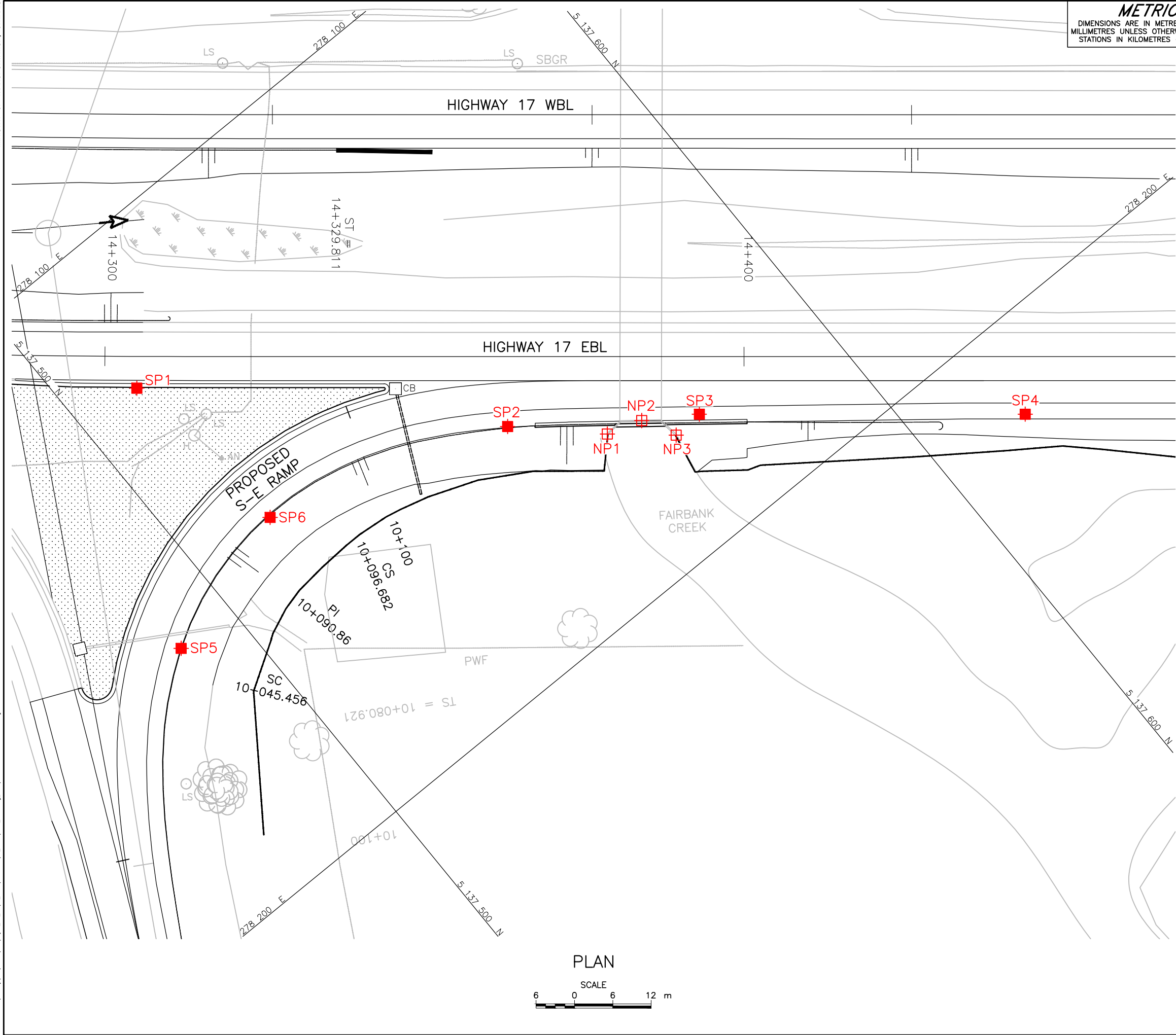
REFERENCE

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. 411-374			
HWY. 17	PROJECT NO. 20253807	DIST.	
SUBM'D.	CHKD. TB	DATE: 11/19/2021	SITE:
DRAWN: TR	CHKD. MT	APPD. KB	DWG. 1



NO.	DATE	BY	REVISION
Geocres No. 411-374			
HWY. 17		PROJECT NO. 20253807	DIST. .
SUBM'D.	CHKD. TB	DATE: 11/19/2021	SITE:
DRAWN: TR	CHKD. MT	APPD. KB	DWG. 2



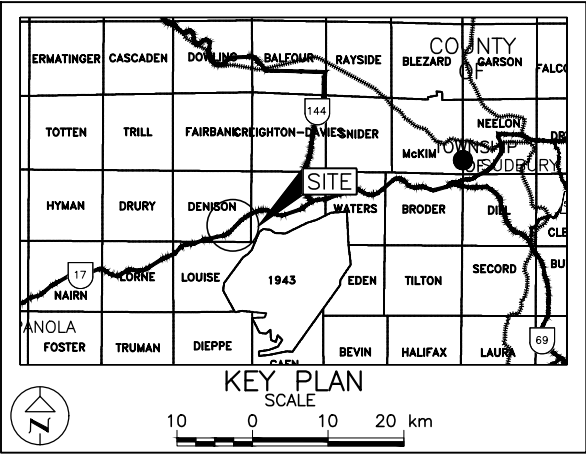
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 5032-19-00

GWP No. 5032-19-00

HIGHWAY 17 AND
MUNICIPAL ROAD 55
MONITORING INSTRUMENT
LOCATION PLAN

SHEET



LEGEND	
	Settlement Plate (SP)
	Nail Pin (NP)

BOREHOLE	ALIGNMENT	STATION (m)	OFFSET (m)
SP1	HWY 17 EBL	14+305.0	5.0
SP2	HWY 17 EBL	14+363.0	11.0
SP3	HWY 17 EBL	14+393.0	9.0
SP4	HWY 17 EBL	14+444.0	9.0
SP5	PROPOSED S-E RAMP	10+050.0	7.5
SP6	PROPOSED S-E RAMP	10+080.0	8.0
NP1	HWY 17 EBL	14+378.6	12.1
NP2	HWY 17 EBL	14+384.0	10.0
NP3	HWY 17 EBL	14+389.4	12.3



NOTES

The location of the Settlement Plate (SP) and Nail Pin (NP) are shown for illustration purposes only.

The final location of the Settlement Plate (SP) and Nail Pin (NP) shall be agreed upon by the Contractor and Contract Administrator prior to installation.

REFERENCE

Base plans provided in digital format by AECOM CANADA LTD., drawing file no. GWP 5032-19-00 NEW CONSTRUCTION.DWG, Received NOVEMBER 3, 2021

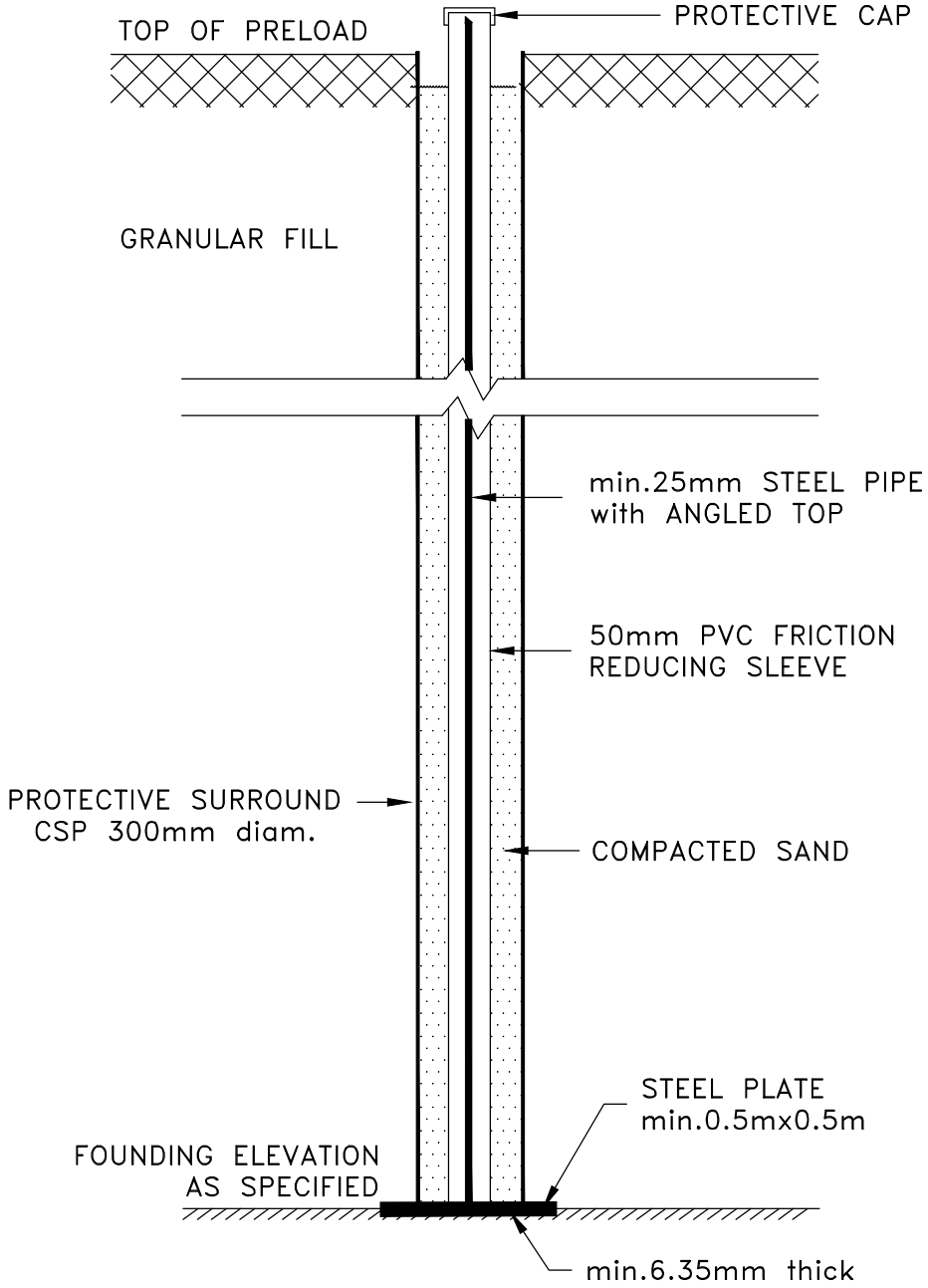
NO.	DATE	BY	REVISION
1			
Geocres No. 411-374			
HWY. 17	PROJECT NO. 20253807	DIST.	
SUBM'D.	CHKD.	DATE: 11/19/2021	SITE:
DRAWN: TR	CHKD: MT	APPD: KB	DWG. 3

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

CONT No. _____
GWP No. 5032-19-00

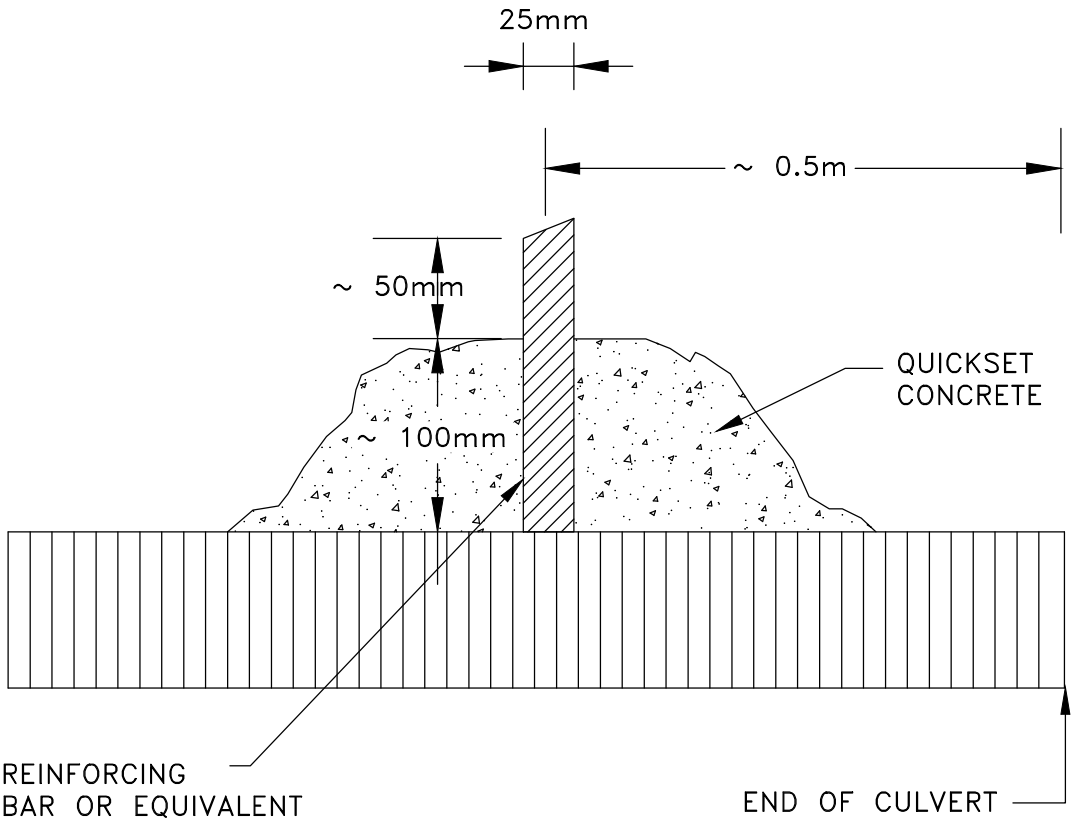
HIGHWAY 17 AND
MUNICIPAL ROAD 55
TYPICAL INSTRUMENT
INSTALLATION DETAILS

SHEET



SETTLEMENT PLATE (SP)

NOT TO SCALE



NAIL PIN (NP)

NOT TO SCALE



NO.	DATE	BY	REVISION
1			
Geocres No. 411-374			
HWY. 17	PROJECT NO. 20253807		DIST. .
SUBM'D.	CHKD.	DATE: 11/19/2021	SITE: .
DRAWN: TR	CHKD. MT	APPD. KB	DWG. 4



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 (Up to 2 m of 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>

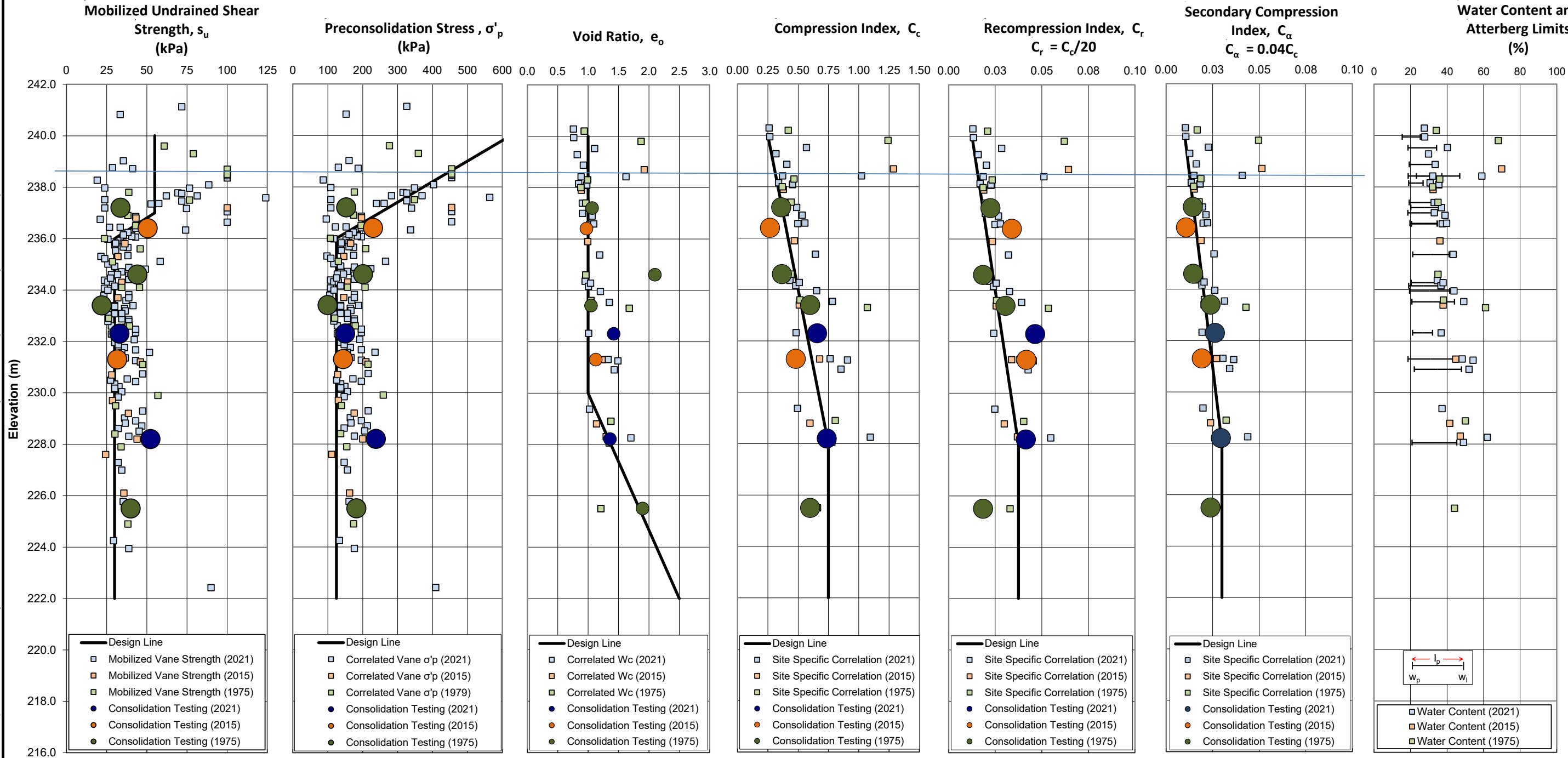
Settlement Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
			<p>walls / systems considered.</p> <ul style="list-style-type: none"> ■ Re-grading is required prior to final pavement structure construction. ■ Time and cost of wick drain and instrumentation installation. 	<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 (Up to 2 m of 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.

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

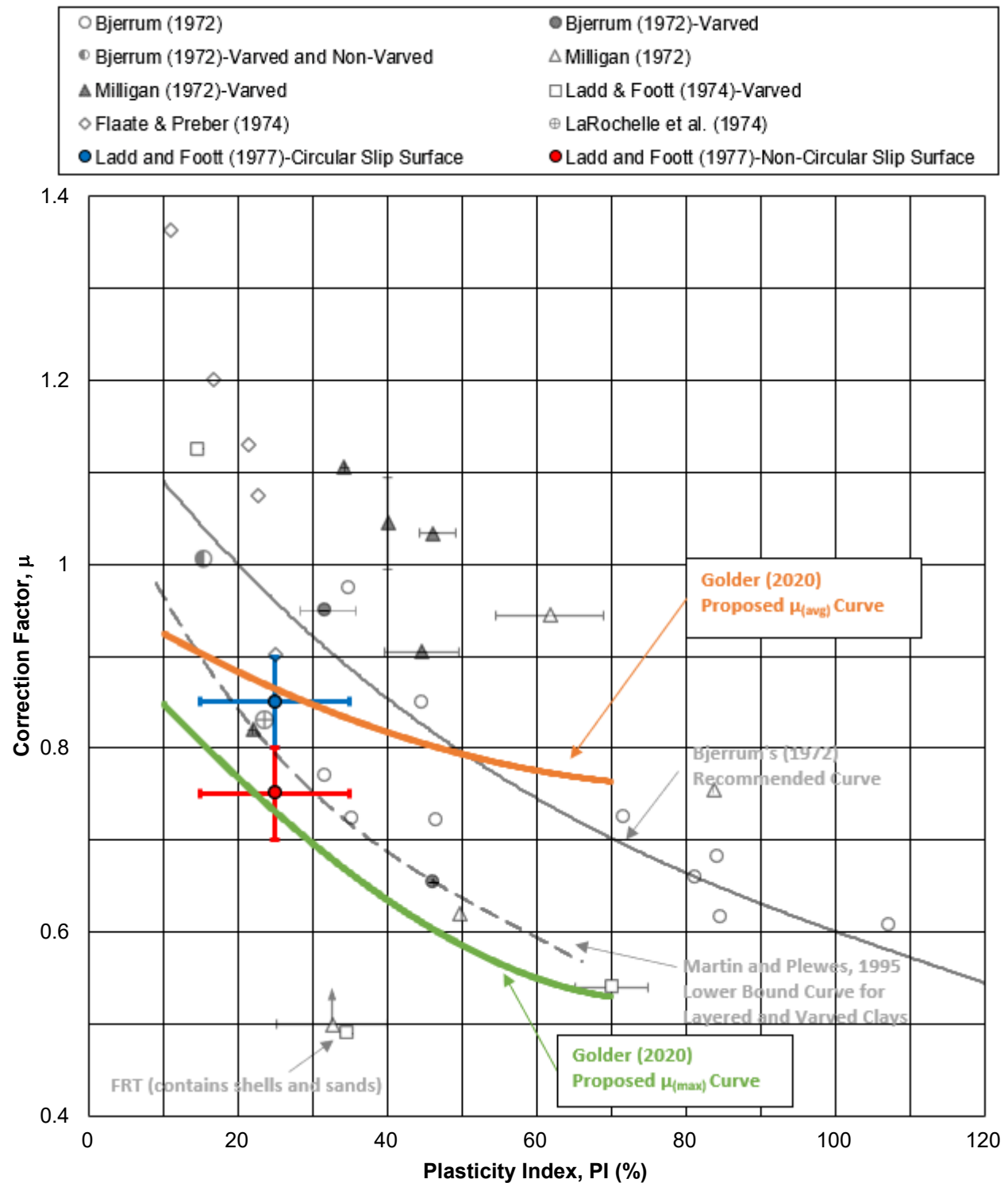
SUMMARY OF ENGINEERING PARAMETERS
FOR COHESIVE DEPOSITS
Highway 17 and MR 55 Widening

FIGURE 1



PROPOSED FIELD VANE CORRECTION FACTORS FOR VARVED CLAYS

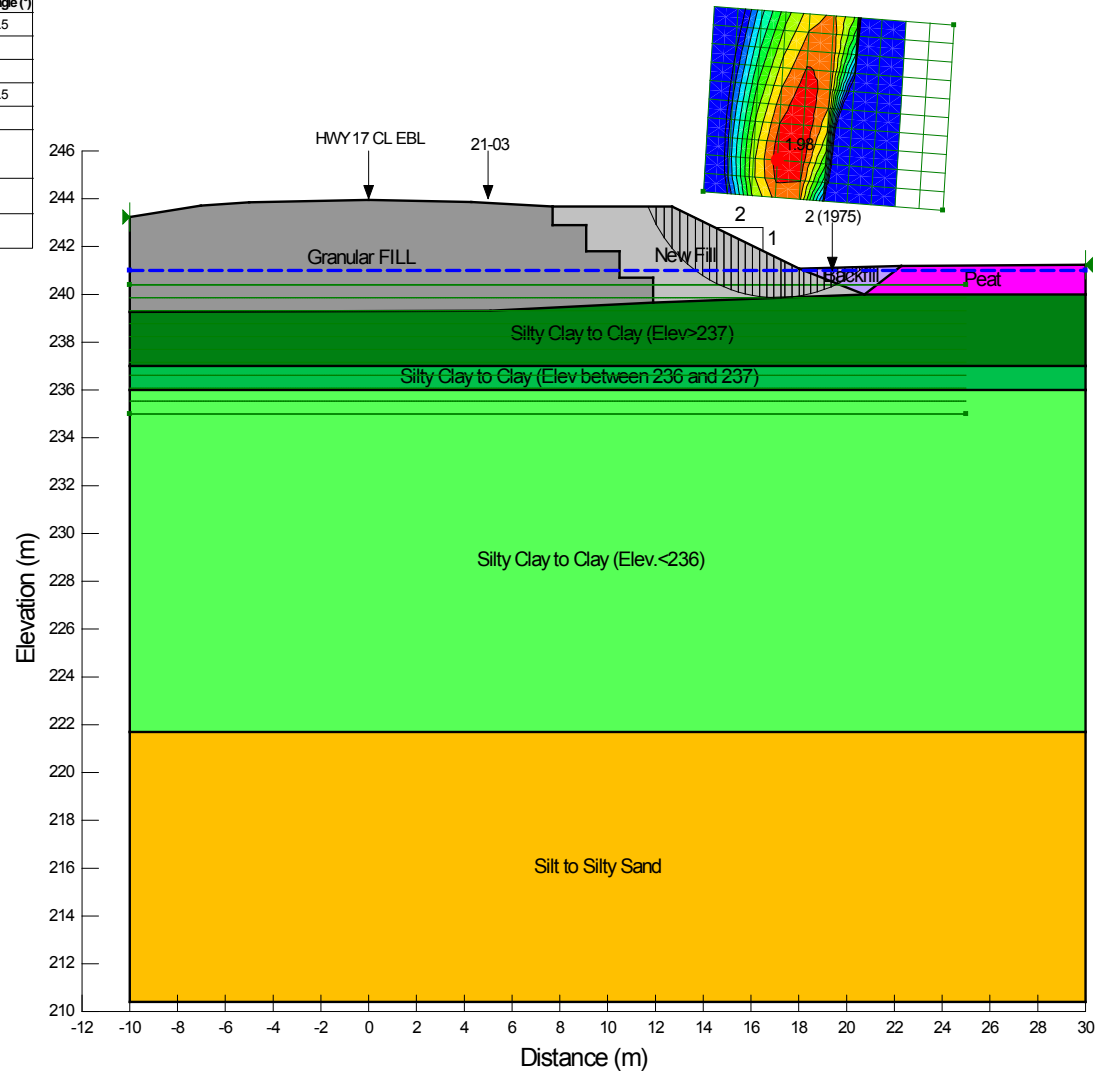
FIGURE 2










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

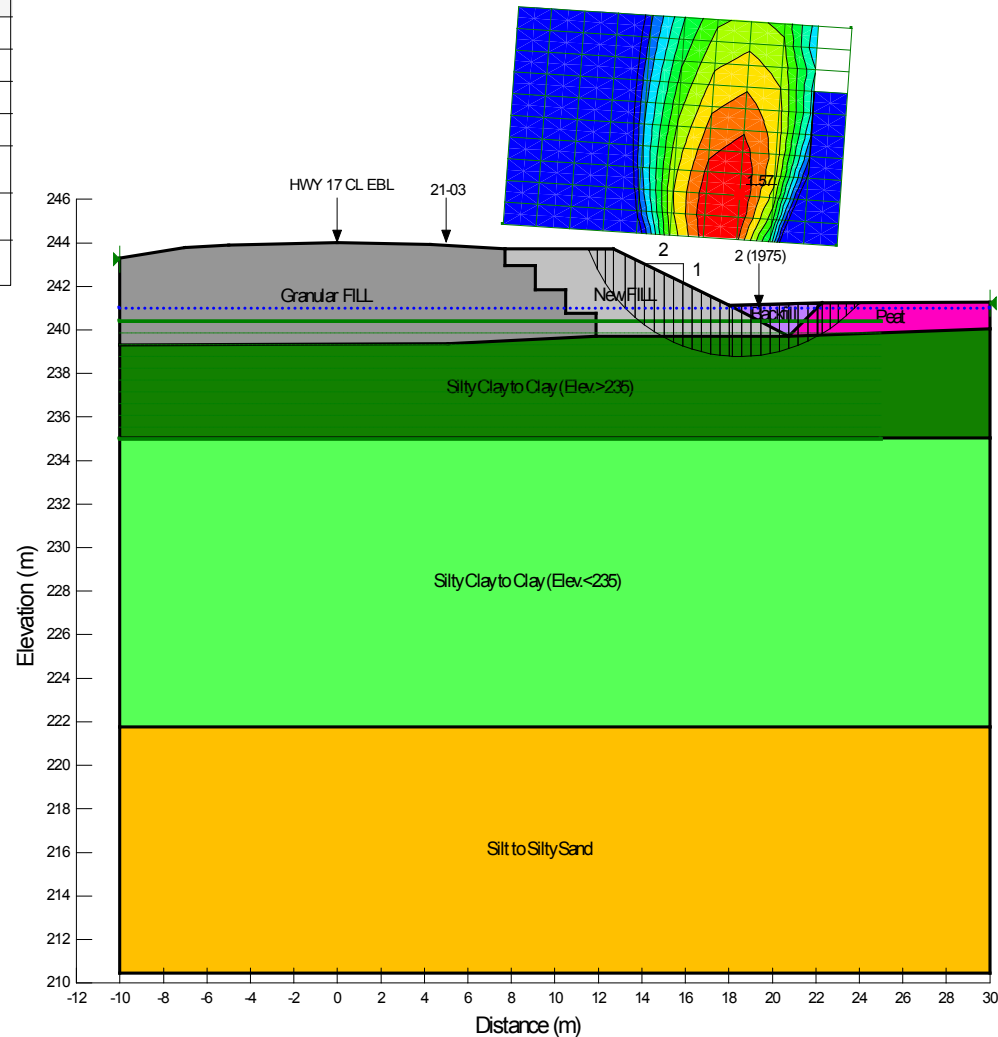
Station 14+380, Short-term Condition (Total Stress Parameters)

Color	Name	Unit Weight (kN/m ³)	Cohesion (kPa)	C-Datum (kPa)	C-Rate of Change ((kN/m ²)/m)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Backfill	12.5				1	27.5
	Granular FILL	21				0	35
	New Fill	21				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	-25		
	Silty Clay to Clay (Elev <236)	18	30				
	Silty Clay to Clay (Elev >237)	18	55				



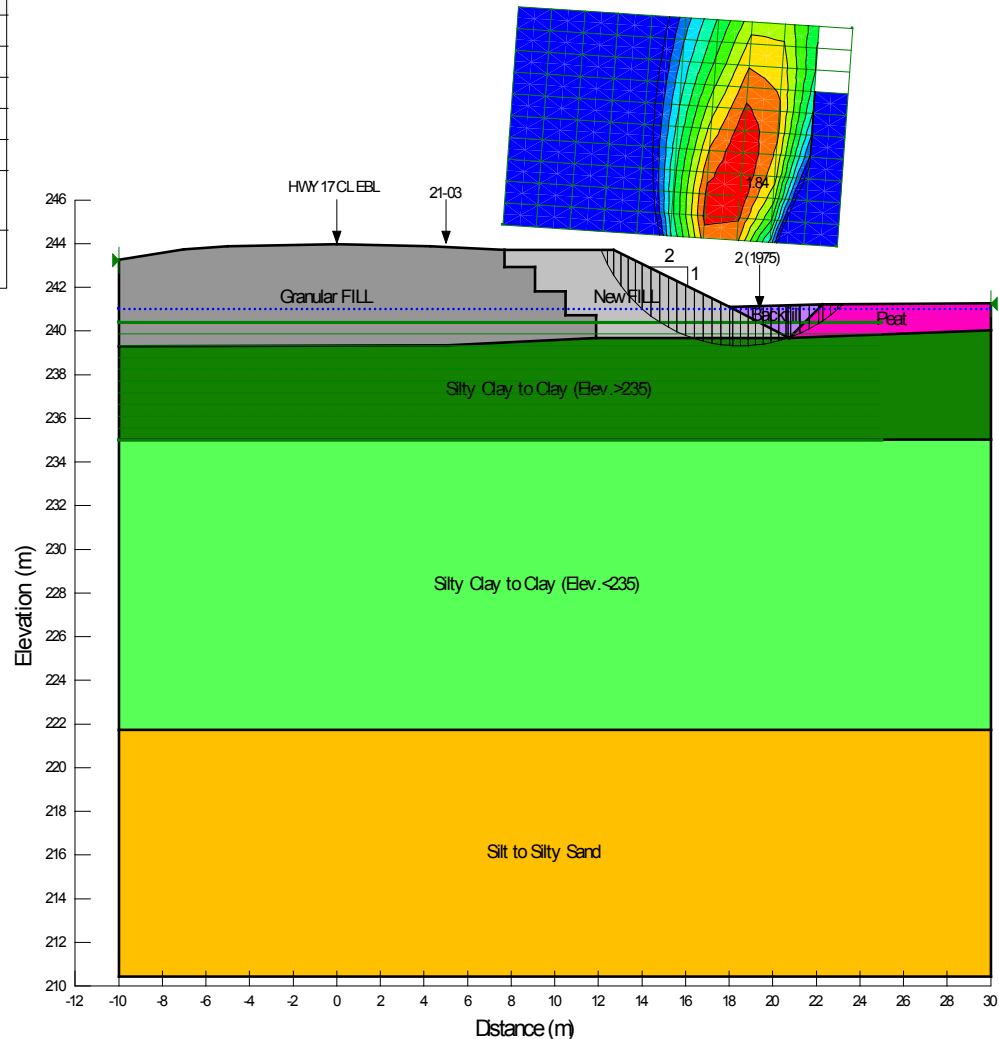
Station 14+380, Short-term Condition (Effective Stress Parameters)

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	21	0	35	0	No
	New FILL	21	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	21	0	35	0	No
	New FILL	21	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



APPENDIX A

Record of Boreholes

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	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
SAND	Coarse	2.00 to 4.75	(10) to (4)
	Medium	0.425 to 2.00	(40) to (10)
	Fine	0.075 to 0.425	(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)

1. Only applicable to components not described by Primary Group Name.

2. 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

1. 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.

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

(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 $= \sigma'_p / \sigma'_{vo}$

(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

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

Notes: 1
2

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

R:\SUD-BURY\SIM\CLIENTS\SIMTO\HWY17 MR 55\02 DATA\GINT\20253807\20253807.GPJ GAL-MISS.GDT 7/16/21 TR

Continued Next Page

+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 ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>						
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		<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>						
						231							
						230							
			13	TO	PH								
						229							
			14	SS	WH								
15.9	END OF BOREHOLE					228							
	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.												

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

R:\SUD-BURY\SIM\CLIENTS\SIMTO\HWY17 MR 55\02 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. 21-04		2 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		W _p	W	W _L		
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
	--- CONTINUED FROM PREVIOUS PAGE ---													
	SILTY CLAY (CI) Firm to stiff Grey Wet		12	TO	PH		231							
							230	7 +	8 +					
			13	SS	WH									
							229	6 +	5 +					
			14A	TO	PH		228						17.0	
			14B										17.7	
							227	5 +	4 +					
			15	SS	WH									
							226	4 +	4 +					
225.8	SILT (ML), trace to some sand, trace clay Very loose to compact Grey Wet		16	SS	4		225							NP
17.8							224	5 +	5 +					
			17	SS	7									0 1 91 8 NP
							223							
			18	SS	2		222							0 18 79 3
							221							
			19	SS	WR									
							220							

Continued Next Page

+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	20	40	60	80	100	W _p	W		
	--- 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 NATURAL MOISTURE CONTENT LIQUID LIMIT Wp W WL	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)	
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × REMOULDED							
						20 40 60 80 100			20 40 60						
242.2	GROUND SURFACE														
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				3A	SS	4									
1.7	ORGANIC SILT (OL) Very loose to loose Dark grey Moist			3B											
				4A	SS	11									
239.6	Gravelly SAND (SP), some silt, trace clay Very loose to compact Grey Wet			4B											
2.6				5	SS	2									
238.5	CLAYEY SILT (CL) Firm to stiff Grey Wet			6	SS	4									
3.7				7	TO	PH									
				8	SS	WH									
236.2	SILTY CLAY (CI) Firm Grey Wet			9	TO	PH									
6.0															
				10	SS	WH									
				11	SS	WH									
232.3	END OF BOREHOLE														
9.9															
NOTES:															
1. Vanes obtained in separate unsampled borehole (21-05A) advanced 2.0 m south of original Borehole 21-05.															
2. Water level measured at a depth of 9.6 m below ground surface (Elev. 232.6 m) in augers upon completion of drilling.															

+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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
+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 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
	--- CONTINUED FROM PREVIOUS PAGE ---												
	SILTY CLAY (CI) Firm Grey Wet		12	SS	WH		231						
								230					
228.2													
15.3	SILT (ML), trace clay, trace sand Very loose Grey Wet		14	SS	2		228					0 1 90 9 NP	
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.												

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

PROJECT		RECORD OF BOREHOLE No. 21-07				2 OF 2 METRIC											
G.W.P. 5032-19-00		LOCATION N 5137704.9; E 278285.0 NAD83 MTM ZONE 12 (LAT. 46.378534; LONG. -81.344687)				ORIGINATED BY TB/NP											
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			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	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p	W	W _L	γ	GR SA SI CL
231.2 12.1	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100									
	SILT (ML), trace clay, trace sand Very loose to loose Grey Wet		12	SS	4		231										0 1 90 9 NP
							230										
			13	SS	1		229										
228.5 14.8	SAND (SP), some silt Very loose Grey Wet						228										
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.																

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			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p	W	W _L	γ	GR	SA	SI	CL
241.0	TOP OF SNOW							20	40	60	80	100								
0.0	SNOW / ICE																			
240.7																				
0.3	SAND (SP) and gravel, trace silt (FILL) Very loose		1	SS	2															
240.0	- No recovery in Sample No. 1.																			
1.0	CLAYEY SILT (CL) Stiff Grey Moist to wet		2	SS	2		240													
			3	SS	5		239													
			4	SS	1		238													
	- Vane could not be advanced at 4.0 m depth.		5	SS	WH		237													
236.0	SILTY CLAY (CI) Firm to stiff Grey Wet						236													
5.0			6	TO	PH		235													
							234													
	- Varves of clayey silt encountered below 7.2 m depth.		7	SS	2		233													
			8A	TO	PH		232													
			8B																	
			9	SS	WH		231													
230.1	END OF BOREHOLE																			
10.9	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.																			

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PROJECT		RECORD OF BOREHOLE No. 21-09/21-09A				2 OF 2 METRIC											
G.W.P. 5032-19-00		LOCATION N 5137748.0; E 278319.6 NAD83 MTM ZONE 12 (LAT. 46.378923; LONG. -81.344241)				ORIGINATED BY TB/NP											
DIST _____ HWY 17		BOREHOLE TYPE CME 55 Track Mount, 108 mm I.D. Hollow Stem Augers				COMPILED BY TR											
DATUM GEODETIC		DATE February 3 & 4, 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					WATER CONTENT (%)			γ kN/m³	GR SA SI CL
							20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60					
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										
227.4	- Heaving sand encountered in Sample No. 14.		14	SS	72		228										
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 Feb 9, 2021 Depth (m) 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.																

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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					WATER CONTENT (%)					
								20 40 60 80 100	○ UNCONFINED + FIELD VANE	○ QUICK TRIAXIAL × REMOULDED	W _p W W _L							
241.2	TOP OF SNOW							20 40 60 80 100				20 40 60						
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		240											
	- Trace organics in Sample No. 1.		2B															
	- Vane could not be advanced at 2.7 m depth.						239											
			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		237											
			5	SS	6													
			6	SS	4		236											
			7	SS	11		235											
	- No recovery in split-spoon No. 7.																	
234.6																		
6.6	SAND (SP) to gravelly SAND (SP) Loose Grey Wet		8	SS	9		234											
							233											
	- Refusal to practical casing advancement at 8.2 m depth.		9	SS	6													
232.4																		
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).																	

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PROJECT <u>11-1191-0007</u>		RECORD OF BOREHOLE No H3-25		1 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/m ³	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									
								20 40 60 80 100	W _P	W	W _L						
242.2	GROUND SURFACE																
0.0	TOPSOIL																
241.9	Sand, some gravel, some silt (FILL) Compact Brown Moist to wet																
0.3			1	AS	-												
			2	SS	12												
239.6	SILTY CLAY to CLAY Firm to stiff Grey Wet Trace organics above 3.7 m depth.																
2.6			3	SS	2												
				4	SS	5											
				5	SS	WH											
				6	TO	PH											
				7	SS	WH											
				8	SS	WH											
			9	SS	WH												
			10	SS	WH												
						</											

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

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

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.

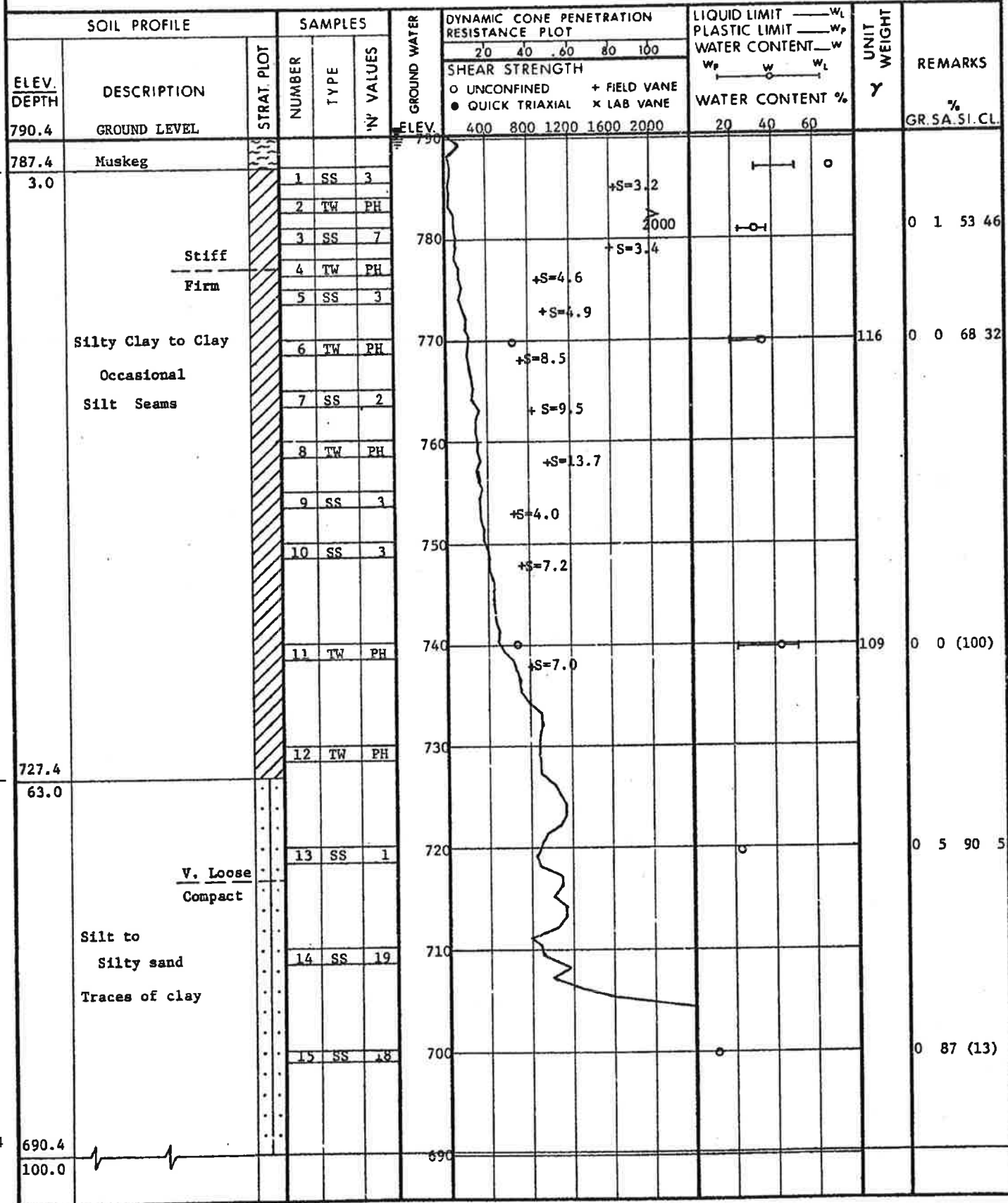
ORIGINATED BY MM

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



OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO
ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 2 (Continued)

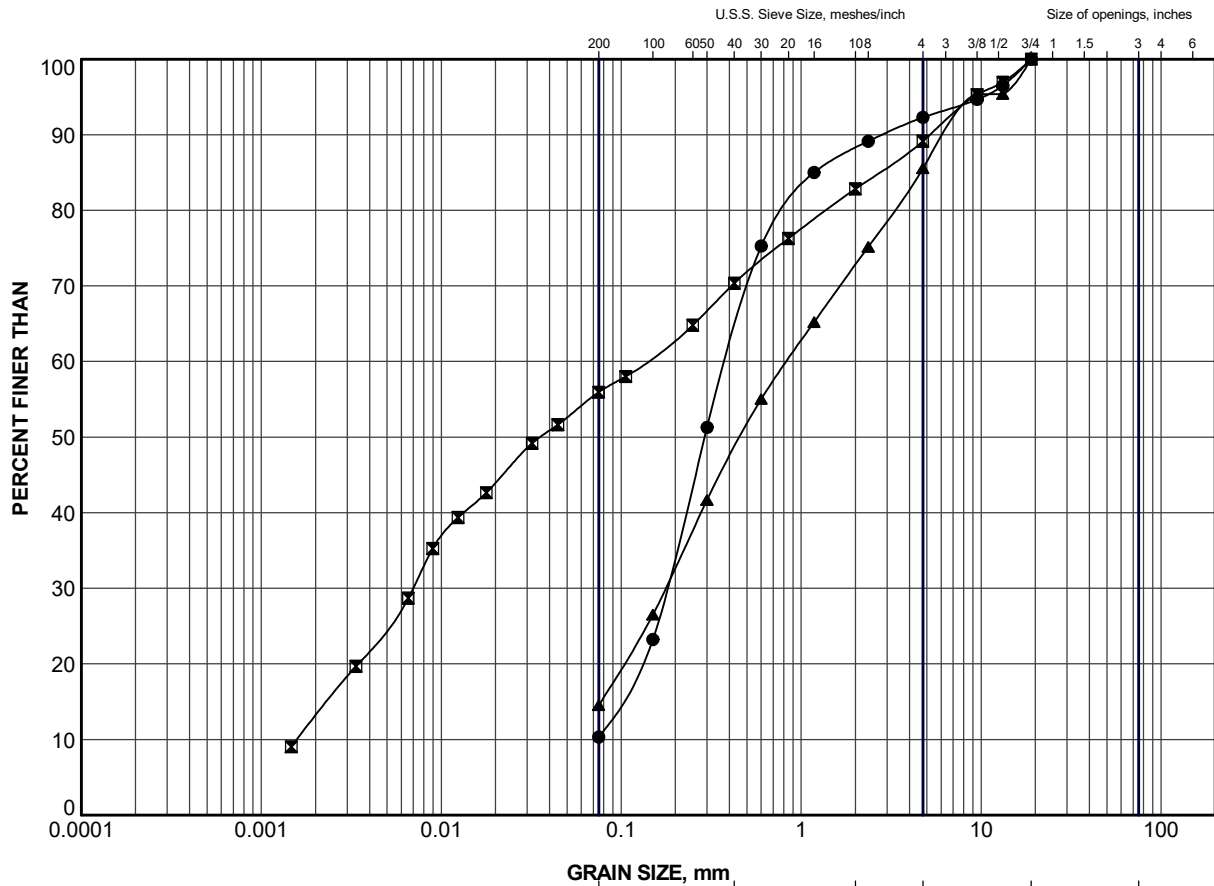
W.P. 61-74-02/03 LOCATION CO-ORDS. 16,854,844N; 912,670E. ORIGINATED BY MM
DIST. 17 HWY. 17 Line 'D' BORING DATE January 16th, 1975 COMPILED BY MM
DATUM GEODETIC BOREHOLE TYPE HOLLOW STEM AUGER & CONE TEST CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w w_p — w — w_L WATER CONTENT % 20 40 60	UNIT WEIGHT γ	REMARKS % GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100			
210.3 690.4			16	SS	35	690								
30.5 100.0	Sand and gravel Traces of Silt Compact to dense		17	SS	28	680								51 42 (7)
203.7 668.4			18	SS	47	670								
37.2 122.0	End of Borehole					660								
197.5 647.9						650								
43.4 142.5	End of Cone Penetration					640								

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX B


Laboratory Test Results

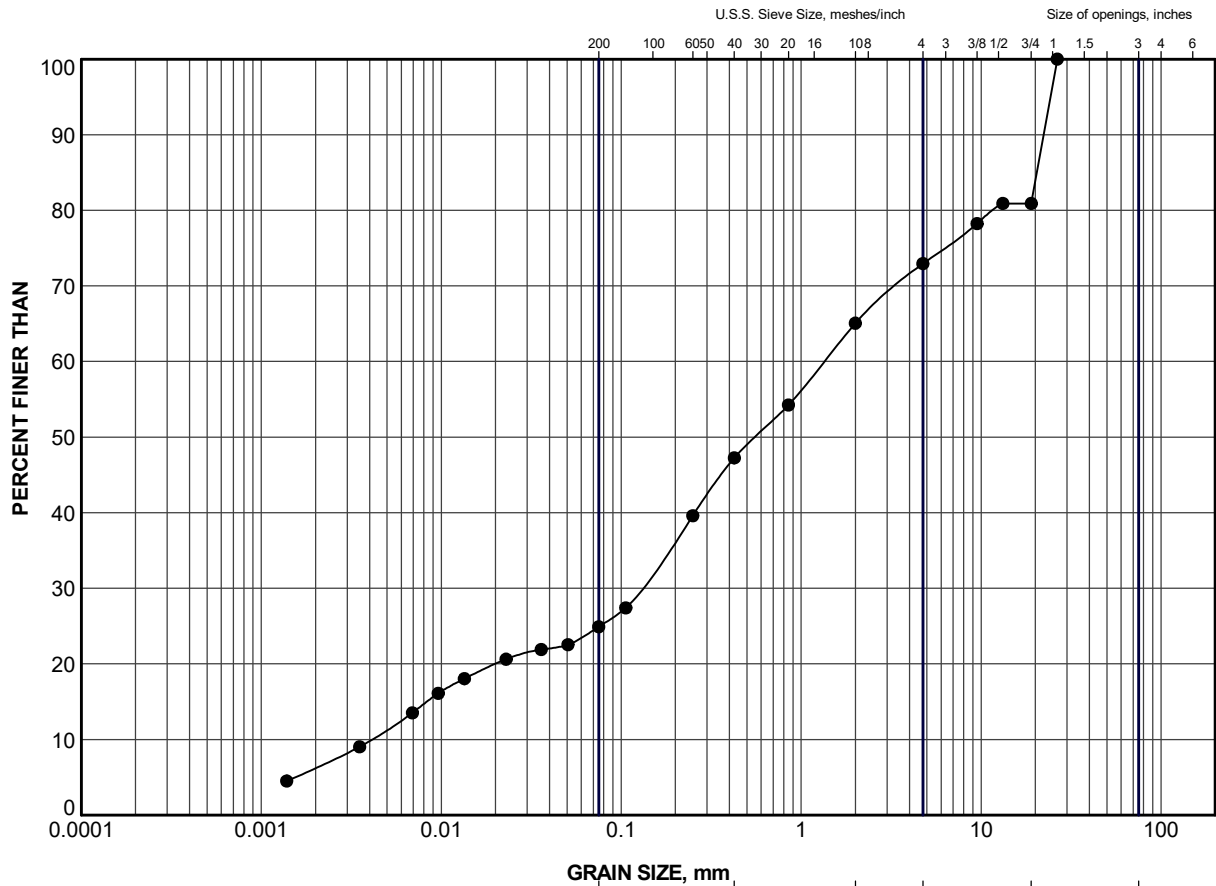


GRAVEL SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

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)					
PROJECT No.			20253807			FILE No.			20253807.GPJ		
DRAWN	TR	Jul 2021	SCALE	N/A	REV.	FIGURE B-1					
CHECK	TB	Jul 2021									
APPR	MT	Jul 2021									
 GOLDER MEMBER OF WSP SUDBURY, ONTARIO											

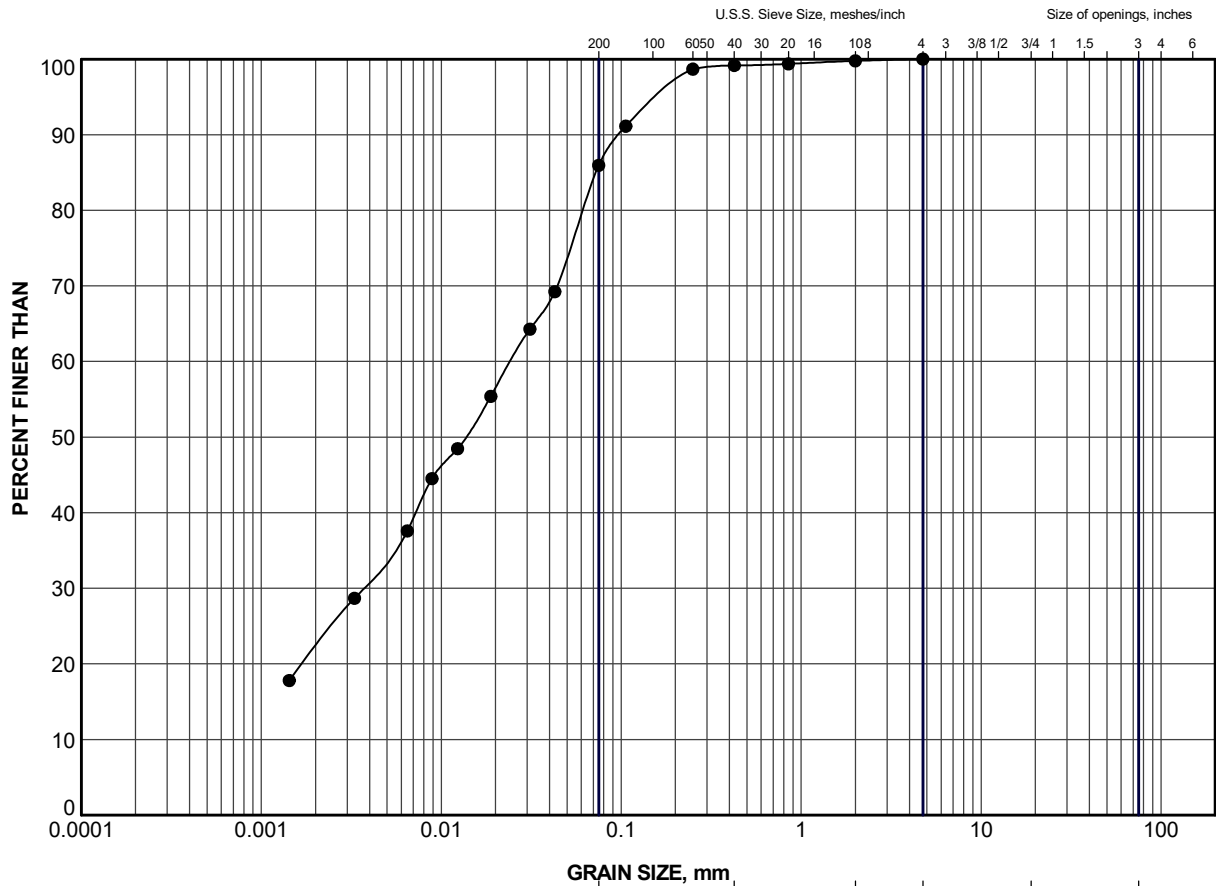


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

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


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

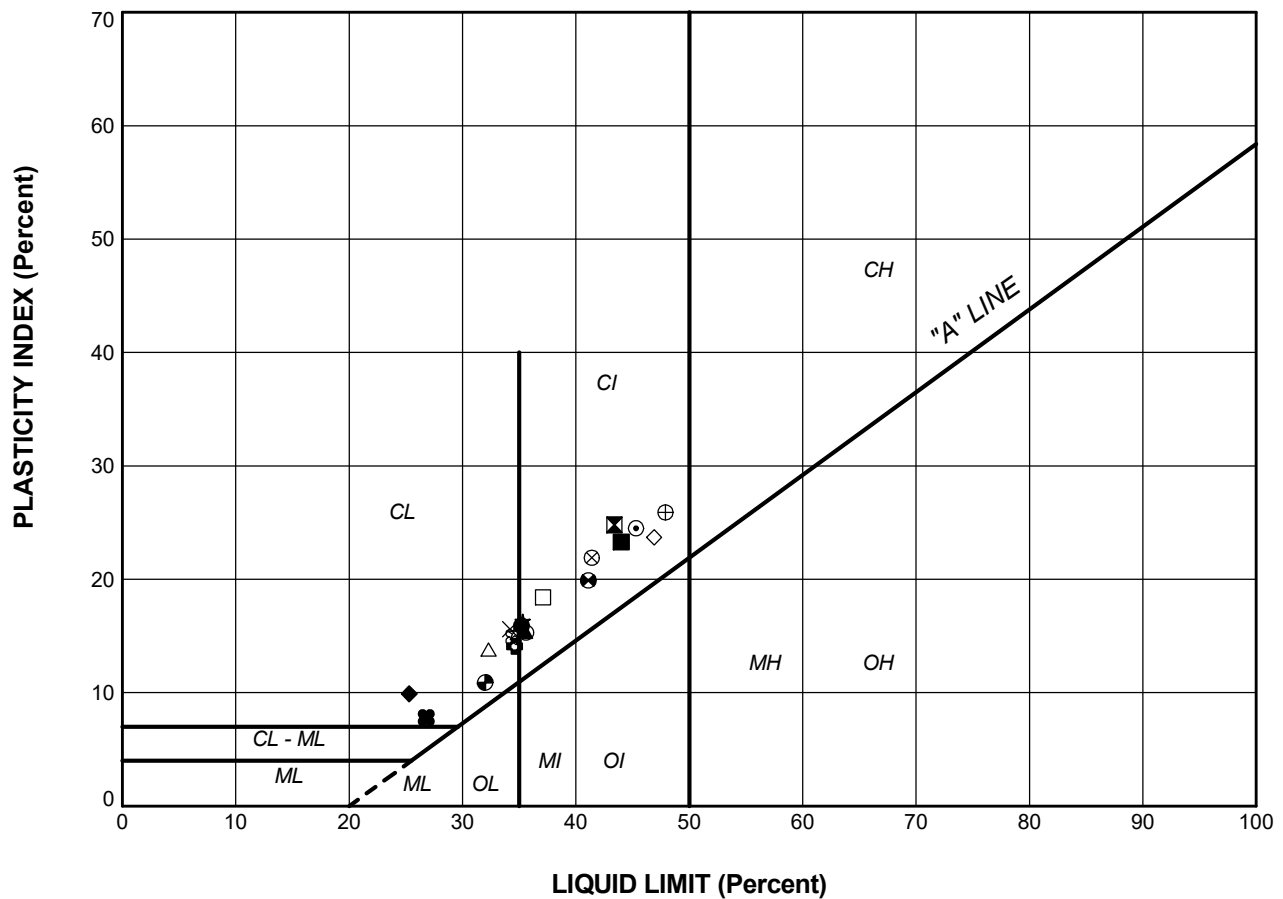


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

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.		20253807.GPJ
DRAWN	TR	Jul 2021	SCALE	N/A	REV.
CHECK	TB	Jul 2021	FIGURE B-4		
APPR	MT	Jul 2021			
GOLDER MEMBER OF WSP			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		



Prepared By: TG

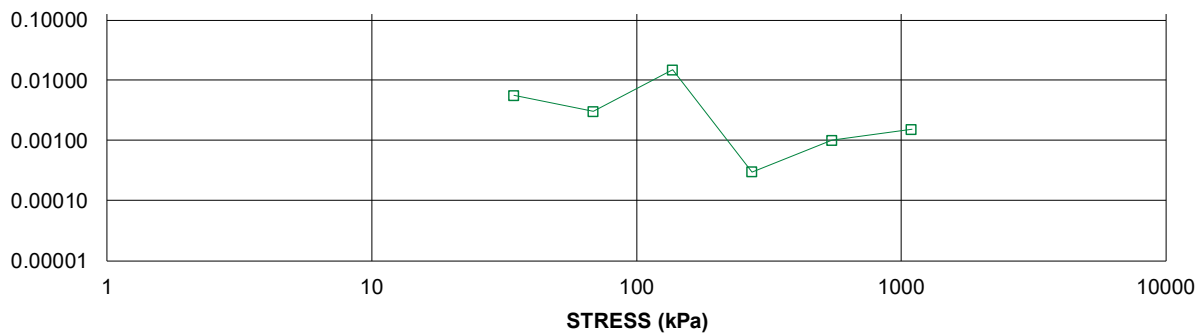
Checked By: MT

CONSOLIDATION TEST SUMMARY

FIGURE B-5
Pg. 2 of 4

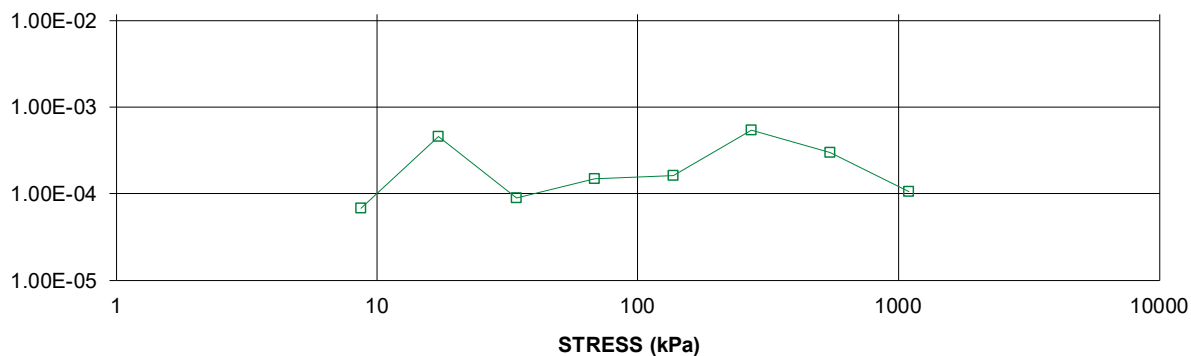
COEFFICIENT OF CONSOLIDATION
(cm²/s)

COEFFICIENT OF CONSOLIDATION (cm²/s) VS PRESSURE (kPa)
21-8 Sa 8A



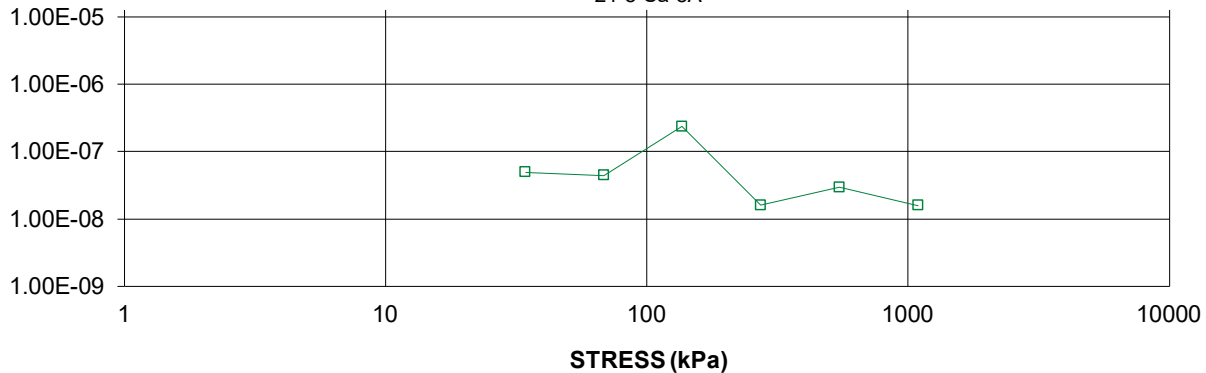
VOLUME COMPRESSIBILITY
(m²/kN)

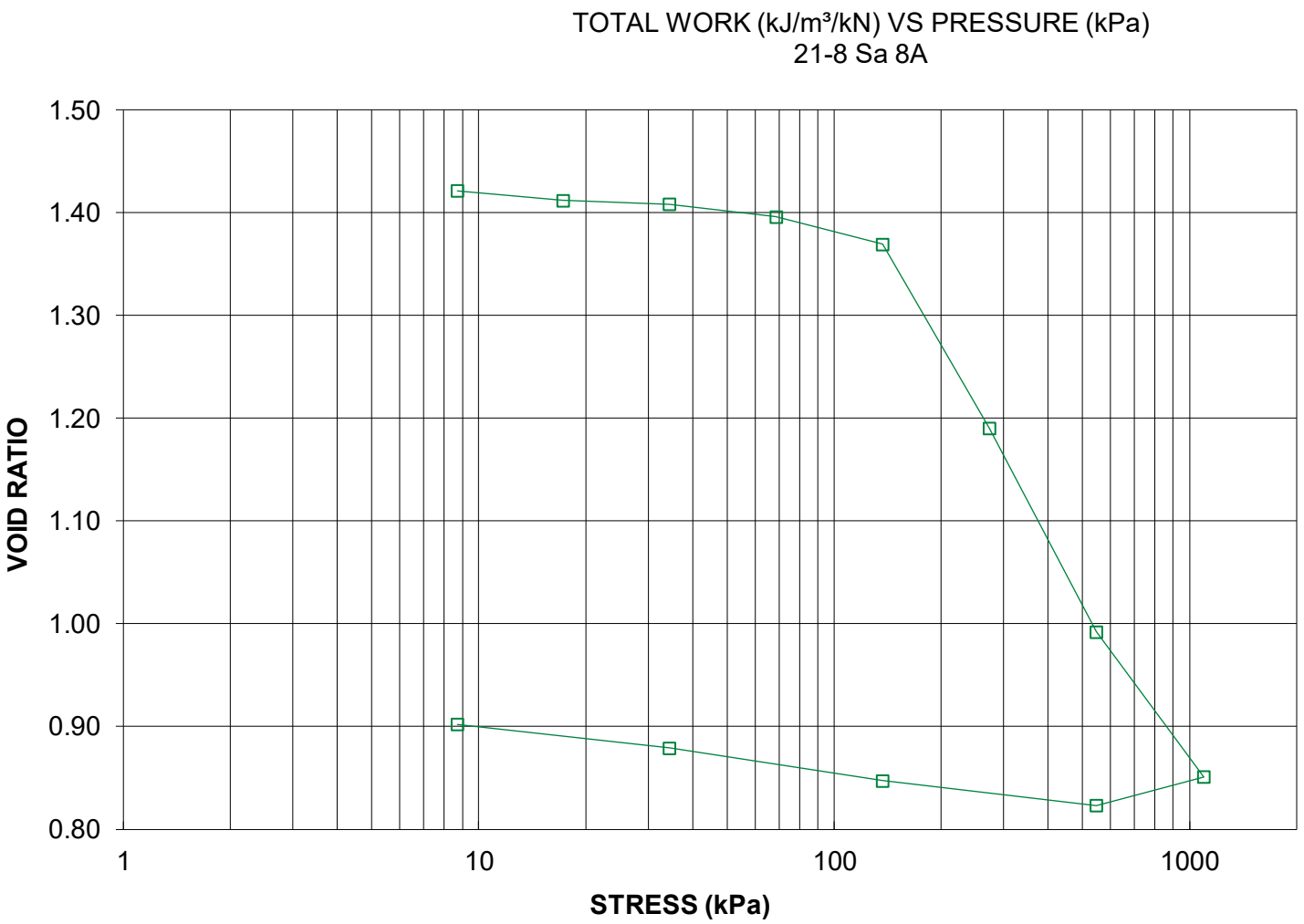
MODULUS OF VOLUME COMPRESSIBILITY (m²/kN) VS PRESSURE (kPa)
21-8 Sa 8A

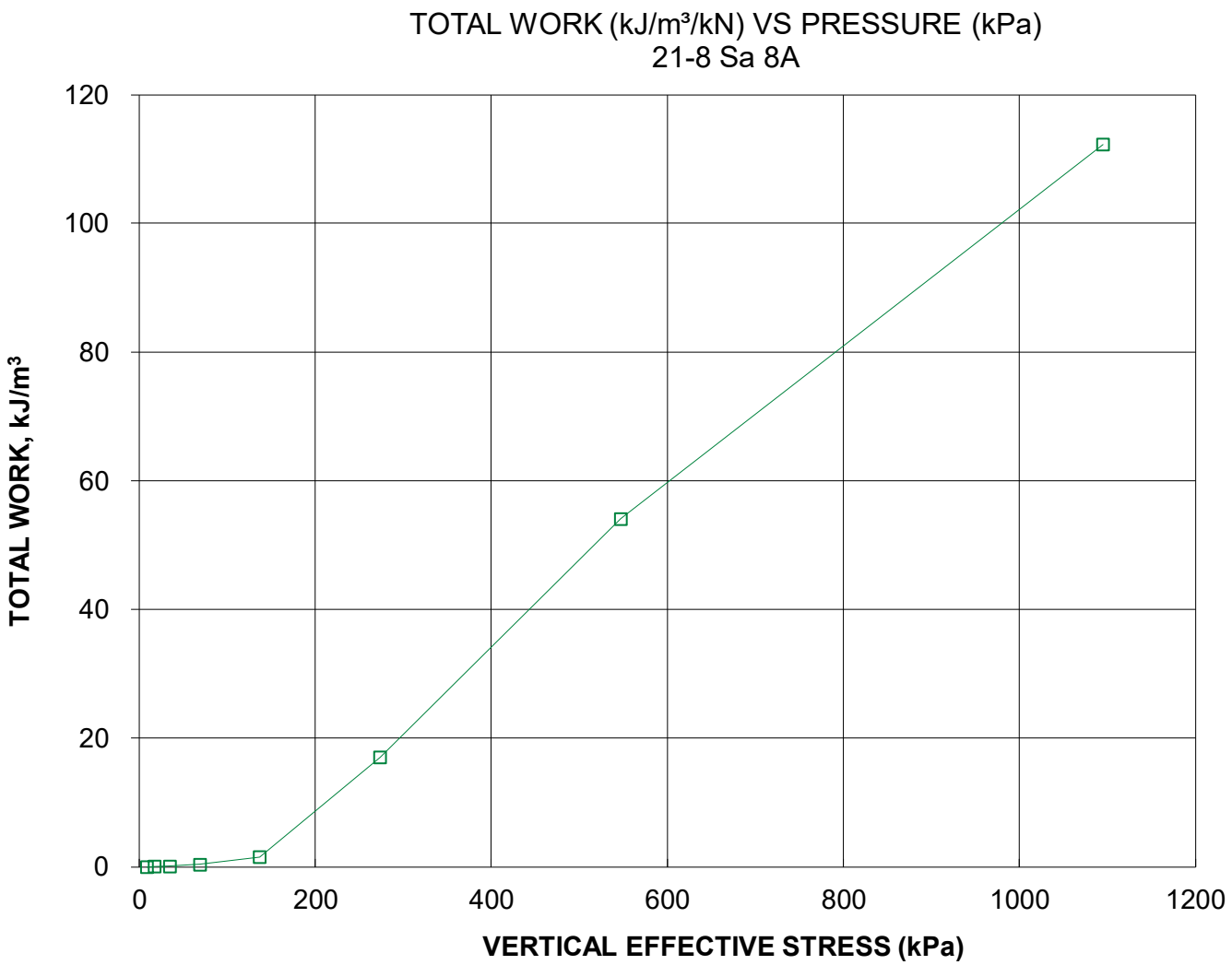


HYDRAULIC CONDUCTIVITY
(cm/s)

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		



Prepared By: TG

Checked By: MT

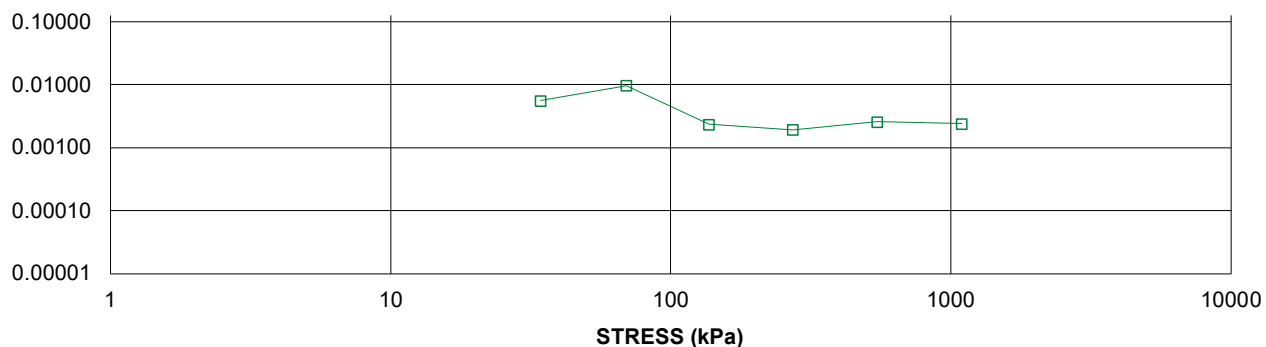
CONSOLIDATION TEST SUMMARY

FIGURE B-6

Pg. 2 of 4

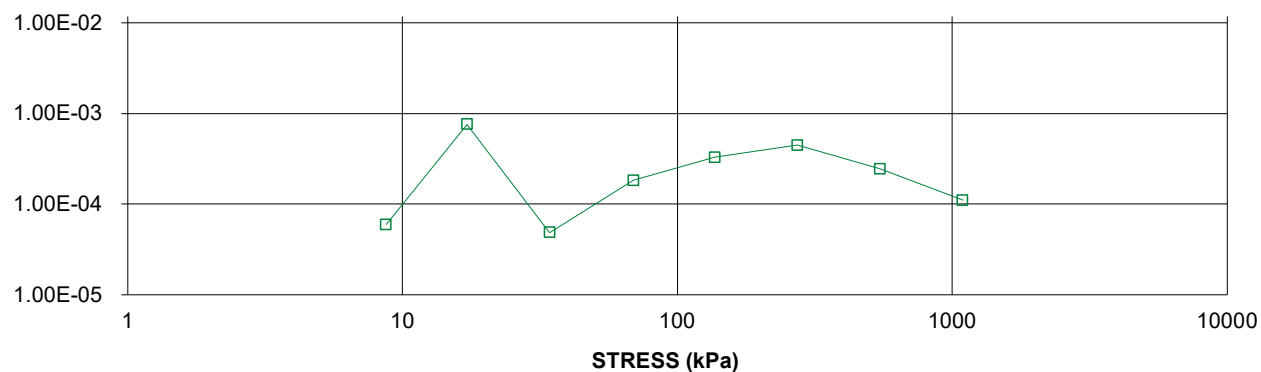
COEFFICIENT OF CONSOLIDATION
(cm²/s)

COEFFICIENT OF CONSOLIDATION (cm²/s) VS PRESSURE (kPa)
21-8 Sa 8B



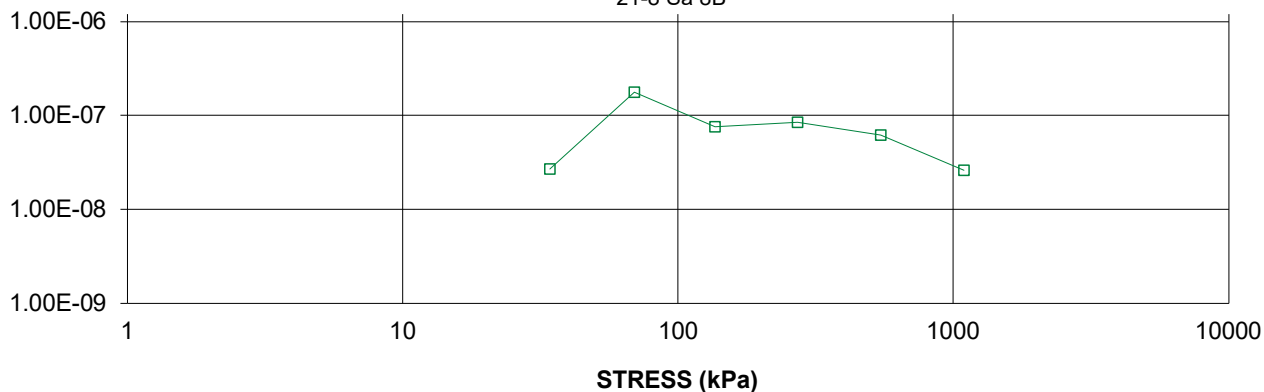
VOLUME COMPRESSIBILITY
(m²/kN)

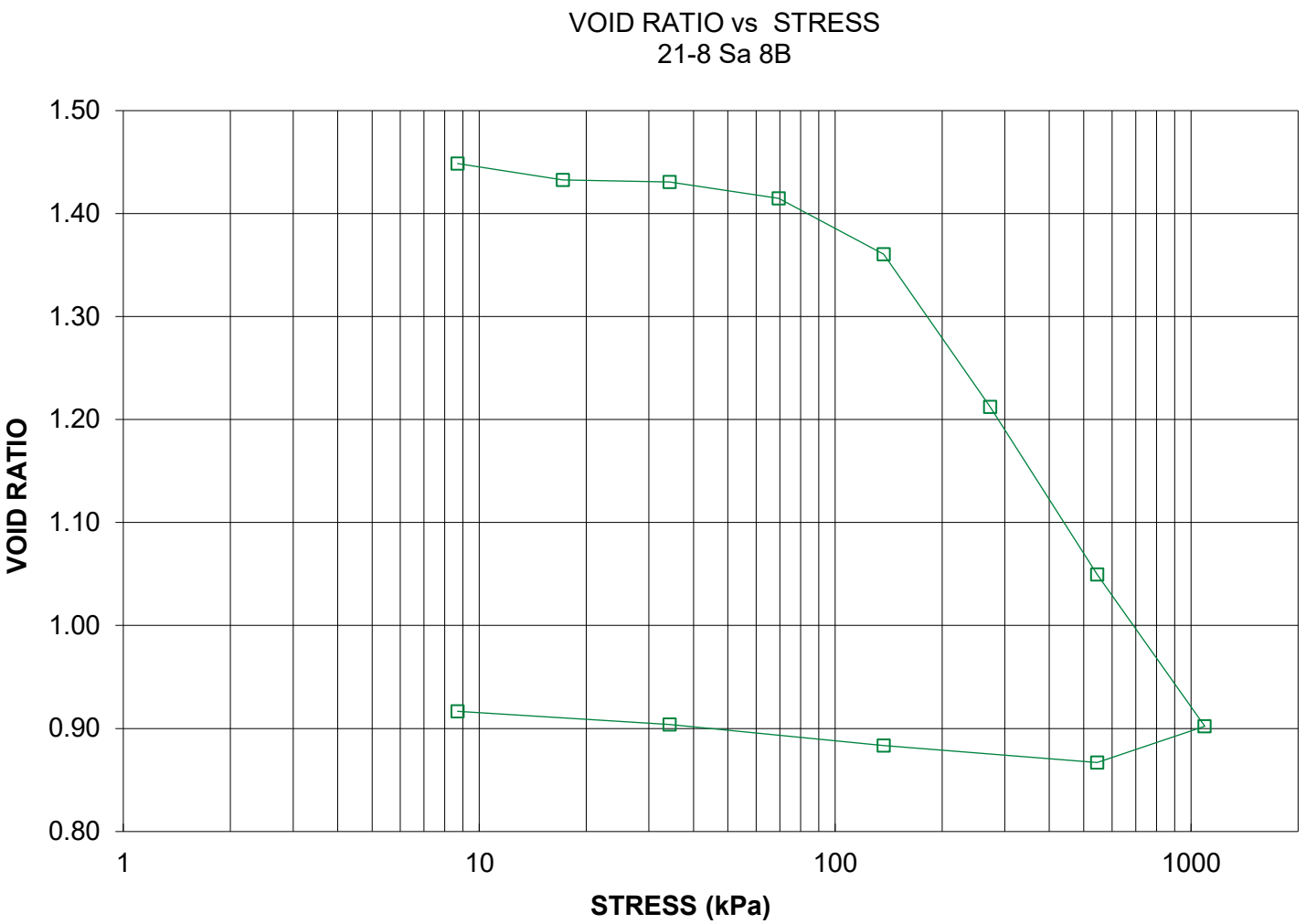
MODULUS OF VOLUME COMPRESSIBILITY (m²/kN) VS PRESSURE (kPa)
21-8 Sa 8B

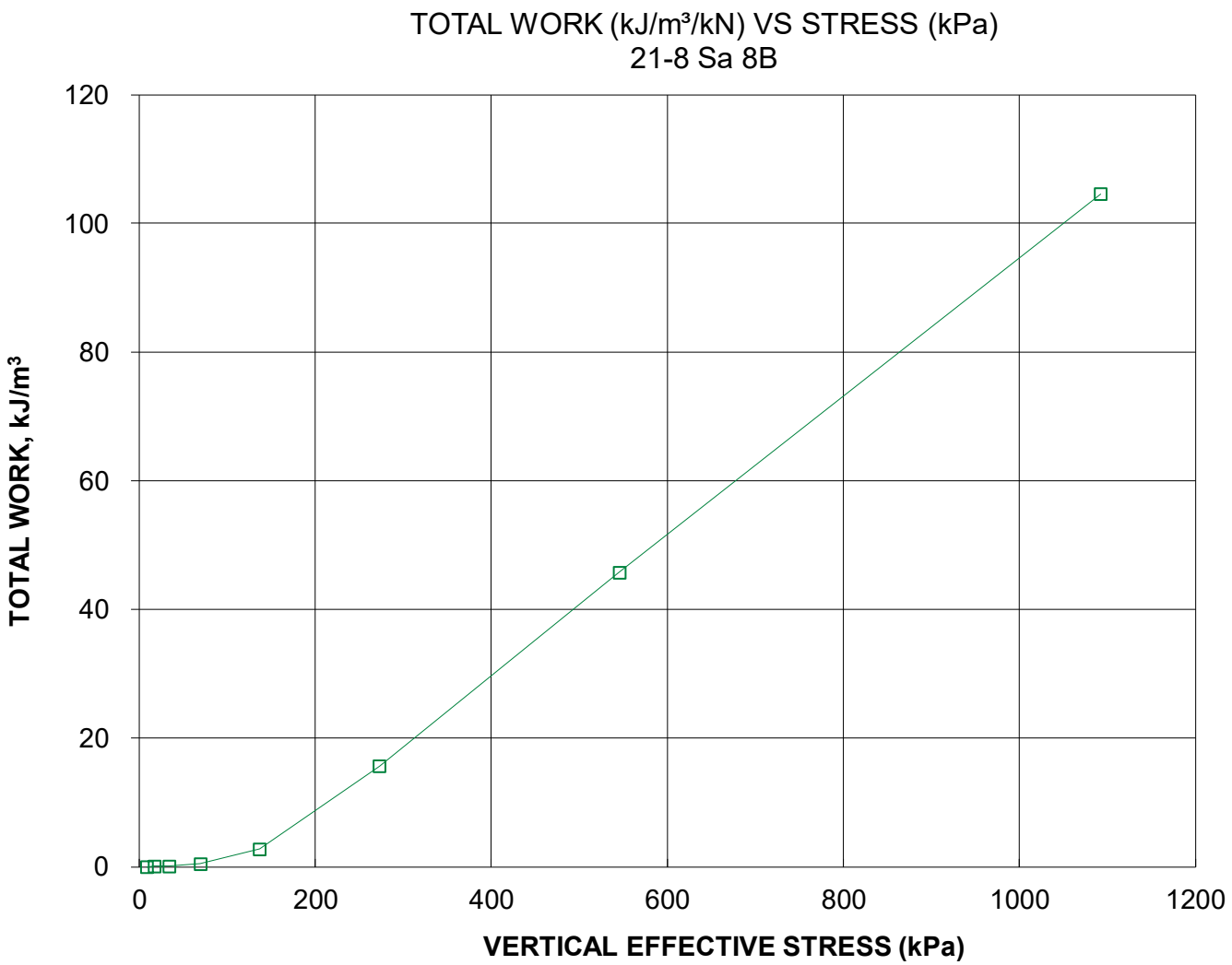


HYDRAULIC CONDUCTIVITY
(cm/s)

HYDRAULIC CONDUCTIVITY (cm/s) vs PRESSURE (kPa)
21-8 Sa 8B







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		



Prepared By: TG

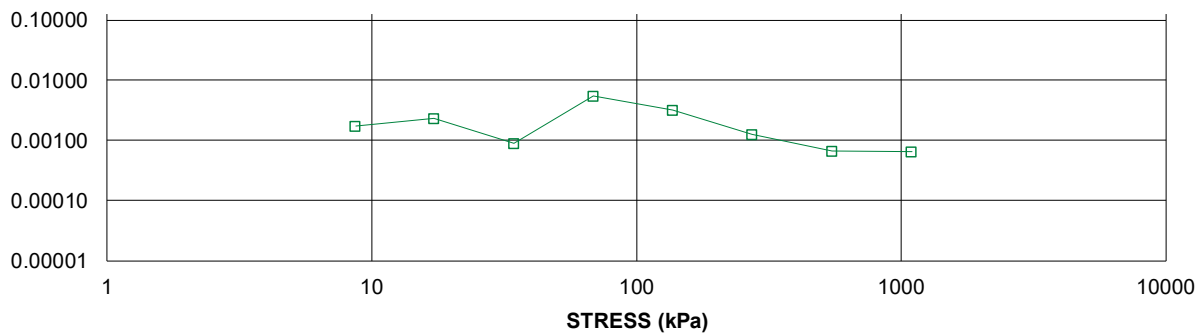
Checked By: MT

CONSOLIDATION TEST SUMMARY

FIGURE B-7
Pg. 2 of 4

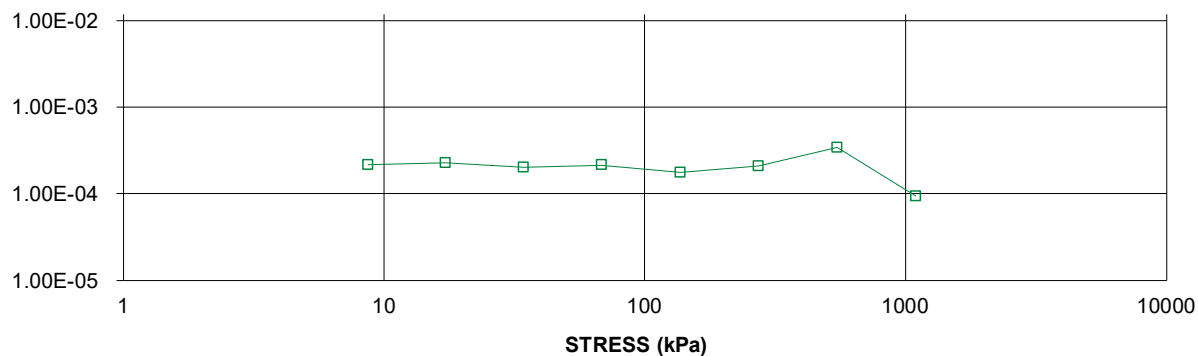
COEFFICIENT OF CONSOLIDATION
(cm²/s)

COEFFICIENT OF CONSOLIDATION (cm²/s) VS PRESSURE (kPa)
21-4 Sa 14A



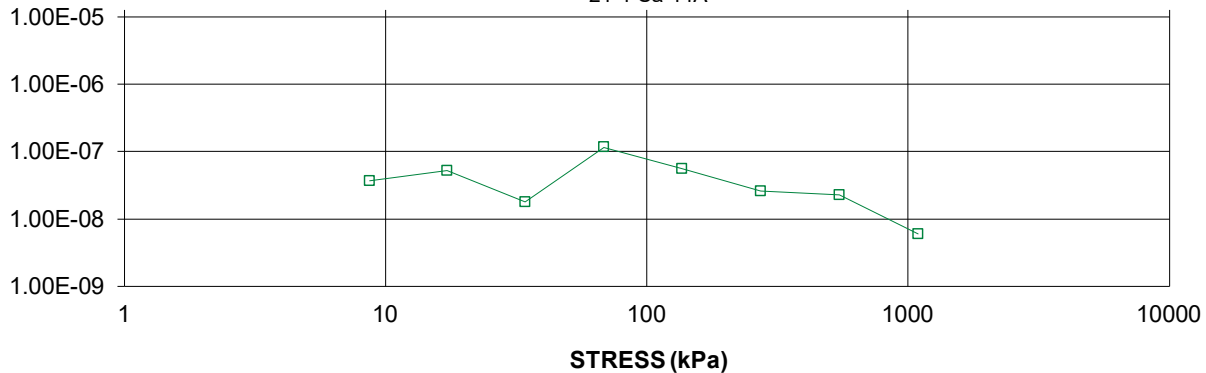
VOLUME COMPRESSIBILITY
(m²/kN)

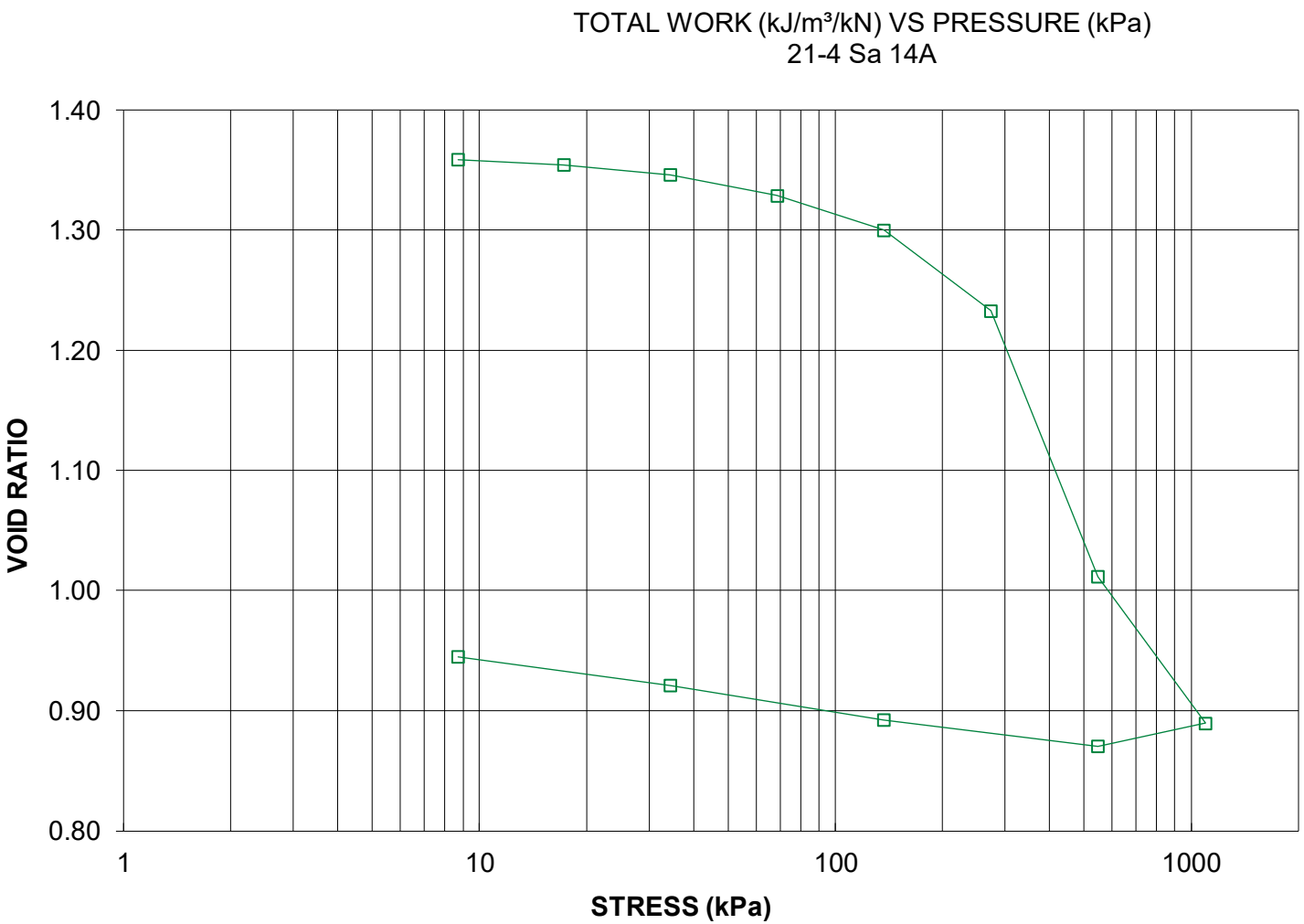
MODULUS OF VOLUME COMPRESSIBILITY (m²/kN) VS PRESSURE (kPa)
21-4 Sa 14A

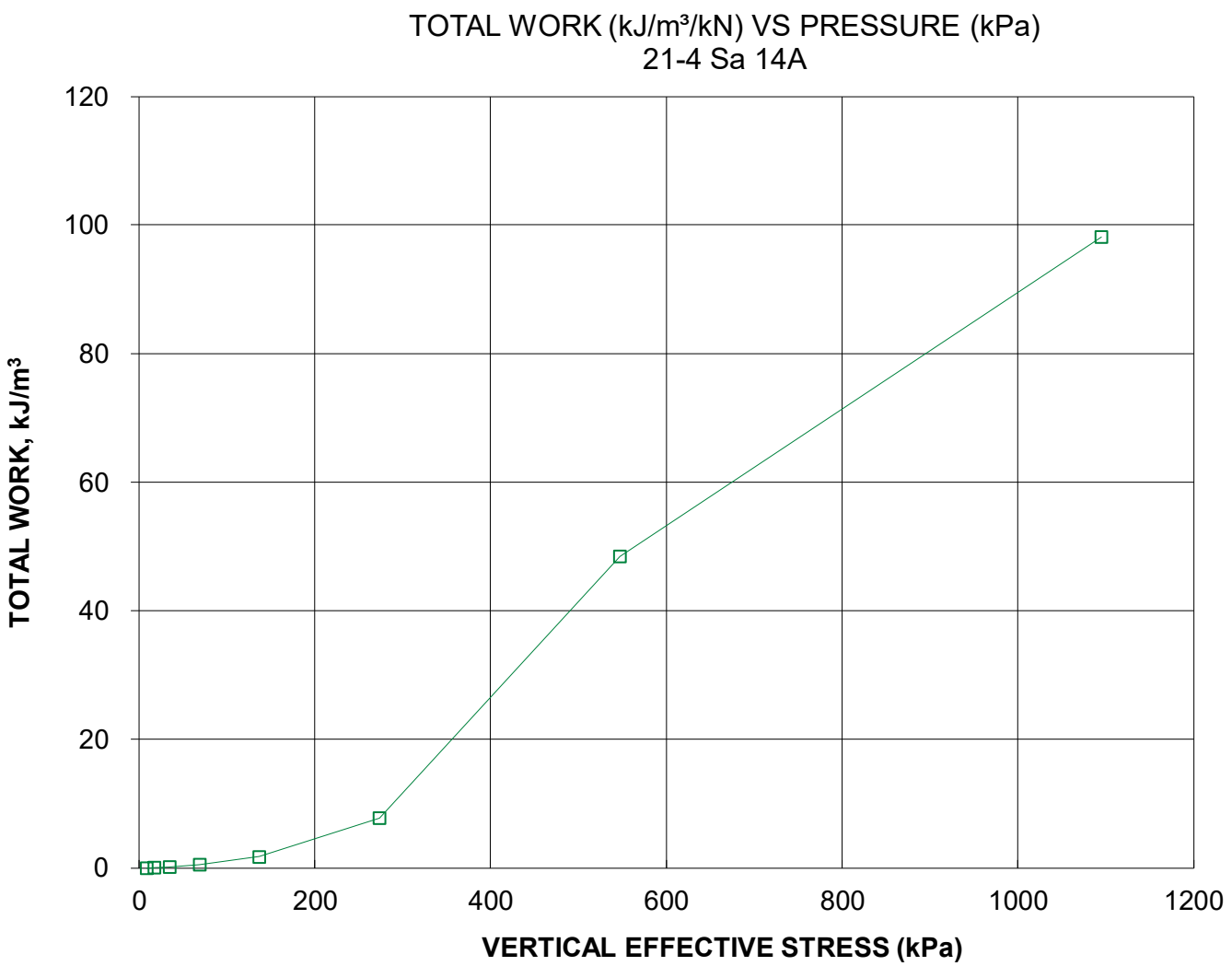


HYDRAULIC CONDUCTIVITY
(cm/s)

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		



Prepared By: TG

Checked By: MT

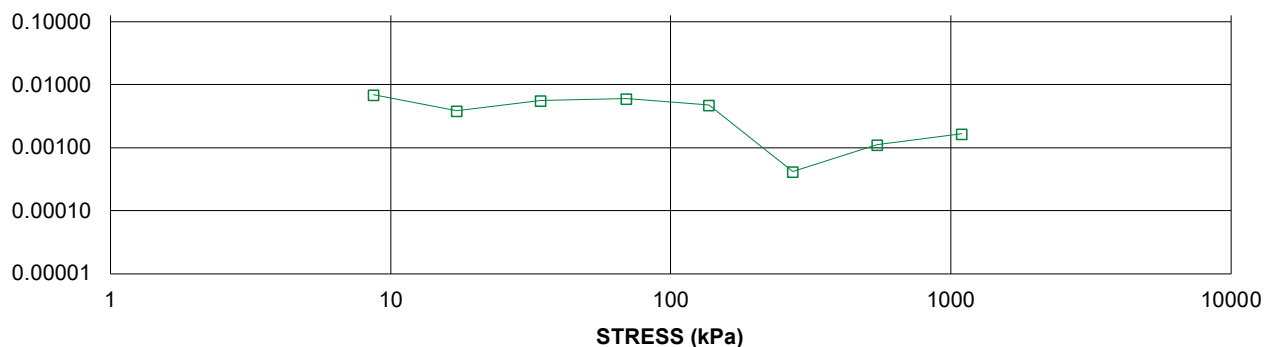
CONSOLIDATION TEST SUMMARY

FIGURE B-8

Pg. 2 of 4

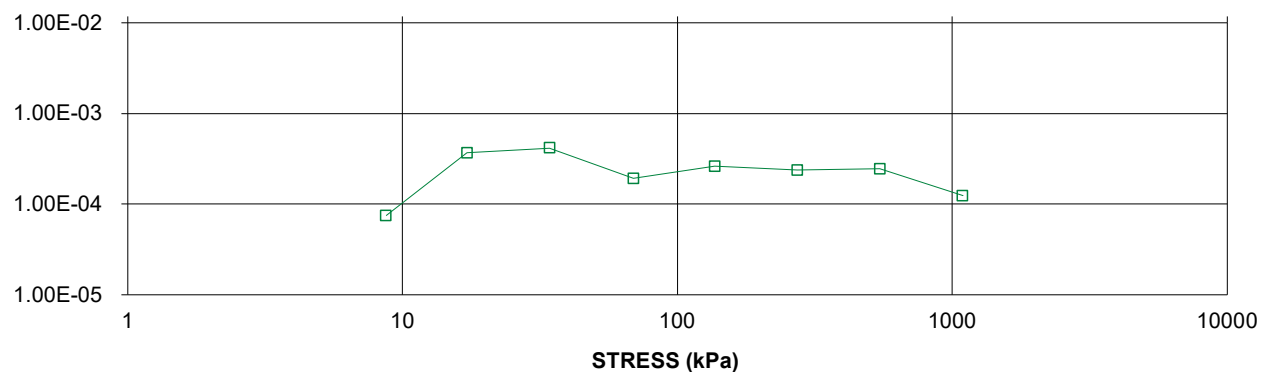
COEFFICIENT OF CONSOLIDATION
(cm²/s)

COEFFICIENT OF CONSOLIDATION (cm²/s) VS PRESSURE (kPa)
21-4 Sa 14B



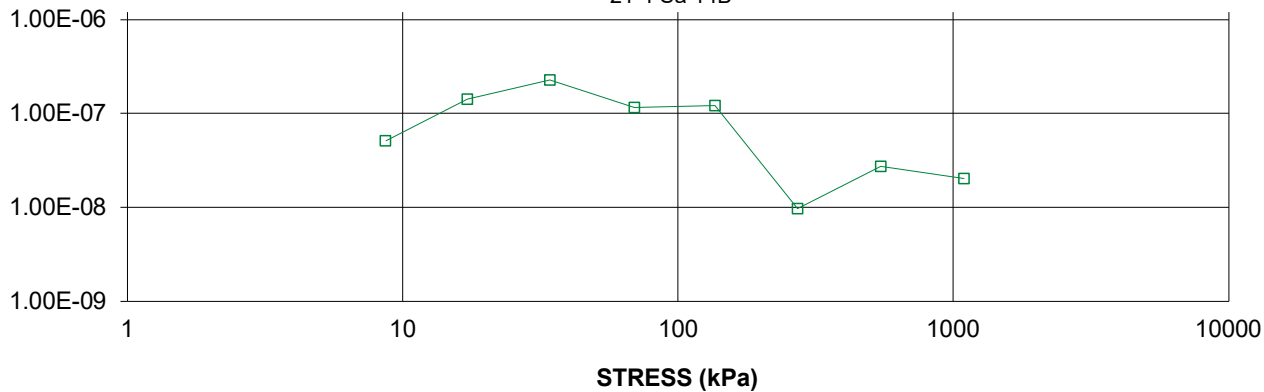
VOLUME COMPRESSIBILITY
(m²/kN)

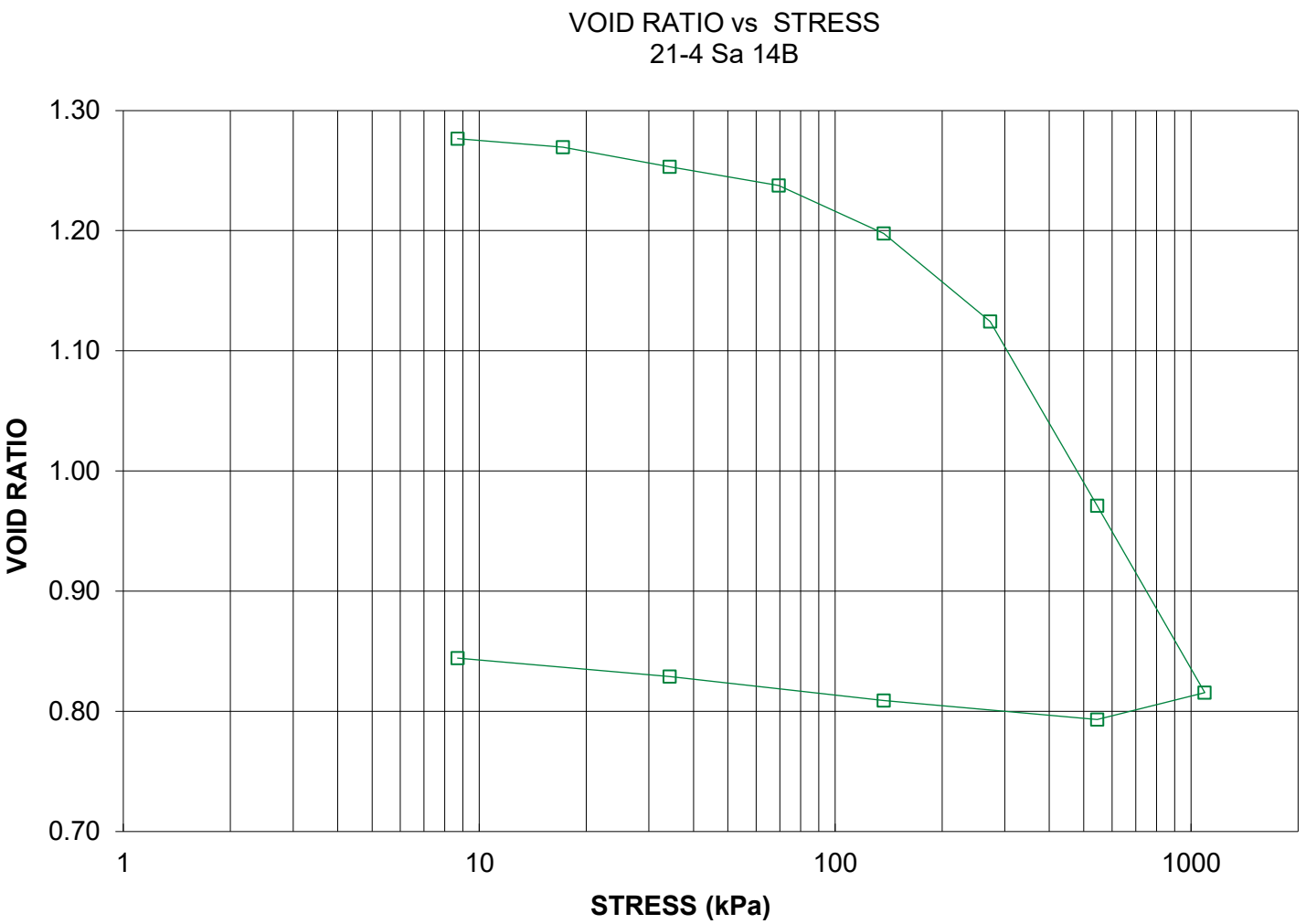
MODULUS OF VOLUME COMPRESSIBILITY (m²/kN) VS PRESSURE (kPa)
21-4 Sa 14B

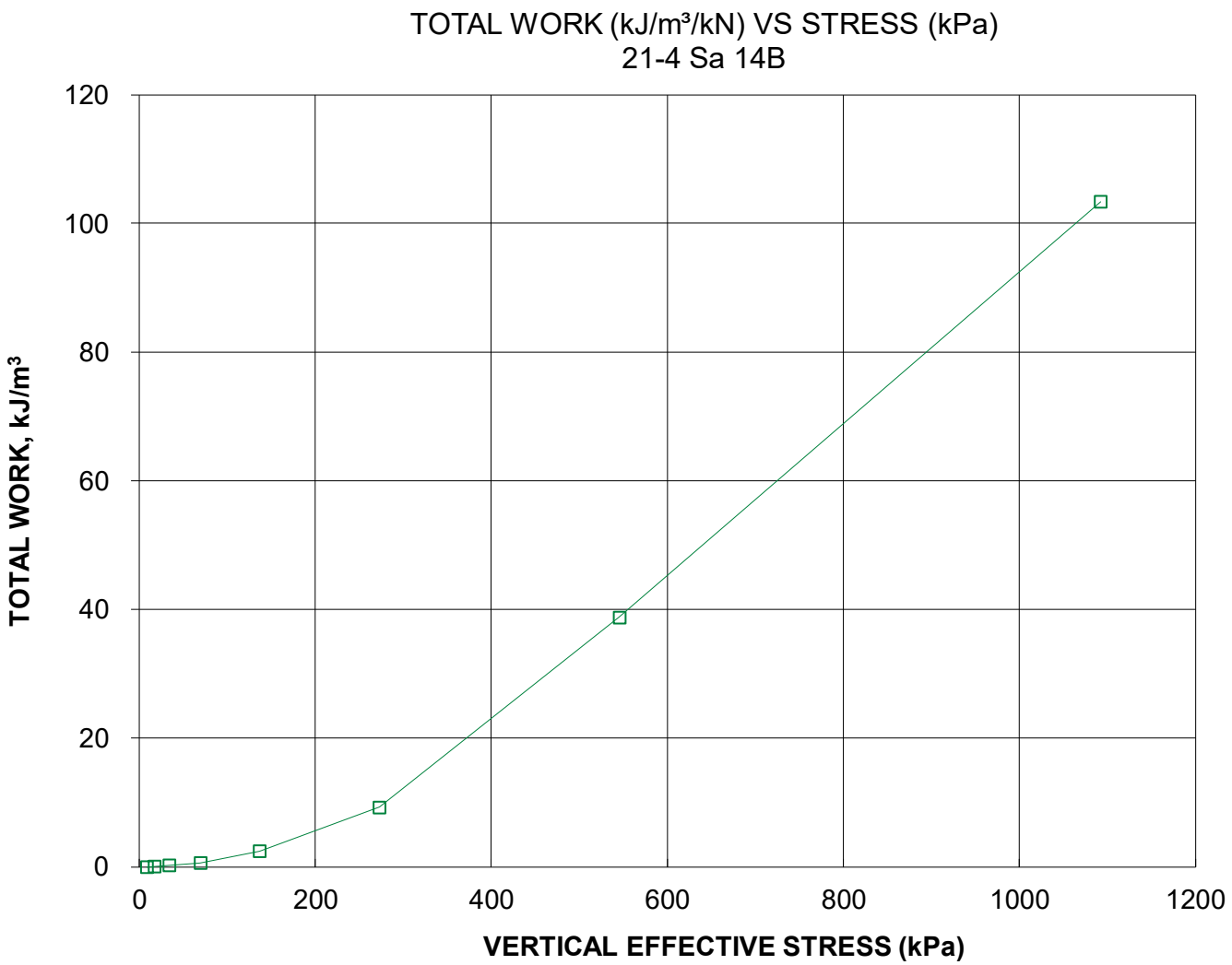


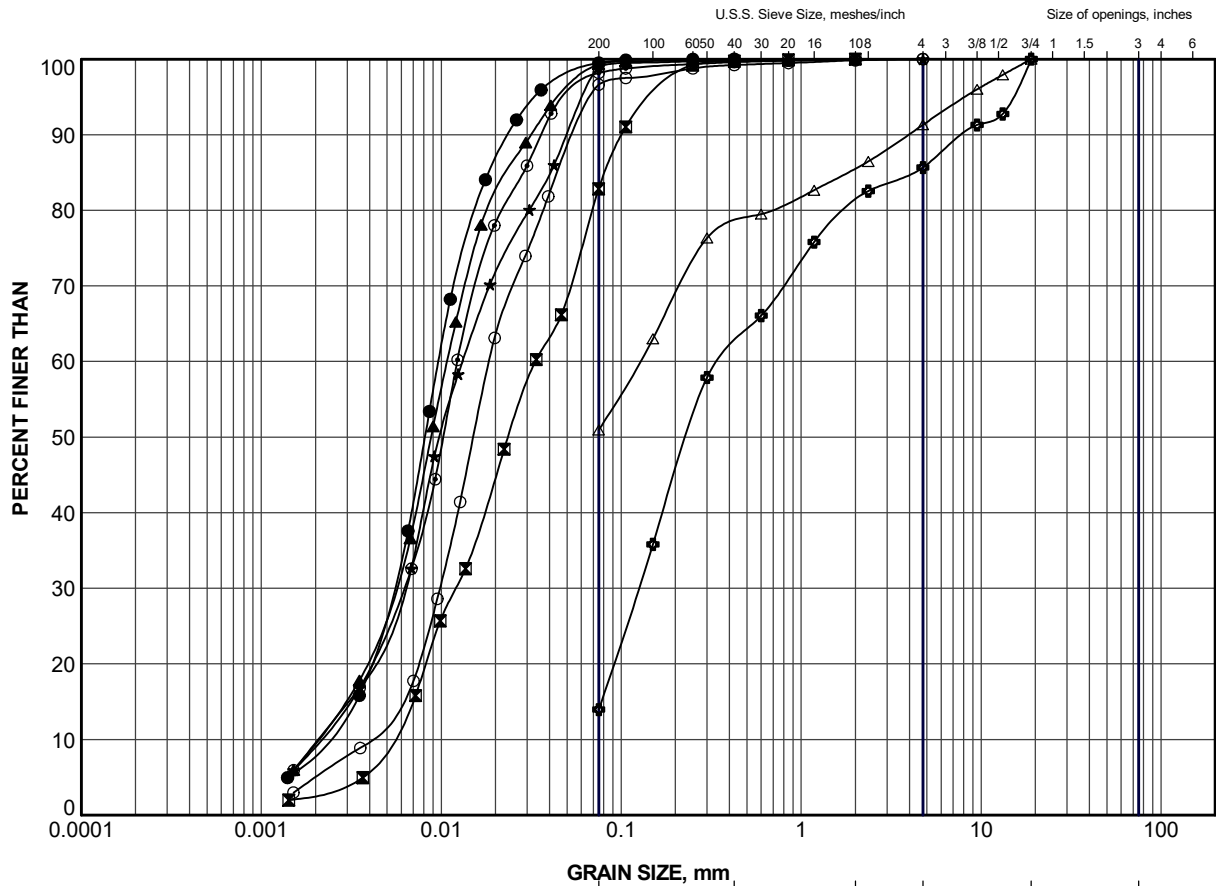
HYDRAULIC CONDUCTIVITY
(cm/s)

HYDRAULIC CONDUCTIVITY (cm/s) vs PRESSURE (kPa)
21-4 Sa 14B










CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	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 SAND (SW)					
PROJECT No.			20253807			FILE No.			20253807.GPJ		
DRAWN	TR	Jul 2021	SCALE	N/A	REV.	FIGURE B-9					
CHECK	TB	Jul 2021									
APPR	MT	Jul 2021									
 GOLDER MEMBER OF WSP SUDBURY, ONTARIO											

APPENDIX C

Non-standard Special Provisions and Operational Constraints

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 3 m in length measured parallel to the highway embankment toe and backfilled immediately.
- Provisions for traffic control measures shall be available on site to maintain the safe operation and traffic flow of Highway 17.

CELLULAR CONCRETE - Item No.

Special Provision

1.0 SCOPE OF WORK

This specification covers the requirements for the supply and placement of lightweight cellular concrete used as lightweight fill in accordance with the contract drawings. The cellular concrete shall be placed in dry conditions and above the groundwater table.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 517	Dewatering
OPSS.PROV 539	Temporary Protection System

Ontario Provincial Standard Specifications, Material

OPSS 1301	Cementing Materials
OPSS 1302	Water
OPSS.PROV 1303	Admixtures for Concrete
OPSS.PROV 1350	Concrete – Materials and Production

American Society for Testing and Materials (ASTM)

ASTM C 150	Portland Cement
ASTM C 869	Standard Specification for Foaming Agents Used in Making Preformed Foam for Cellular Concrete
ASTM C 796	Standard Test Method for Foaming Agents for Use in Producing Cellular Concrete Using Preformed Foam
ASTM C 495	Standard Test Method for Compressive Strength of Lightweight Insulating Concrete

Ministry of Transportation Publication:

LS-407 Method of Test for Compressive Strength of Moulded Cylinders

Designated Source of Material:

List # 2.35.30 Lightweight Fill Material

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Cellular Concrete: Cellular concrete is a material with flowable consistency during placement, produced by the substitution of a uniform cellular structure of air cells (voids) for some or all of the aggregate particles found in standard concretes.

Contractors Engineer: A Foundation Engineer, licensed in the Province of Ontario, with a minimum of five years experience related to the design and/or construction of cellular concrete of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality assurance services for the work at a minimum of two projects of similar scope to the Contract.

Manufacturer: Means the firm who supplies the cellular concrete.

Production Lot: The quantity of cellular concrete produced for a continuously placed lift of cellular concrete.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.01 Prequalification of Cellular Concrete Product

Prior to the commencement of work, the Contractor shall submit to the Contract Administrator a statement from the Supplier verifying that the Supplier has successfully completed the MTO Prequalification Process for Lightweight Fill and confirming that the product has been prequalified for use as lightweight fill by the MTO.

4.02 Qualifications

The Contractor shall submit a resume of the contractor's experience in the production and placement of cellular concrete. The resume shall include the qualifications of contractor's superintendent and/or foreman. The resume shall be submitted to the Contract Administrator for information purposes a minimum of three weeks prior to the start of cellular concrete construction.

The Contractor shall have satisfactorily completed at least five projects of similar nature and complexity during the last three years.

Workers, including the Contractor's superintendent and/or foreman, shall be qualified by the foaming agent manufacturer for production of foam for use in cellular concrete and thoroughly trained and experienced in the production and placement of cellular concrete.

At the commencement of the work, the Contractor shall have on site a representative of the cellular concrete supplier to advise on recommended construction procedures. The Contractor shall also have a representative of the cellular concrete supplier on site during the placement of cellular concrete.

4.03 Submission of Shop Drawings

At least eight weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and a method statement that provides full details of the materials and construction procedure.

The contractor shall submit method statements with full details of the following:

- a) Foundation excavation and preparation;
- b) Forming each cellular concrete lift;
- c) Placement of cellular concrete including lift thickness and plan dimensions on a lift-by-lift basis;
- d) Control of cellular concrete placement to minimize discharge of liquid slurry or foam breakdown products into lifts;
- e) Verification that liquid slurry has been fully removed from discharge hoses or pipes between plant and discharge point prior to each placement event;
- f) Obtaining and measuring the required surface slope at the top of the cellular concrete mass;
- g) Protecting the top cellular concrete surface from damage during pavement structure placement and compaction;
- h) Placement of pavement subbase material;
- i) Placement of side slope cover;
- j) Placement of cellular concrete;
- k) Placement of bond break between concrete culvert wall or wing walls and cleaning of such leakage or spillage from culvert wall facing;
- l) Protection of cellular concrete during precipitation and freezing weather; and
- m) Quality Control Plan.

Submittals shall also include:

- a) Mix design(s) identifying proportions of foaming agents, water, air, and Portland cement;
- b) Expected density and rate of strength gain showing data from 7-day and 28-day tests on similar mix designs;
- c) Expected in-place volumetric change during and following placement and during curing;
- d) Site layout of mixing plant, delivery hoses and discharge points;
- e) Design calculations and basis for design including:
 - a. internal shear stability;
 - b. unit weight;
 - c. internal and external drainage design details to address ice formation, and frost protection.
- f) Results of Quality Control (QC) testing (as outlined further in Section 8.01).

Production and placement of cellular concrete for this contract shall not commence until the Contract Administrator has received, reviewed, and approved the shop drawings.

4.04 Submission of Certificate of Conformance

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Contractors Engineer upon completion of the cellular concrete work and prior to any other backfilling. The Certificate shall state that the work has been carried out in conformance with the contract documents, specifications and/or stamped working drawings.

4.05 Submission of Environmental Protection Strategy

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of an environmental protection strategy as specified under Section 7.05.

5.0 MATERIALS

5.01 Cementing Materials

Cementing materials shall be according to OPSS 1301. Supplementary cementing materials shall not be used.

5.02 Water

Water shall be free of contamination and any deleterious substance. Water shall conform to OPSS 1302.

5.03 Admixtures

Admixtures shall conform to OPSS.PROV 1303.

5.04 Foaming Agents

Foaming agents shall conform to the requirements of ASTM C 869 when tested in accordance with the provisions of ASTM C 796. The Subcontractor shall be pre-qualified and approved in writing by the foaming agent manufacturer referencing this Project.

5.05 Cellular Concrete Properties

Cellular concrete shall be prequalified by the MTO Lightweight Fill Committee Prequalification Process and have the following properties:

- a) Minimum unconfined compressive strength at 28 days of 1 MPa.
- b) Wet cast density of 430 kg/m³ (4.2 kN/m³) (+/-10%)
- c) Must not contain any other waste or process by-product including fly ash.
- d) In place density of 430 kg/m³ (4.2 kN/m³) as measured on cores obtained from the full lift thickness

6.0 EQUIPMENT

The specialized batching, mixing, and placing equipment shall be automated and certified for the purpose by the manufacturer of the cellular concrete material. Dry-mix equipment must be able to receive bulk cement and produce over 100 cubic metres per hour on-site, continuously, from one piece of equipment, and pump through hoses or pipes up to a flat lineal distance of 200 metres. Bulk cement shall be weighed on a scale that operates within a tolerance of one and one-half percent (1.5%) per batch. Wet-mix equipment must be able to receive slurry on-site into the equipment and process it continuously during ready-mix supply, and pump through hoses or pipes up to a flat lineal distance of 200 metres.

Cellular concrete must be pumped by a positive displacement pump (Peristaltic or similar). A foam generator shall be used to continuously produce pre-formed foam which shall be injected and mixed with the cementitious slurry downstream of the positive displacement slurry pump. The equipment shall be calibrated to produce a precise and predictable volumetric rate of foam with stable uniform microbubbles.

7.0 CONSTRUCTION

7.01 Excavation and Subgrade Preparation

Excavation to the subgrade level shall be carried out to the design elevations and horizontal and vertical limits shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be sub excavated and replaced with Granular 'A' or Granular 'B' Type II material.

The prepared subgrade shall be good competent level ground. Water, snow, and ice must be removed from the area prior to placement.

The excavation may need to be completed in limited widths to maintain a stable temporary side slope prior to and during placement of the cellular concrete.

7.02 Dewatering

The prepared subgrade shall be free of standing water during placement of cellular concrete and until backfill is placed on top of the cellular concrete. If necessary, dewatering shall be continuous during placement of materials.

Dewatering shall be according to OPSS.PROV 517.

7.03 Protection System

The construction of all protection schemes shall be according to OPSS.PROV 539 and paid for under the appropriate tender item. Where the stability, safety or function of an existing roadway, railway, other works, or proposed works may be impaired due to the method of operation, such protection as may be required shall be provided by the Contractor.

7.04 Placement

The placement of cellular concrete shall be under the direct supervision of the Contractors Engineer.

The Contractors Engineer shall be on site to oversee the placement of the cellular concrete and to verify that the cellular concrete is being supplied and placed in accordance with the Contract Documents.

A Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the placement of the cellular concrete.

Any items to be fully or partially encased in the cellular concrete shall be properly set and stable prior to the installation of the cellular concrete. Where required, formwork shall be designed and installed to withhold cellular concrete and may require lining with poly sheeting or similar impermeable membrane to prevent leakage.

Placement of cellular concrete during freezing conditions should be avoided.

Cellular concrete may be placed during freezing conditions, provided measures are taken to prevent damage to the cellular concrete until sufficient strength has been attained. Care should be taken to avoid freezing before initial set and insulating systems or heat shall be provided to prevent freezing of the cellular concrete. Cold weather protection shall be provided in accordance with OPSS.PROV 1350.

If temperatures above 38°C are expected during casting, special precautions shall be considered including casting before dawn and use of additives to maintain moisture in the mixture and minimize plastic-shrinkage cracking.

Cellular concrete must not be placed during precipitation and shall be protected from precipitation until initial set has been achieved.

Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling. The maximum flat line pumping distance through hoses or pipes shall be limited to 200 m regardless of wet-mix or dry-mix placement methods. Initial discharge of cellular concrete that has accumulated in the discharge lines during prior placements or any cellular concrete mix that has not been fully aerated shall be wasted prior to discharge into the intended lift. Cellular concrete shall not be discharged into the intended lift after the foam generator has been turned off.

The maximum lift thickness shall be determined based on density and any other considerations that may affect placement. The depth of the cellular concrete lifts shall be designed to prevent any thermal damage to the cellular concrete caused by heat of hydration. Cellular concrete shall be cast in a formed area within 1 to 2 hours, to permit undisturbed curing. Foot traffic within the cellular concrete mass shall not be permitted.

Finished surface elevation shall be within ± 25 mm of the design grades shown on the drawings. Cellular concrete surface slopes greater than 1%, if required, shall be created in accordance with the Contractor's approved method statement and shop drawings. Flat benches or slopes of less than 2% will not be permitted for cellular concrete used as backfill. Sloped concrete caps, trimming of cellular concrete or other pre-approved methods shall be used to obtain appropriate drainage slopes.

Loading of, or traffic on, the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfilling can commence when cellular concrete supports foot traffic without leaving an indentation.

7.05 Environmental Protection

The Contractor shall handle materials and conduct the work in a manner that will ensure protection of the natural environment and prohibit cellular concrete from entering surface or ground water. The Contractor shall take measures as necessary to prevent the material from entering the natural environment and/or leaking outside of the intended placement location, and, shall have established methods for stopping flow of the product as required, and for prompt remediation of any leaks or spills. These measures and any other contingency planning requirements shall be documented in an Environmental Protection Strategy.

8.0 QUALITY CONTROL

8.01 Sampling Frequency and Methods

Fresh cellular concrete shall be collected for density testing once per production run, or once for every 25 cubic metres, or once per 15 minutes, whichever is more frequent. The unit weight shall be maintained within $\pm 10\%$ of the design unit weight and shall be adjusted as required to obtain the specified density at the point of placement. Routine density testing shall be carried out on samples of fresh cellular concrete collected from the standing pool of fresh cellular concrete and directly from the delivery hose discharge point with points of collection recorded for each sampling and testing instance.

Cellular concrete samples shall also be captured, cured, and tested at the point of placement to verify the specified compressive strength and the dry unit weight. One sample shall be taken for each placement lift, or

every 100 m³, whichever is more frequent. One sample is comprised of one set of six cellular concrete cylinders, with cylinders cast in 75 mm by 150 mm cylindrical plastic molds. Cellular concrete cylinders shall be cured and tested for density and compressive strength as per ASTM C495 and LS 407. One cylinder shall be collected from the fresh cellular concrete by hand from the full depth of the standing pool of cellular concrete, one directly from the delivery hose discharge point during production placement with the remaining four in accordance with the approved quality control plan.

Three core samples shall be obtained for each placement volume or lift, or every 100 m³, whichever is more frequent prior to placement of subsequent lifts. Each core sample shall be measured for weight and volume, wrapped in plastic and stored for curing with other quality control test cylinders.

8.02 Production Data Records

The following data shall be recorded for each placement, lift or 100 m³ of in-place cellular concrete, whichever is more frequent:

- a) Confirmation that fluids have been removed from hoses and pipes prior to placement;
- b) Lengths of hoses and pipes from plant to discharge point;
- c) Times of foam generator start and stop;
- d) Times of slurry flow start and stop;
- e) Quantity of materials wasted from hoses and pipes prior to and following production discharge;
- f) Fluid concrete sampling times, locations, sample numbers and curing conditions;
- g) Core sampling times, locations, sample numbers and curing conditions;
- h) Production volume discharged based on flow rates, wasted materials, and plant input quantities;
- i) Dimensions and volumes of placement based on physical measurements at completion of placement and prior to subsequent lifts or backfilling;
- j) digital photographs of each lift;
- k) Measurements and photographs of slope of cellular concrete surface at completion; and
- l) Results of all laboratory tests on cured cellular concrete cylinders and cores.

8.03 Acceptance/Rejection

Failure of any one of the samples to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the production lot or any alternative mitigation accepted by the Contract Administrator shall be at the Contractor's expense.

9.0 MEASUREMENT FOR PAYMENT

Measurement for payment shall be the calculated neat volume in cubic meters specified to consist of cellular concrete within the theoretical lines and grades shown in the stamped working drawings. In no case will placed volumes be determined by multiplying the known volume of slurry by the ratio of slurry density to average cellular concrete density (expansion ratio).

10.0 BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, materials, and equipment to do the work as described above.

EMBANKMENT CONSTRUCTION AND PRELOAD

Operational Constraint

Scope

Embankment Construction – Highway 17 STA 14+300 to 14+450 and S-E Ramp STA 10+000 to 10+075

The preload period given is estimated to be 6 months for bidding purposes only. The actual construction schedule will be determined by the results of the foundation monitoring program in place during construction.

Fill placement for embankment construction shall not commence sooner than five (5) working days following completion of installation of all monitoring instrumentation, including notification and submission of required information (to the Contract Administrator) for all instrumentation. In any case, fill placement shall not commence before establishment of baseline readings. Baseline readings shall be conducted no sooner than three (3) days following notification of completion of installation of instrumentation and receipt of required installation information. If the baseline monitoring shows that the baseline has been established (i.e., consistent readings reflecting initial conditions obtained over the three (3) day period), embankment construction may commence. If the baseline is not established within a three (3) day period, additional daily readings shall be completed until three (3) consistent readings on consecutive days have been obtained prior to commencement of embankment construction.

The Contractor shall confirm that the elevation of the top of the preload is within 150 mm of the design top of preload. Elevations shall be provided to the Contract Administrator within five (5) working days of placement to the preload elevation. The Contractor shall keep records of the thickness of each layer of fill placed and provide these records to the Contract Administrator within five (5) working days of reaching the top of each layer.

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING INSTRUMENTS– Item No.

Special Provision

1.0 SCOPE

The Contractor shall retain a Foundation Engineering consultant registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical Specialty – Medium Complexity", to undertake the supply and installation of geotechnical settlement monitoring instrumentation (settlement plates, nail pins, and temporary benchmarks) and for providing baseline measurement readings.

The Contractor and Foundation Engineering Consultant shall also take, record, and distribute all appropriate and timely survey measurements during instrument installation to the Contract Administrator in a timely manner (i.e., as-constructed documentation).

1.1 General Scope

This general special provision and the other item-specific special provisions contain the requirements for the supply and installation of the following geotechnical monitoring instrumentation:

- Settlement Plates (SP); and,
- Nail Pins (NP).

This general special provision also contains the requirements for the supply and installation of Temporary Survey Benchmarks (TBM) related to the geotechnical monitoring instrumentation.

1.2 Purpose

The purpose of these instruments and equipment is to monitor the progress of the settlement of the embankment widening using the SPs. The NPs will be used to monitor the settlement of the existing structural culvert and wing walls. The purpose of the temporary survey Benchmarks is to provide non-settling references for the surveying of the monitoring instruments.

The duration of the preloading period will be controlled by the instrumentation readings, as specified elsewhere in the Contract Documents. The completed, preloaded embankments shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of consolidation of the foundation soil has been achieved.

2.0 REFERENCES

2.1 General

When the Contract Documents indicate that provincial oriented specifications are to be used and there is a provincial oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be considered to be the OPSS listed, unless use of a municipal oriented specification is specified in the Contract Documents.

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

Ontario Provincial Standard Specifications, Construction

OPSS 905 Steel Reinforcement for Concrete

Ontario Provincial Standards Specifications, Material

OPSS1010 Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production

OPSS 1301 Cementing Materials

OPSS 1801 Corrugated Steel Pile (CSP) Products

Ontario Water Resources Act RRO 1990:

Regulation 903 Wells

2.2 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Reports for this Contract.

Foundation Investigation Report – Embankment Widening - STA 14+300 to STA 14+725
Township of Denison, Highway 17 and Municipal Road 55 West Junction Intersection
Improvements, Agreement No. 5019-E-0026, GWP 5032-19-00

Foundation Investigation Report – Extension of Fairbanks Creek Culvert (Site No. 46X-
0298/CO), Township of Denison, Highway 17 and Municipal Road 55 West Junction
Intersection Improvements, Agreement No. 5019-E-0026, GWP 5032-19-00

3.0 DEFINITIONS

Contractor means the Contractor and his Geotechnical Consultant.

Equal shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

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

Foundation Engineering Specialist means a consultant retained by the Contract Administrator with MTO classification of “Geotechnical (Structures and Embankments) - Medium Complexity”, to undertake the subsequent monitoring program after installation of monitoring instruments.

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

Nail Pin means a pin or bar installed at the defined locations and fixed to the existing structure(s) for the purpose of settlement monitoring.

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

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

4.0 SUBMISSION REQUIREMENTS

4.1 Submission Requirements

4.1.1 Notification

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

4.1.2 Installation Methods

The Contractor shall submit details of the proposed installation methods including locations and types of the data acquisition system(s), monitoring enclosure(s) and/or means to keep instruments free and clear of equipment, snow, on-site activities to allow for readings to be taken by others throughout monitoring period, temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

5.0 MATERIALS

5.1 Materials for Temporary Benchmarks (TBM)

The Contractor shall supply all materials and equipment required for the installation of the Temporary Benchmarks for baselining and for the duration of the monitoring program for the full preload period.

5.1.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm, supplied in lengths as required to complete the installation as described in Section 6.2.2.

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

5.1.2 Sand

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

5.1.3 Rod Anchor Grout

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

5.1.5 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.2 Settlement Plates (SP)

5.2.1 Plate

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

5.2.2 Rod

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

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

5.2.3 Friction Reducing Sleeve

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

5.2.4 Protective Surround

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

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

5.3 Nail Pins (NP)

5.3.1 Pin

The Contractor shall supply a 25.4 mm minimum diameter reinforcing steel bar (OPSS 905) cut 0.15 m long or equivalent.

The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and repeated.

5.3.2 Concrete

The Contractor shall supply concrete (OPSS Prov. 1350) of minimum 25 MPa compressive strength and set

time sufficient to secure the Nail Pin within two (2) days of pouring. Alternatively, the nail pin can be epoxy grouted within a pre-drilled hole within the concrete culvert structure.

6.0 CONSTRUCTION

6.1 Monitoring Instrument Installations

6.1.1 Drawings

Reference shall be made to the following drawings that are contained elsewhere in the Contract Documents:

- Monitoring Instrument Location Plan; and,
- Typical Instrument Installation Details.

6.1.2 Quantities and Locations of Instruments

The quantities and approximate location of instruments are presented in Table 1A and are shown on the Contract Drawings. The final locations shall be “field fit” by the Contractor, with the agreement of the Contract Administrator, to take account of any utilities that may be present, construction operations, and safe access conditions.

Table 1A – Instrument Quantities and Locations

Monitoring Section	Quantities	
	SP	NP
Hwy 17 Shoulder STA 14+300 to 14+450	4	3
Proposed S-E Ramp	2	--
TOTAL:	6	3

6.1.3 Materials and Equipment

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

6.1.4 Instrument Location

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

6.1.5 Underground Utilities

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

6.1.6 Marking and Labelling

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

Instruments shall be clearly labelled in the field, with each instrument having a unique identifier as contained in the other item-specific special provisions. The labelling shall remain legible for the entire duration of monitoring.

6.1.7 Protection of Instruments

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

6.1.8 Survey Personnel

Surveying to establish the benchmarks and other elevations shall be carried out by a registered surveyor with appropriate equipment. The surveyor shall be retained by the Contractor.

6.1.9 Accuracy of Surveying for Elevations

Elevations shall be surveyed to a repeatable accuracy and precision of ± 2 mm or better.

6.1.10 Installation Program

The instruments shall be installed prior to the commencement of the embankment construction. Table 1B gives a summary of the installation schedule requirements.

Table 1B – Instrument Installation Program

Instrument Type	Instrument Location	Start Installation	Finish Installation
SP	Levelling pad for plate to be installed on top of Granular B Type II backfill.	After stripping / subexcavation and backfill with Granular B Type II, but prior to further fill placement.	At completion of embankment construction to preload grade.
NP	On culvert/wingwalls as identified herein.	After stripping / subexcavation and backfill with Granular B Type II, but prior to further fill placement.	At completion of embankment construction to preload grade.

6.2 Benchmark Installation

6.2.1 Number and Locations

The minimum number and approximate locations of the Benchmarks are to be determined by the Contractor and his Registered Surveyor, in collaboration with his Foundation Engineering consultant and in conjunction with the Contract Administrator and the Foundation Engineering Specialist. For bidding purposes, it is assumed that 2 benchmarks are required to be anchored 1 m depth into bedrock or at least to a 15 m depth. The number and locations of Benchmarks shall be determined in the field to satisfy the following conditions:

- Direct sighting is possible from all instruments to at least one Benchmark.
- Each Benchmark is located in an area that will not experience a change in loading (due to grade raise or excavation) that could induce settlement or heave in the ground in which the Benchmark is installed (i.e. non-settling benchmark).
- Each Benchmark is located in such a way to minimize interference with and damage by construction activities and/or traffic.
- The rod anchor elevation shall be adjusted in the field to extend (as a minimum) approximately 1 m into soils having Standard Penetration Test 'N' values of greater than 25 blows per 0.3 m of penetration or better.

Intermediate tie-in points may be required as deemed necessary by the surveyor and shall be tied into the temporary benchmarks during each reading.

6.2.2 Installation

The Contractor shall install Benchmarks in accordance with the following:

6.2.3 Borehole

The borehole shall be advanced to rod anchor elevations controlled by the Standard Penetration test "N" values given above, using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve, and rod anchor. The sides of the borehole shall be stable, and the borehole shall be free of drilling mud and debris.

6.2.4 Rod

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

6.2.5 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding native soil by grouting the bottom 0.5 m of the borehole to form a concrete/soil or concrete/rock anchor.

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

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

6.2.6 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

6.2.7 Installation Details

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

6.3.0 Settlement Plates

6.3.1 General

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

The SPs shall be placed on the Granular B Type II backfill at the approximate elevation identified in Table 2A. As embankment construction proceeds the settlement measuring rods shall be extended above the new top of embankment.

Table 2A – Settlement Plate Locations

Alignment	SP Designation	Station	Offset from Centreline	Approximate Settlement Plate Founding Elevation (m)	Estimated Thickness of Embankment Fill Above Settlement Plate (m)
Existing Hwy 17 EBL	SP1	14+305.0	5.0 m Rt	241.9	2.1
	SP2	14+363.0	11.0 m Rt	241.4	2.1
	SP3	14+393.0	9.0 m Rt	243.0	2.1
	SP4	14+444.0	9.0 m Rt	242.3	2.3
Proposed S-E Ramp	SP5	10+050.0	7.5 m Rt	240.1	3.2
	SP6	10+080.0	8.0 m Rt	240.5	2.7

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

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

6.3.2 Plate

The settlement plates shall be installed horizontally on top of the Granular B Type II backfill.

6.3.3 Rod

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

6.3.4 Friction Reducing Sleeve

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

6.3.5 Extension of Rod

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

6.3.6 Protective Surround

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

6.4.0 Nail Pins

6.4.1 General

The locations of the NPs are shown on the Contract Drawings and are given in Table 2B.

One (1) NP shall be installed on the top of the existing culvert, and one (1) NP shall be installed at each end of the existing wingwall.

Table 2B – Nail Pin Locations

Structure	Culvert Station ¹	Nail Pin Location
Hwy 17 Fairbanks Creek Culvert	14+378.6	Southern wingwall
	14+384.0	Culvert centreline, approximately 0.5 m from east end of culvert
	14+389.4	Northern wingwall

NOTE: ¹Stations for the nail pins are approximate.

6.5 Monitoring Program

6.5.1 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days after the completion of installation of Benchmarks, Settlement Plates and Nail Pins and after completion of embankment fill placement up to design preload level adjacent to each instrument.

6.5.2 Reporting

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

6.5.2.1 Temporary Survey Benchmarks

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

- TBM Northing and Easting in MTM NAD 83 coordinates;

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

6.5.2.2 Settlement Plates

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

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

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

6.5.3 Monitoring

The Contractor shall meet with the Contract Administrator and staff responsible for the ongoing monitoring immediately after installation of the instruments and before the start of embankment construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments.

Monitoring by the Contract Administrator's representative for the baseline readings shall commence within seven working days after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the construction of the embankments, and for up to approximately 6 months following the completion of construction to the preload grade. The contractor shall ensure the access to the instruments is maintained over the duration of the monitoring period.

6.5.4 Decommissioning of Instruments

At the end of the monitoring period, the Contractor shall decommission all the temporary survey Benchmarks and tie-in points by removing the rod and friction-reducing sleeve to at least 1.5 m below grade by excavating and backfilling with compacted granular fill in accordance with the specifications for fill placement.

At the end of the monitoring period, the Contractor shall decommission all Settlement Plates and Nail Pins, unless otherwise advised by the Contract Administrator.

7.0 QUALITY ASSURANCE – Not Used

8.0 MEASUREMENT FOR PAYMENT – Not Used

9.0 BASIS OF PAYMENT

9.1 Supply and Installation of Embankment Monitoring Equipment - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including the supply, installation and decommissioning of survey benchmarks, settlement plates and nail pins.

APPENDIX D

Foundation Engineering Specialist
Services for Contract Administration

FOUNDATION MONITORING PROGRAM

Foundation Engineering Specialist Services for Contract Administration Services

1.0 GENERAL

This work plan contains the requirements for the monitoring of the following geotechnical instruments:

- Settlement Plates (SP);
- Nail Pins (NP);

The instrumentation monitoring services include data collection; data reduction and reporting; and adherence to criteria used to assess the embankment performance based on the monitoring data collected from the instruments installed by others. The completion of the preload wait time (estimated to be 6 months) will be confirmed or revised by this foundation monitoring program to be performed by the Foundation Engineering Specialist as part of the Contract Administration services.

1.0.1 Specialist Qualifications

The Foundation Engineering Specialist services required for this assignment have been categorized as “Geotechnical Specialty – Medium Complexity”.

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

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

1.0.2 Services, Deliverables and Records

The Foundation Engineering Specialist shall:

- Review the monitoring program and, if deemed necessary, submit in writing to the Contract Administrator recommendations for modifications to the Monitoring Program;
- Meet with the Contractor and the Contractors Foundation Engineer in order to receive reports with details about installation of instruments installed by the Contractor, as specified in the Special Provision titled, “Supply and Installation of Embankment Monitoring Equipment”, included in the contract documents;
- Supply all materials and equipment that are required for the Monitoring Program;

- Take instrument readings, reduce data, prepare reports;
- Provide transmittal of instrumentation readings and reports to the Contract Administrator;
- Interpret instrumentation readings as needed for the purpose of on-going construction;
- Notify the Contract Administrator of required modifications to the construction procedures accordingly, if necessary. Interpretation shall include making correlations between instrumentation data and specific construction activities; and
- Notify the Contract Administrator if critical instrument readings (Review and Alert Levels), as specified herein, for any instrumentation are reached. Discuss as soon as possible (within 48 hours) with the Contract Administrator response action(s).

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

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

1.0.3 Submission of Foundation Monitoring Plan

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

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

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

1.0.4 Purpose

The purpose of this Monitoring Program is to monitor settlements in the foundation soils at select locations during and after construction of the embankment widening to preload grade associated with the new Highway 17 acceleration lane over soft ground for the following extents of embankment construction:

- Highway 17 EBL – STA 14+300 to STA 14+450
- S-E Ramp – STA 10+000 to STA 10+075

1.0.5 Drawings

Reference shall be made to the drawings titled, “Monitoring Instrument Location Plan” and “Typical Instrument Installation Details” included in the Contract Package.

1.0.6 Subsurface Conditions

The subsurface Conditions at the site are described in the Foundation Investigation Reports as referenced elsewhere in the Contract Documents.

1.0.7 Equipment Operation

Monitoring shall be conducted year-round. All monitoring equipment shall be maintained and rendered operational throughout the monitoring period. Site access to instruments shall be provided by the Contractor as per the Contract documents.

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

1.0.8 Reading Schedule and Frequency

The Foundation Engineering Specialist shall save and archive raw data in electronic and hard copy format.

Monitoring shall commence immediately after the installation of an instrument to the end of the preload period. The actual length of the monitoring period depends on the construction schedule, time between completion of instrument installation and completion of fill to preload grade, and the results of monitoring amongst other factors.

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

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

Table 1 – Minimum Monitoring Frequency

Stage	Frequency	Anticipated Number of Readings¹
Baseline Readings ²	Three readings on 3 consecutive days, no sooner than 7 days following installation	3
Immediately prior to start of embankment widening construction above settlement plate elevation	Once	1
During embankment widening construction	Once every 1 m thick fill lift within 20 m of the monitoring section	3
Preload Period (anticipated duration: about 6 months)	Weekly: First month Bi-weekly: Second month Monthly: Third month to end of preload	10

NOTE: ¹ Due to uncertainty of the construction schedule, the anticipated number of readings per monitoring section is an estimate of the site visits required to carry out the monitoring program described herein; however, the estimate is provided for bidding purposes only and additional visits may be required depending on actual construction schedule.

² Baseline Readings: value of instrumentation readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of the ground.

2.0 INSTRUMENTATION SPECIFIC REQUIREMENTS

2.0.1 Settlement Plates (SP) and Nail Pins (NP)

Surveying

The elevations of Settlement Plates and Nail Pins shall be surveyed to an accuracy of plus/minus 2 mm or better and shall be reported to the nearest millimetre.

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

Reporting

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

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

- A plot of settlement of the base of the embankments (SPs) versus time;
- A plot of settlement of the culvert structure (NPs) versus time;
- Fill height within 20 m of the instruments versus time;
- Plan view, cross section and profile sketches showing the top of fill location while the SPs and NPs were being surveyed.

Review and Alert Levels

If the maximum settlement measured exceeds the Review Levels in Table 2, the Foundation Engineering Specialist shall immediately inform the Contract Administrator and discuss response action(s). The Foundation Engineering Specialist shall submit a plan of action(s) to prevent Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached.

If the maximum settlement measured exceeds the Alert Levels in Table 2, the Foundation Engineering Specialist shall immediately inform the Contract Administrator and the Contract Administrator shall instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on the affected embankment until all the following conditions are satisfied:

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

Table 2 – Review and Alert Levels for Instruments Monitoring Settlements

Instrument Type	Location	Station	Offset from CL of Specified Lane (m)	Settlement Response Levels (mm)	
				Review	Alert
Settlement Plates (SP)	Highway 17	14+305.0	5.0 m Rt	25	50
		14+363.0	11.0 m Rt	50	75
		14+393.0	9.0 m Rt	25	50
		14+444.0	9.0 m Rt	25	50
Nail Pin (NP)	Fairbanks Creek Culvert	14+378.6 (near end of wingwall)	12.1 Rt	25	50
		14+384.0 (near centre of culvert)	10.0 Rt	25	50
		14+389.4 (near end of wingwall)	12.3 Rt	25	50
Settlement Plates (SP)	S-E Ramp	10+050.0	7.5 m Rt	75	100
		10+080.0	8.0 m Rt	75	100

NOTES: ¹ Stations for the nail pins are approximate. All nail pins shall be installed on the top of the existing culvert/wingwall based on the locations above.

3.0 CONTROL MONITORING LEVELS

General

The monitoring program will provide input for the control of the appropriate time for end of preloading (which may shorter or longer than the estimated 6 months) and removal of earth fill / placement of cellular concrete, and construction of permanent Fairbanks Culvert modifications.

Stabilization of Settlements due to Primary Consolidation

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

The anticipated total settlement that will occur and the required time for settlements due to primary consolidation to reach a degree of consolidation of 70% shall be assessed for each of the SPs and NPs using an appropriate method.

4.0 FINAL REPORT

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



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